NUMERICAL STUDY ON THE BEHAVIOUR OF REINFORCED CONCRETE COLUMNS EXPOSED TO FIRE INCLUDING THE COOLING PHASE

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ABSTRACTS: It has been proven that reinforced concrete (RC) structures subjected to fire are likely to collapse during cooling phase. However, many insights into this phenomenon have not been covered in both research and building codes/standards. This paper presents a quantitative study on the behaviour of RC columns subjected to natural fire including cooling phase, particularly the fire resistance R and burnout resistance DHP of columns. The study adopted Eurocode parametric fire model to simulate the natural fire curve and finite element models in SAFIR to simulate the during-and-post-fire responses of RC columns with various cross-sections. A parametric study on the effects of various factors (concrete cover, load ratio, and reinforcement ratio) on the DHP-R relationship is also presented.

KEYWORDS: reinforced concrete, structures, fire, cooling phase, SAFIR, numerical study.

1. INTRODUCTION

Both research and real-life events have proven that reinforced concrete (RC) structures exposed to fires are prone to collapse during cooling phase, threatening the safety of fire brigades and inhabitants. Some prime examples of post-fire collapses are the collapse of an underground car-park in Switzerland in 2004 which killed 7 firefighters [1], the collapse of a 12-story building in Egypt in 2004 which caused damage to its residents [2], and the failure of a building in Philadelphia in 2002 which killed a firefighter [3]. Studies and investigations into these incidents have shown that the thermal inertia of concrete causes the delay in the time the material reaches its maximum temperature, eventually resulting in the collapse of RC structures during cooling phase [4-6]. Moreover, concrete may continue to lose its strength after heating and cooling, reducing the post-fire capacity of the structure [7, 8]. It is therefore essential to understand the behavior of RC structures in fire, especially during cooling phase. In this study, the focus was on RC columns, a crucial member of the structural system.

National building codes and design standards provide guidelines for design of RC members subjected to fire, but they do not take into account the effects of cooling phase. In the US, ACI 216.1 [9] offers prescriptive-based design for a certain fire resistance rating of RC columns, which determines the concrete cover thickness and the minimum column dimensions. In Europe, Eurocode 2, Part 1-2 [10] provides simplified and advanced methods of calculating fire resistance of RC columns. In the simplified approach, tables and empirical equations are provided, based on minimum column dimensions, concrete cover thickness, load level and reinforcement ratio. Meanwhile, in the advanced approach, detailed guidelines for fire resistance analysis are presented but cooling phase is not considered. As defined in these documents, fire resistance of a structure is the longest duration of the standard fire that the structure can sustain while remaining its function; fire resistance does not take into account cooling phase.

Several experimental and numerical studies have investigated the performance of RC columns in fire including cooling phase [4, 6, 7, 11]. When cooling phase is considered, the term "burnout resistance" or DHP - defined as the shortest Duration of Heating Phase leading to eventual failure (meaning that for any shorter fire, the structure survives until full burnout) - provides more important information than fire resistance. An experimental study was carried out by Gernay and colleagues [6] to investigate the burnout resistance of RC columns exposed to fires. In that research, RC columns were exposed to standardized ISO 834 heating followed by linear cooling. 4 different durations of heating phase were tried on 4 identical RC columns in order to estimate the DHP of the column. The experiments indicated that for a column with 83-min. fire resistance, the burnout resistance or DHP was substantially lower (between 55 and 72 min.). The study also conducted numerical models whose results matched well with experimental tests. Both experimental and numerical data suggested a linear relationship with a 0.72 factor between the burnout resistance and fire resistance.

Burnout resistance of RC columns was also studied in the numerical study done by Gernay [11], which adopted SAFIR software for finite element models which were verified against the data of 74 standard fire tests on columns. The models considered the irreversibility of material properties and explicit modeling of transient creep. It was found that the burnout resistance of columns is shorter than the fire resistance and increases almost linearly with the fire resistance. A simple equation was proposed to estimate the burnout resistance based on the fire resistance of columns.

