EFFECT OF CONCRETE COVER ON AXIAL LOAD RESISTANCE OF REINFORCED CONCRETE COLUMNS IN FIRE

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***Summary**: The prescriptive rules in national codes specify that the fire resistance of reinforced concrete (RC) structures is proportionally dependent on the concrete cover to main reinforcing steel bars. However, performance-based design approaches developed from rational analyses of thermal and mechanical responses of materials and structures require more specific information of the relationship between concrete cover and the axial load resistance of RC structures, which may result in more economical design compared to those obtained from prescriptive rule. This paper introduces an analytical model that is capable of simulating the effects of concrete cover on the temperature-dependent mechanical properties of concrete and reinforcing steel used for the cross-sectional analysis for RC columns when subjected to fire. Parametric studies are conducted on prototype columns, based on which the axial load resistance of the column can be quantitatively determined at a certain time of a standard fire. The proposed analytical model is validated by a number of fire tests on RC columns and can be efficiently used for simplified assessment procedure of load resistance of RC columns in fire.*

***Keywords**: Column; reinforced concrete; concrete cover; axial load; fire.*

Received: August 16th, 2016, revised: August 29th, 2016, accepted: October 13th, 2016

1. Introduction

In reinforced concrete (RC) structures, the concrete cover to main reinforcing steel bars is not only for the efficient bonding between steel reinforcement and concrete but also for the protection of the reinforcement from the impact of natural corrosion and elevated temperatures in fire incident. The prescriptive rules in national codes specify that the fire resistance of RC structures is proportionally dependent on the concrete cover. For example, Vietnam building code of fire safety of buildings [1] requires the minimum average concrete covers to main reinforcing bars of 15, 25 and 35mm for the respective fire rates of R30 (fire resistance of 30min), R60 and R90 of RC beams. The Eurocode [2] specifies that when the reduction factor for the design load level in the fire situation is μ_{fi}=0.5, the axis distance from the RC column surface to the centroid of steel bars is to be 25, 36 and 45mm for R30, R60 and R90, respectively. Recently, performance-based approaches have been developed in some countries in order to introduce more rational solutions in fire-resistant structural design codes. These approaches are based on the selection of various fire development models, as well as on the physical based thermal actions to produce simple or advanced calculation models to gain more economical design compared to those obtained from prescriptive rules. The idea is applied to RC columns in this paper. An analytical model is introduced to simulate the effects of concrete cover on the temperature-dependent mechanical properties of concrete and reinforcing steel, which will be used for cross-sectional analysis of RC columns when subjected to fire. The axial load resistance of the column can be quantitatively determined at a certain time of a standard fire based on parametric studies conducted on prototype columns. The proposed analytical model is validated by a number of fire tests on RC columns and can be efficiently used for simplified assessment procedure for load resistance of RC columns in fire condition.

2. Effects of elevated temperatures

a) Natural fire development and standard fire curves

Essentially, fires occur when combustible materials are ignited by heat sources. Natural fire can be represented by fires in a compartment, so-called compartment fires. The temperature-time relationship of a

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compartment fire depends mainly on the compartment type and size, available combustible material and air supply. If there is no active scheme control in the compartment, there are three sequenced stages of the development of natural compartment fire, namely, growth stage (also known as pre-flashover stage), fully-developed stage and decay stage. The two latter stages form the post-flashover stage (Fig. 1(a)). Unlike compartment fires, standard fire exposures are controlled by measured temperature regimes in the furnace following prescribed temperature-time relationships, so-called standard fire curves. Standard fire curves are generally used in fire resistance analysis. European countries commonly apply ISO 834 [3], whereas the American standard ASTM E119 [4] is widely used in Northern American countries. Eq. (1) and Eq. (2) represent the gas temperature-time relationships of ISO 834 and ASTM E119, respectively. These fire curves are graphically shown in Fig. 1(b).

$$T_g = 20 + 345\log_{10}(480t + 1) \qquad \text{(ISO 834)} \qquad (1)$$

$$T_g = 20 + 750\left(1 - e^{-3.79553\sqrt{t}}\right) + 170.41\sqrt{t} \qquad \text{(ASTM E119)} \qquad (2)$$

where T_g is gas temperature in °C and t is exposure time in hour.