2. FIRE DESIGN OF REINFORCED CONCRETE COLUMNS ACCORDING TO EUROCODES

2.1. Development of compartment fire

Eurocode 2, Part 1-2 [10] specifies nominal temperature-time curves (i.e., standard fire curve, hydrocarbon fire curve and external fire curve) and natural fire curves with cooling phase. In natural fire models, there are simplified fire models (which are based on specific physical parameters with a limited range of applications) and advanced fire models (which takes into account gas properties, mass exchange and energy exchange). Parametric temperature-time curves in Annex A of Eurocode 1 [12] is a simplified natural fire model, an example of which is shown in Figure 1.

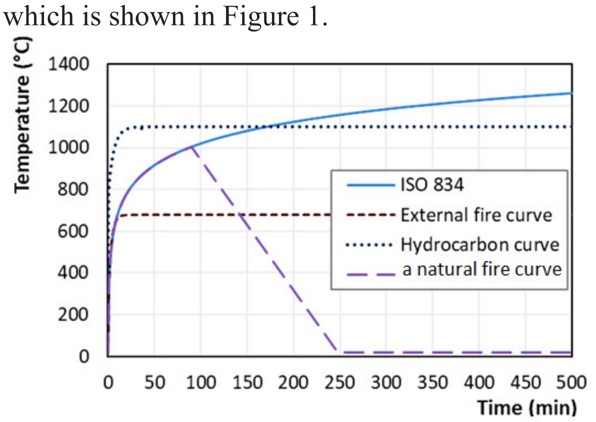

Figure 1. Natural fire curve and nominal fire curves

2.2. Mechanical properties of concrete and reinforcing steel at elevated temperatures

The stress-strain relationships at elevated temperatures of concrete and reinforcing steel used in the study follow specifications in Eurocode 2, Part 1-2 [10], as shown in Figure 2 and Figure 3.

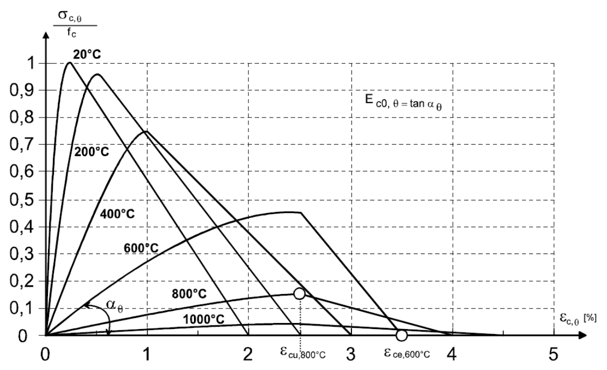

Figure 2. Stress-strain relationships for siliceous concrete with a linear descending branch, specified in Eurocode 2, Part 1-2 [10]

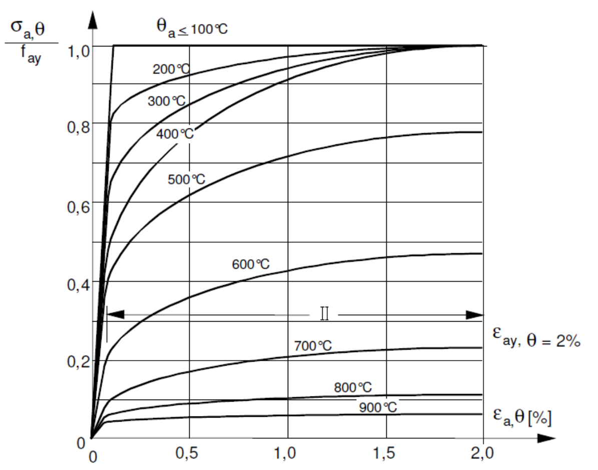

Figure 3. Stress-strain relationships for reinforcing steel up to 2% strain, specified in Eurocode 2, Part 1-2 [10]

2.3. Verifying the fire resistance of RC columns

In Eurocode 2, Part 1-2 [10], there is no method for calculating the fire resistance of RC columns but it shall be verified for the relevant duration of fire exposure t:

$$E_{d,fi} \leq R_{d,t,fi} \qquad (1)$$

where

$E_{d,fi}$ is the design effect of actions for the fire situation, determined in accordance with Eurocode 1 [12], including effects of thermal expansions and deformations;

$R_{d,t,fi}$ is the corresponding design resistance in the fire situation.

The following design methods are permitted in order to satisfy (1):

- detailing according to recognized design solutions (tabulated data or testing);

- simplified calculation methods for specific types of members;

- advanced calculation methods for simulating the behaviour.

Recognized design solutions, or tabulated data, are purely based on empirical data, combined with experience and theoretical evaluation of test results. It bypasses the procedure of selecting design fires and determining the applied loads and the load capacity of the structure. They are only based on the standard fire, a type of nominal fire curve, and only member analysis is possible. For columns, walls, beams and slabs, the Eurocode provides several tables which specify minimal dimensions of the cross-section and the axis distance (distance from the nearest concrete exposed surface to the centroid axis of reinforcing bars), for different fire loads. This makes it easy to use, but conservative and only useful for simple and common cases. Table 1a shows an example of tabulated data.

Table 1a: Minimum column dimension and axis distances for columns with rectangular or circular section (Table 5.2a in Eurocode 2, Part 1-2 [10])

Standard fire resistance	Minimum dimensions (mm) Column width b_{min}/axis distance a of the main bars			
	Column exposed on more than one side			Exposed on one side
	μ_{fi} = 0.2	μ_{fi} = 0.5	μ_{fi} = 0.7	μ_{fi} = 0.7
1	2	3	4	5
R 30	200/25	200/25	200/32 300/27	155/25
R 60	200/25	200/36 300/31	250/46 350/40	155/25
R 90	200/31 300/25	300/45 400/38	350/53 450/40**	155/25
R 120	250/40 350/35	350/45** 450/40**	350/57** 450/51**	175/35
R 180	350/45**	350/63**	450/70**	230/55
R 240	350/61**	450/75**	-	295/70
** Minimum 8 bars For prestressed columns the increase of axis distance according to 4.2.2. (4) should be noted.				

It is noted that tabulated data are given for braced structures and standard fire only.

Simplified calculation methods use the same procedure as with a normal temperature design but take into account the strength loss of both concrete and steel due to high temperatures. This is done by reducing the cross-section which in turn is done by omitting any concrete over a certain limit temperature and possibly reducing the strength of the remaining concrete. Although this method takes more work than the first one, it is still fairly simple and most calculations can be done by hand. The applications are however still limited. The biggest challenge is the determination of the temperature profile in the concrete. Simplified calculation methods only apply to structural members and in some cases to parts of structures. They mostly make use of the standard fire curve.

Advanced calculation methods allow a complete thermal and mechanical analysis of the structure. The continuous alterations of the thermal and mechanical characteristics of the materials and their influence on each other and the complete structure must be taken into account. Advanced calculation methods account for the boundary conditions and the non-homogeneous distribution of the temperature inside elements. They provide a very realistic analysis but the use of sophisticated computer programs, combined with a thorough background-knowledge, is necessary. Advanced calculation methods can be used with all types of analysis (members, part of the structure or entire structure) and with all types of design fires.

3. COMPUTATIONAL MODELS OF REINFORCED CONCRETE STRUCTURES EXPOSED TO FIRE INCLUDING COOLING PHASE

Advanced calculation method is adopted in this study. With the help of the finite element (FE) software SAFIR, fire resistances of RC columns are calculated with both standard fire curve and natural fire curve.