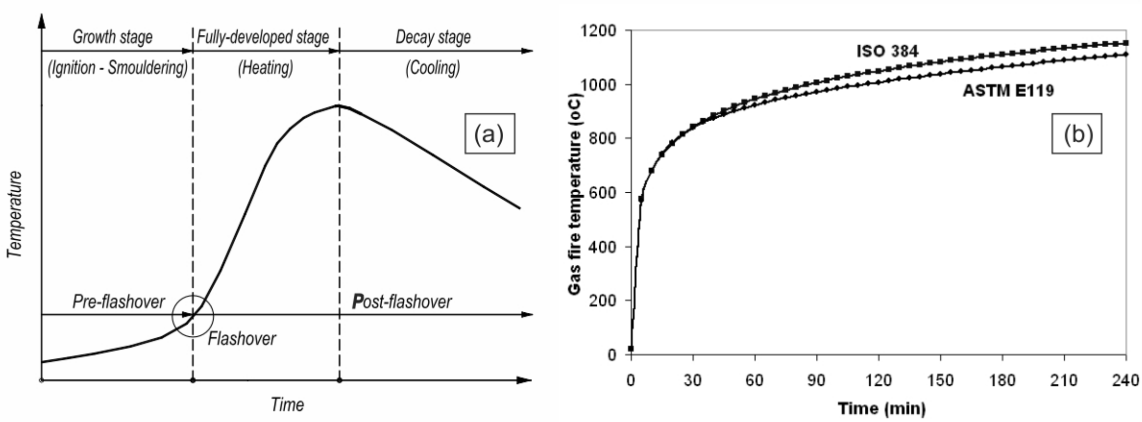


Figure 1. *(a) Typical natural fire curve (b) Standard fire curves*

It is shown that different from natural fire curves, standard fire curves develop without any decay stage. In standard fire curves, factors considered in natural compartment fires such as fuel size, room geometry and ventilation openings, are ignored. Furthermore, only one gas temperature-time relationship is considered in the standard fire curve whereas natural fire curve allows for different heating rates. Thus, standard fire curves are less flexible with little potential in saving the construction cost. Nevertheless, there is a need to provide a common basis for comparison so that the fire resistance of structural elements can be compared on the same temperature-time relationship. As a result, standard fire curves are generally used for analysis purpose.

b) Thermal response of RC columns in fire

In the fire tests using standard fire curves, so-called standard fire tests, while beams and slabs are usually heated from beneath, column specimens are generally heated on all four sides. In real fire situations, columns with rectangular cross-section may also be subjected to 1-, 2- or 3-face fire attack depending on its position in a building layout, as well as on the partition arrangement. Temperatures of different points in RC columns are determined by heat balance analysis. Those temperatures are lower than the gas temperature on the columns surface since it takes time for the heat transfer process to take place. This process in RC columns can be determined based on material thermal properties and heat transfer methods such as radiation, convection and conduction. Basically, thermal analysis can be conducted based on the conservation of thermal energy, which dictates that the system heat storage is equal to the heat source thermal energy. Current methods to calculate the temperature distribution in RC columns are mainly based on sectional analysis, assuming that temperature is uniformly distributed along the column length.

Compared to structural steel, concrete has much better fire-resistant properties, such as lower thermal conductivity and greater specific heat capacity. Hence, the reinforcing steel bars embedded in concrete can be protected from high-rate temperature increment at the surface owing to the concrete cover. Figs. 2(a,b) illustrate the Eurocode temperature distribution in a 300mm square cross-section RC column subjected to standard fire curve ISO 834 after 30min and 90min, respectively [2,3,5].

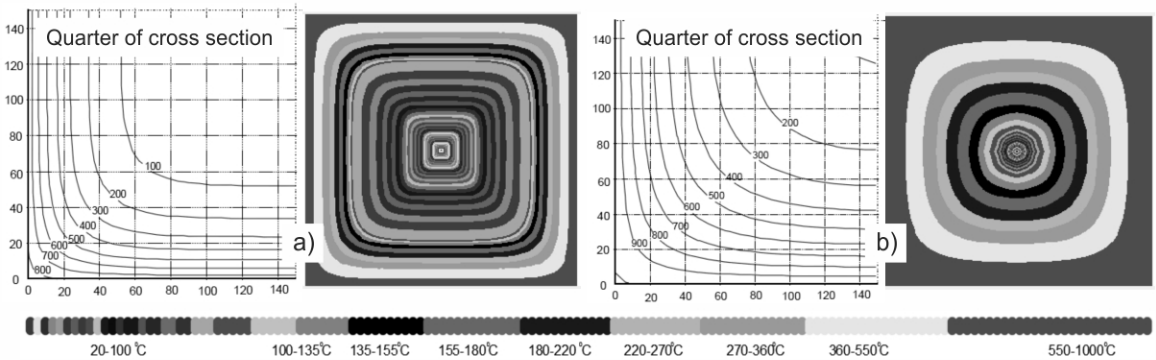


Figure 2. *Temperature distribution in RC columns (a) After 30min (b) After 90min*

It can also be seen that temperature of steel bars at certain positions in the column cross section increases with time in fire situation. However, for the inner positions of reinforcing bars, which is in the case of the thicker concrete cover, the temperature increment is slower due to the heat transfer process.

c) Strength reduction of materials at elevated temperatures

Both concrete and reinforcing steel experience continuous deterioration in strength as temperature increases. Figs. 3(a,b) respectively illustrate the temperature-dependent strength reduction factors for concrete and reinforcing steel specified in the Eurocode [2].

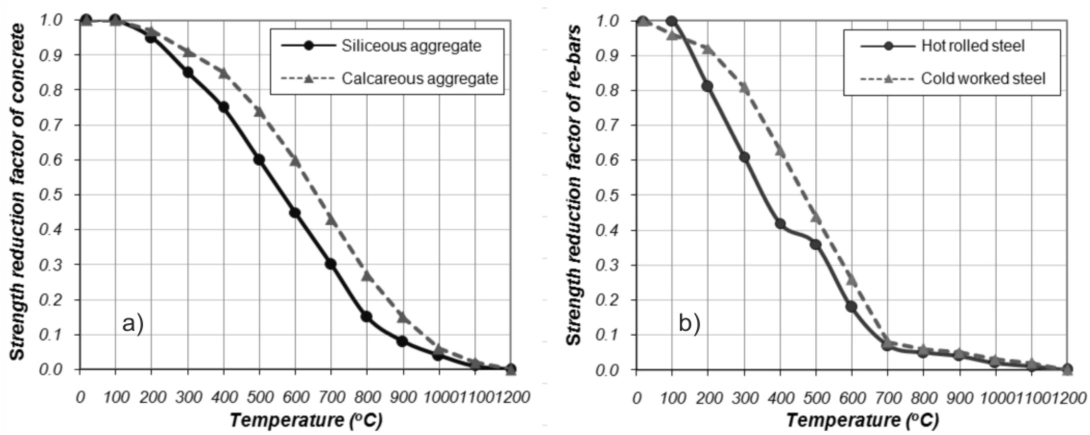


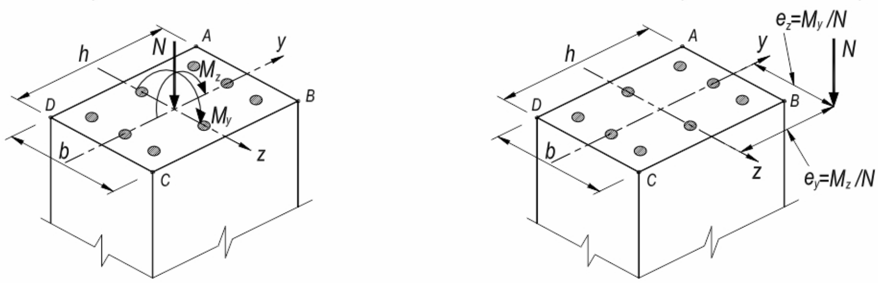
Figure 3. *EC2 strength reduction factors (a) Concrete (b) Reinforcing steel [2]*

In Fig. 3(a), the continuous and hidden lines are the reduction curves for compressive strength of concrete using siliceous and calcareous aggregates, respectively. The continuous and hidden lines in Fig. 3(b) are the reduction curves for tensile strength of hot rolled and cold worked steel, respectively.