SAFIR [14] is a non-linear FE software developed at the University of Liege, Belgium for the simulation of thermal and structural behavior at room temperature and in fire. A substantial number of studies on RC structures exposed to fire using SAFIR for their numerical models which were verified with experimental data prove the validity of SAFIR in simulating RC structures exposed to fire [4-6,15,16].

3.1. Fire model

In Eurocode 2, Part 1-2 [10], it is specified that "The load-bearing function should be maintained during the complete endurance of the fire including the decay phase, or a specified period of time" and "Where application rules given in this Part 1-2 are valid only for the standard temperature-time curve". This part of the research used natural fire models (including the decay/cooling phase).

In this research, the adopted model for natural fires is the parametric fire model from Eurocode 1, Part 1-2 [12]. The value of the factor Γ is 1.0 in the model, which makes the heating phase of the time-temperature curve of this natural fire model the same as the standard ISO curve. Figure 4 shows the time-temperature curves for different values of HeatT. Curve HeatT_60 expresses the natural fire with 60 minutes of heating phase.

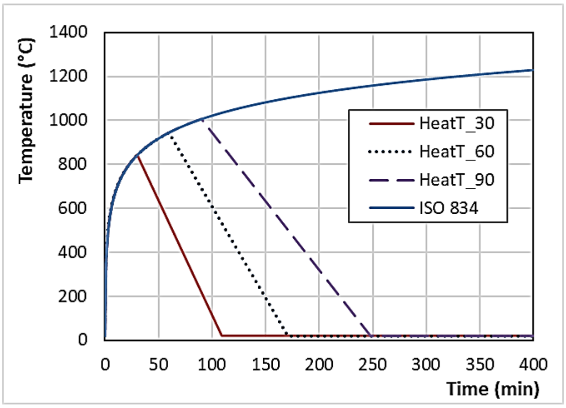

Figure 4. Eurocode parametric fire model for natural fire

3.2. Heat transfer analysis

It is assumed that conduction is the main heat transfer mechanism within the structural element. Convection and radiation act essentially as heat transfer between the surrounding environment and the element.

It is assumed that a uniform temperature is distributed through the height of RC column. Therefore, a two-dimensional model is adequate for the thermal analysis for the column. Triangular and quadrilateral elements are used for simulating the cross-section. Figure 5 shows one of the RC column sections investigated in the study, all four sides of the column are exposed to fire.

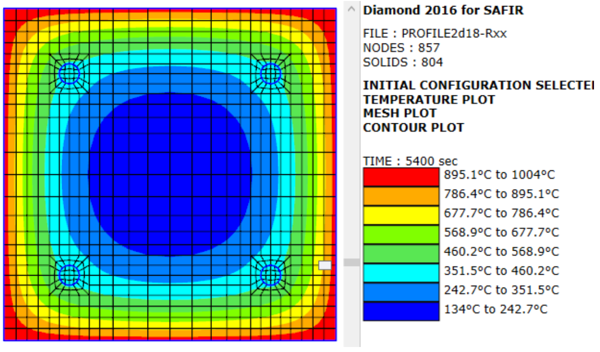

Figure 5. Temperature distribution in a column cross-section

For each element, the material can be defined separately. Any material can be analyzed provided its physical properties at elevated temperatures are known; the changes of material properties with temperature are also considered.

The thermal properties of concrete and reinforcing steel follow the recommendations of Eurocode 2 [10]. Siliceous concrete was used in this research.

3.3. Structural analysis

The program SAFIR is deployed in the study to investigate the behaviour of two and three-dimensional structures. It is because SAFIR can facilitate a wide range of elements with various idealizations, calculation procedures and materials. The elements include the 2-D SOLID elements, 3-D SOLID elements, BEAM elements, SHELL elements and TRUSS elements. The thermal analysis is first conducted to determine time-temperature evolution in each fiber, which is then input into the structural analysis. The structural analysis takes into account geometrical and material non-linearity, including large deflections.