3. Effect of concrete cover on axial load resistance of RC columns in fire

a) Analytical model for cross-sectional analysis of RC columns in fire

Fig. 4(a) shows a column of rectangular cross-section (b×h) subjected to an axial load N and bending moments M_z and M_y corresponding to the respective biaxial eccentricities $e_y=M_z/N$ and $e_z=M_y/N$ as depicted in Fig. 4(b).



(a) Axial load and bending moments *(b) Equivalent eccentricities*

Figure 4. *RC columns subjected to eccentric loads*

In order to account for non-uniform temperature distribution resulted from low thermal conductivity of concrete, the cross-section is discretized for analysis as shown in Fig. 5.

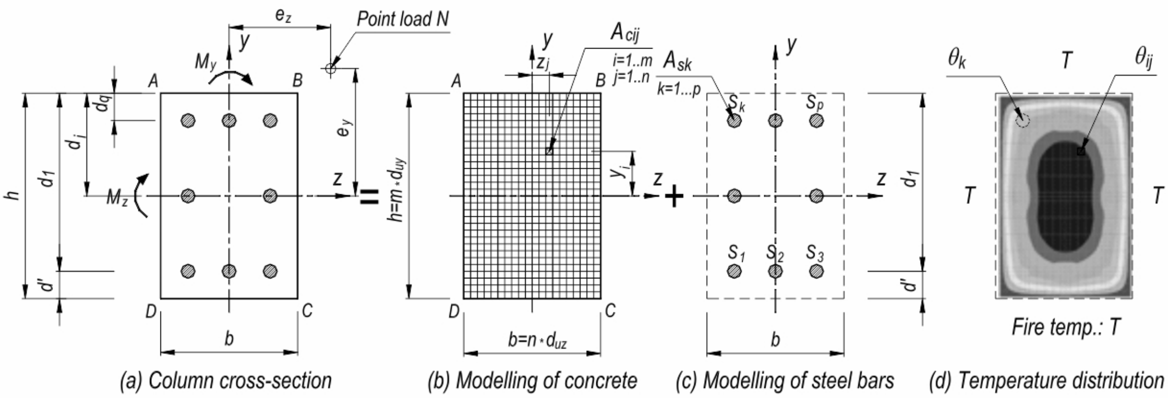


Figure 5. *Analytical model for cross-section*

The proposed analytical model includes a two-dimensional array of (m×n) rectangular concrete sub-elements having a unique area of $A_{c\text{-}ij}=d_{uz}\times d_{uy}$ (i=1 to m, j=1 to n) and a number of round-shaped sub-elements representing reinforcing bars, namely, S_1 to S_p, with the corresponding areas $A_{s\text{-}k}$ (k=1 to p). The temperature distribution in this model can be interpreted from the Eurocode data presented in Section 3 or finite element method SAFIR software [6]. At every time interval of fire exposure, the corresponding interpreted temperature profile, such as θ_{ij} of concerete sub-element (ij) and θ_k of re-bar (k), will be used as fundamental data to determine the strenght reduction within each sub-element for further analyses [7]. In the proposed rational analysis, the key target is to address the failure time of a column based on its thermal and structural responses to realistic conditions of loading, restraint and fire exposure, which exist in framed concrete structures. The analysis principles used in this approach can be demonstrated by a restrained RC column subjected to uniaxial bending (Fig. 6). At ambient condition, the column is initially designed to sustain the second-order ultimate loads (N_1, M_2) represented by point P_2 located on the ambient M-N interaction diagram of the column cross-section (Fig. 6(a)).When fire occurs, the column is only subjected to a lower axial load N_0 determined by multiplying the ultimate load N_1 by a reduction factor μ_{fi}. This initial axial load N_0 and the corresponding moment M_0 are represented by point I. During a fire, there are two critical movements acting in opposite directions in Fig. 6(a). Since both concrete and reinforcing steel experience deteriorations in strength when temperature rises, the column M-N interaction diagram at 20°C gradually regresses into similar but smaller shape, resulting in the first inward movement. On the other hand, since the column is constrained from free thermal elongation, additional restraint compression forces are generated. This increment in total axial force, associated with the stiffness degradation of materials in fire, magnifies the bending moment acting on the column, resulting in an outward movement of the applied load-moment curve.

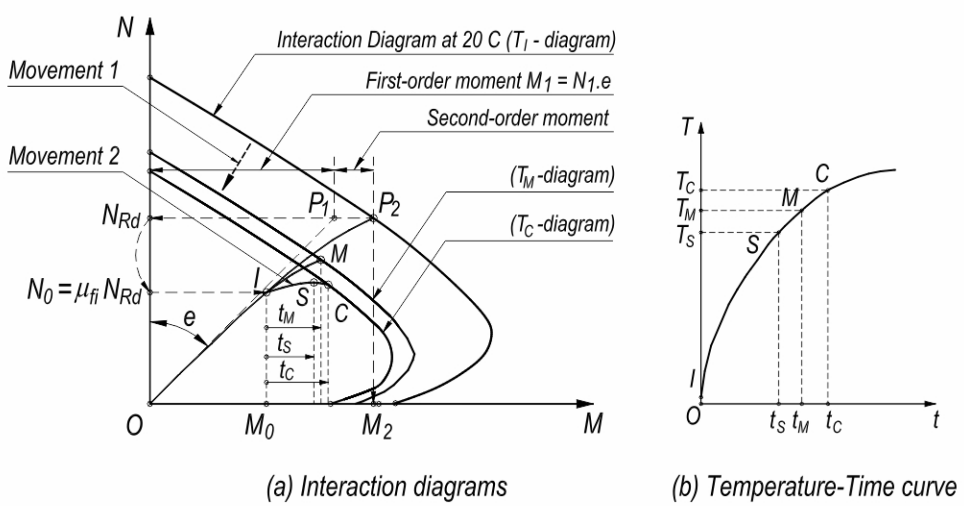


Figure 6. *Principles of rational analysis*

If the column is stocky or the eccentricity is small, material failure occurs after a period of time t_M at the corresponding temperature T_M (Fig. 6(b)), when the load-moment curve and the T_M-interaction diagram coincide at point M (Fig. 6(a)). As a result, the term t_M can be referred to as material failure time. If the column is either of high slenderness or subjected to large eccentricity, due to the deterioration of material stiffness at elevated temperatures, the column may lose stability at point S where the derivation $\partial N/\partial M$ approaches zero before becoming negative and a run-away trend occurs in the column lateral deflection. Thus, the column is deemed to have failed in an instability manner before the temperature-dependent effects of action exceed the column resistances. Stability failure time is then represented by t_S. A combined mode of failure is identified when after the column buckles, the load-moment curve and the regressing M-N interaction diagram coincide at point C. The term t_C then represents the combined failure time of the column. In Fig. 6(a), the intersection between the interaction diagram and N-axis can be referred to as the axial load resistance of RC columns.

b) Experimental validation

Fire tests on twelve column specimens were conducted by the author in the Nanyang Technological University (Singapore) [8,9]. All the columns were pin-ended and restrained from thermal elongation. Six columns were under uniaxial bending whereas the remaining columns were biaxially loaded. The test set-up is shown in Fig. 7.