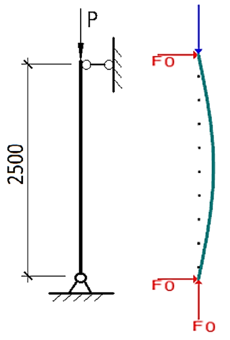

Figure 6. Column model

In the structural model, the columns are simulated by 2-D beam elements. The integration of the longitudinal stresses and stiffness into the section is based on the fibre model; the section is supposed to be composed of a number of parallel fibres. The discretization adopted in the structural analysis is the same as the one used in the thermal analysis. Each finite element of the thermal analysis, with its known material type and temperature, is considered as a fibre. The effect of thermal expansion of steel and concrete is also considered. More information is given in [14].

In this research, all the mechanical properties of concrete and reinforcing steel follow the recommendations of the Eurocodes. The mechanical properties of steel are considered as reversible, which means that stiffness and strength are recovered to full initial values during cooling. The mechanical behaviour of reinforcing steel follows the model of Eurocode 2 [10]. In the case of varying temperatures, the plastic strain is a variable describing the complete stress-strain history as illustrated in Figure 7. The point P of the stress-strain curve is recalculated after each increment of temperature increase or decrease.

For concrete, a residual thermal expansion or shrinkage is considered when the concrete is back to ambient temperature, the value of which is taken as a function of the maximum temperature according to experimental tests done by Franssen (1993) [17]. As prescribed in Eurocode 4 [13], an additional loss of 10% of the concrete compressive strength is considered during cooling, with respect to the value at maximum reached temperature. This additional reduction during cooling is supported by many experimental studies such as Li and Franssen (2011) [18]. The Explicit Transient Creep Eurocode model (explicit model) is adopted to take into account the transient creep strain irreversibility during cooling (Gernay and Franssen 2012 [19]).

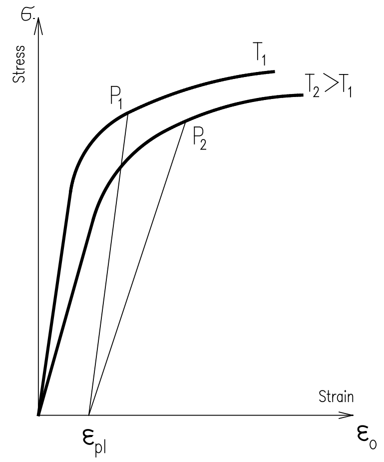

Figure 7. Stress-strain curves at varying temperatures

3.4. Fire resistance (R) and burnout resistance (DHP)

For a standardized fire, the temperature is continuously increased in the compartment, so that the temperatures in the elements also continuously increase and, assuming that all materials' properties degrade as temperature increases, the load-bearing capacity continuously decreases. Failure occurs at the time when the demand meets the capacity; this time is defined as the Fire Resistance (R) of the structural component.

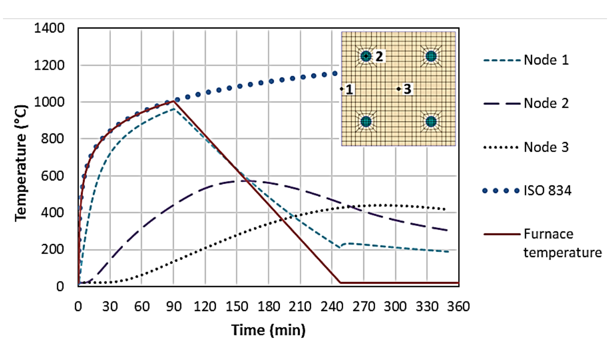

Figure 8. Temperature evolution in RC column in the natural fire with 90 minutes of heating

For a natural fire, the temperature in the furnace is first increased to a maximum and then decreased back to room temperature. In that case, the load-bearing capacity of the component will first decrease until reaching a minimum and then it may remain constant or recover, partially or completely, after the furnace temperature has come back to room temperature. Importantly, the time of the maximum furnace temperature and the time of the minimum load-bearing capacity are generally not simultaneous, the latter arising later than the former. To illustrate this, Figure 8 shows the temperature evolution in section Profile 1 (see Table 1). The furnace temperature is a natural fire with 90 minutes of heating phase then cooling to room temperature after 245 minutes. The temperatures at the surface of the column (node 1), the reinforcing steel (node 2) and the concrete center (node 3) are shown as well. It is found that the furnace and surface of the column reach maximum temperature at time 90 minutes while the reinforcing steel reaches maximum temperature after 150 minutes. The temperature of concrete in central zone keeps on increasing even after 240 minutes. The column may not fail after 90 minutes of exposure to heating (if the fire resistance R is greater than 90 minutes) but it may fail after more than 120 minutes, during the cooling phase of the fire. Hence, the Fire Resistance R does not provide enough information to characterize the performance of structures under natural fires. A new indicator is thus needed to complement it. This indicator must be related to a certain level of severity of the natural fire. The new metrics, so-called Duration of Heating Phase (DHP) or burnout resistance, has been introduced in [11, 20].

By definition, the DHP represents the minimum exposure time to standard ISO fire (followed by cooling phase in accordance with the Eurocode parametric fire model as in [12] that will eventually result in the failure of the structural component (either in the heating phase, in the cooling phase or after termination of the fire). In performance-based design, the Fire Resistance indicator is interpreted as information about the time of resistance during the heating phase of a fire, although it is obviously not a direct measurement of this time since the real fire conditions will differ from the prescriptive fire conditions. Meanwhile, the DHP is interpreted as information about the occurrence of delayed failure as a function of the time when the fire started to decrease, whether by self-extinction or due to the action of firefighters. In design or test standards, only the Fire Resistance indicator is concerned, DHP is not yet introduced. DHP is always smaller than Fire Resistance R. It is important to notice that the DHP does not give any indication about the time of collapse (time of failure). Generally, collapse can occur several minutes or hours after the time corresponding to the DHP, and it may even occur after the end of the fire when the temperature in the compartment is back to ambient.

In the research done by Gernay [11], a simple relation was given between DHP and R:

$$\text{DHP} = 0.72\text{R} - 3.0 \text{ (in min.)} \qquad (2)$$

The aim of this study is to investigate the parameters affecting the DHP - R relationship.

4. PARAMETRIC STUDY OF THE FIRE RESISTANCE AND BURNOUT RESISTANCE OF RC COLUMNS

Approximately 100 RC columns with 28 different cross-sections were studied. Only square sections were concerned in this study for the first step of the research. The cross-sections are named from Profile 1 to Profile 28. Some profiles are listed in Table 1, others only differ from concrete cover thickness or the number of rebars. Profile 1 has the same dimension as a column in research of Kodur et al. [21]. The length of column is 2.5 meter, the initial eccentricity of the load is b/30, concrete compresive strength is 30 MPa, and steel strength is 500 MPa. Parameters under the investigation are the cross-section dimensions, the concrete cover thickness, the reinforcement ratio (i.e, number and size of rebars), and the load ratio. For a given structural column, numerical simulations are run under different natural fire exposures to find the fire resistance (R) and burnout resistance (DHP) of columns. DHP is also calculated by the simple equation (2) as suggested in research [11] and denoted as $\text{DHP}_{\text{simple}}$.

Table 1. Cross-section of studied columns

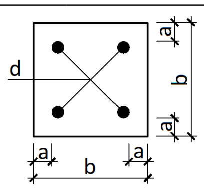		b (dimension of square section) (mm)	a (concrete cover thickness) (mm)	Number and diameter of rebar
Profile 1	C203-4d19a30	203	30	4d19
Profile 4	C300-4d18a50	300	50	4d18
Profile 5	C300-4d22a50	300	50	4d22
Profile 6	C300-4d28a50	300	50	4d28
Profile 7	C300-4d20a40	300	40	4d20
Profile 8	C300-4d20a20	300	20	4d20
Profile 9	C300-4d20a30	300	30	4d20
Profile 10	C300-4d20a50	300	50	4d20
Profile 11	C300-8d20a50	300	50	8d20
Profile 12	C300-12d20a50	300	50	12d20
Profile 20	C300-8d20a40	300	40	8d20
Profile 26	C400-16d20a30	400	30	16d20
Profile 28	C400-24d20a30	400	30	24d20

Table 2. Results of columns with various load ratios

Element	Load ratio	R (min)	DHP (min)	DHPsimple (min)
Profile1 C203-4d19a30	0.2	62	49	42
	0.3	48	36	32
	0.4	38	25	24
	0.5	**30**	**18**	**19**
	0.6	23	12	14
	0.7	18	7	10
Profile7 C300-4d20a40	0.2	145	108	99
	0.3	115	84	78
	0.4	91	64	61
	0.5	**72**	**47**	**47**
	0.6	54	33	35
	0.7	38	19	24
Profile12 C300-12d20a50	0.2	180	130	127
	0.3	146	97	102
	0.4	119	76	83
	0.5	**94**	**58**	**65**
	0.6	70	39	47
	0.7	49	22	32
Profile20 C300-8d20a40	0.2	161	115	110
	0.3	127	91	86
	0.4	102	87	68
	0.5	**80**	**53**	**53**
	0.6	61	38	40
	0.7	44	22	28

The following sections present the influence of the studied parameters: load ratio, concrete cover thickness and reinforcement ratio. Some not all results are shown here as others are similar.

4.1. Effect of load ratio

Load ratio is defined as the ratio between the load applied on a structural element in fires and the load-bearing capacity of the element at room temperature. In a fire event, the applied loads are much lower than the maximum design loads at room temperature because load combination factors are smaller under fire conditions compared to room temperature. The parameter Load ratio is of more importance than the load value applied on the column, as the Load ratio represents the extent to which the column is stressed. Load ratio is recommended less than 0.65 in Eurocode 1 Part 1-2 [12].

Results of the study show that Load ratio significantly affects the fire resistance R and the DHP. Table 2 shows the results of columns with load ratio ranging from 0.2 to 0.7. With a load ratio of 0.5 the simple equation (2) works well.

4.2. Effect of concrete cover thickness

Table 3: R and DHP with various concrete cover thickness (reinforcement ratio = 0.014, load ratio = 0.5)

Element	a (mm)	R (min)	DHP (min)	DHPsimple (min)
Profile 8	20	59	43	39
Profile 9	30	66	46	45
Profile 7	40	72	47	49
Profile 10	50	76	47	52

Results show that the concrete cover thickness has minor effect on the DHP, provided constant load ratio value. The simple equation (2) seems to work well with various concrete cover thickness.

4.3. Effect of reinforcement ratio

Table 4: R and DHP with various reinforcement ratio (concrete cover thickness = 50 mm and load ratio = 0.5)

Element	As/Ac	R (min)	DHP (min)	DHPsimple (min)
Profile 4	0.011	74	46	50
Profile 10	0.014	76	47	52
Profile 5	0.017	78	49	53
Profile 6	0.027	86	53	59
Profile 11	0.028	87	54	60
Profile 12	0.042	94	58	65

Results show that the reinforcement ratio has minor effect on the DHP and R relationship provided constant load ratio value. The simple equation (2) works well with various reinforcement ratio.

5. CONCLUSION

Some key findings of this research are as follows:

The shortest duration of heating phase causing the failure of structures in the cooling phase of fire (DHP) needs more attention.

For RC columns, DHP is always less than the fire resistance R given in regulations and codes. The correlation between DHP and R is considerably affected by the load ratio of the column. With the limited number of studied columns, the simple correlation given in [11] works well with the load ratio 0.5 only. More numerical studies should be conducted to determine the correlation between load ratio and DHP-R relationship.

The value of reinforcement ratio and concrete cover thickness have minor effect on DHP-R relationship.

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