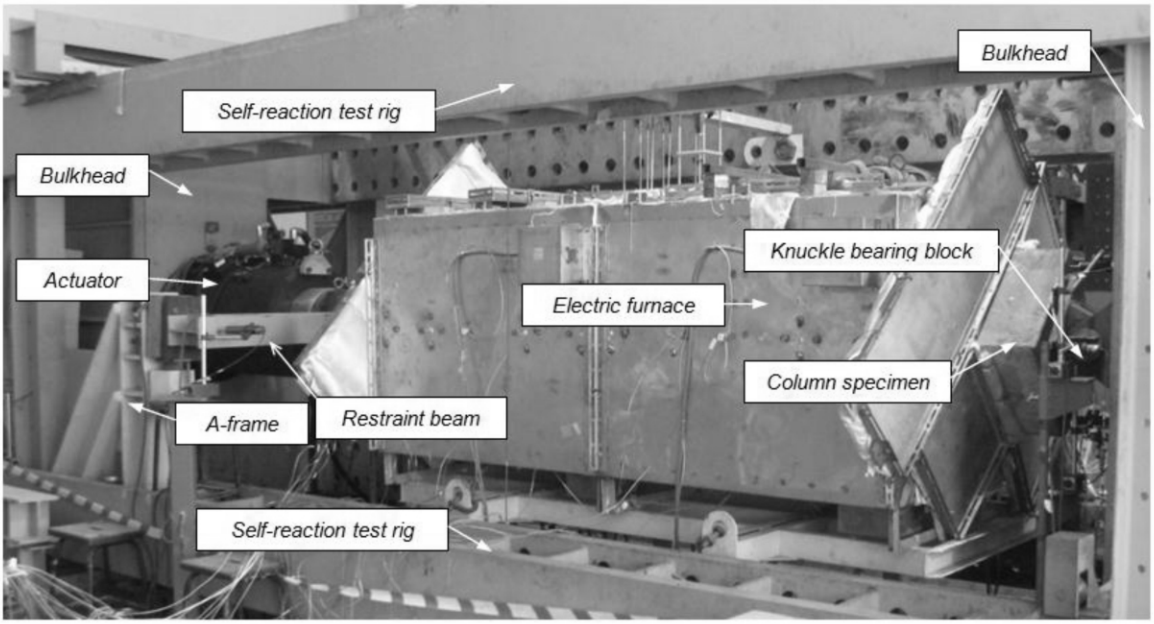


Figure 7. *Set-up of tests for validation*

Fire tests were all carried out under transient heating state conditions. A displacement-controlled loading mode was applied at a rate of 0.5mm/min and then held constant at N_0. With the supports of the restraint beam connected to the reaction A-frames and the load cells at each beam end indicating proper contact, the fire condition was generated by an electric furnace. While N_0 was still kept unchanged, readings of thermocouples, LVDTs and load cells were recorded at 30-second intervals until the hydraulic actuator could no longer maintain the load, indicating that the initial applied load and the thermal-induced compression force could not be sustained by the test specimen and the specimen has failed. At this moment, testing was terminated and failure time was recorded.

Compared the failure times recorded in the tests to the time determined from the proposed analytical model, relatively good agreement are obtained from both method of nominal curvature (MNC) and method of nominal stiffness (MNS), with mean ratios of 1.12 and 1.04 and coefficient of variations of 0.14 and 0.11, respectively (Table 1) [7,10]. Experimental validation of the proposed analytical model was also conducted using 89 fire test results (with concrete cover varying from 25 to 64mm) from other five laboratories with good agreement [7].

Table 1. *Results of experimental validation [7-10]*

No	Ref.	Cross-section $b{\times}h\text{-}n\mathrm{T}\Phi\text{-}c'$ (mm)	l (m)	f_c (MPa)	f_y (MPa)	End	e (mm)	N_{test} (kN)	t_{test} (min)	$T_{predict}$ (min)	
										MNC	MNS
1	C1-1-00	300×300-6T20-30	3.54	55.3	550	p-p	0	1720	310	328	262
2	C1-2-00	300×300-6T20-30	3.54	55.3	550	p-p	0	2100	251	286	244
3	C1-3-25	300×300-6T20-30	3.54	55.3	550	p-p	25	1700	252	272	228
4	C1-4-40	300×300-6T20-30	3.54	55.3	550	p-p	40	1400	221	274	228
5	C1-5-60	300×300-6T20-30	3.54	55.3	550	p-p	60	1100	180	270	228
6	C1-6-80	300×300-6T20-30	3.54	55.3	550	p-p	80	900	214	270	236
7	C2-1-25	300×300-6T20-35	3.54	55.3	550	p-p	25,25	1150	219	246	240
8	C2-2-40	300×300-6T20-35	3.54	55.3	550	p-p	40,40	920	253	246	240
9	C2-3-60	300×300-6T20-35	3.54	55.3	550	p-p	60,60	650	248	246	246
10	C3-1-25	300×300-4T25-35	3.54	29.3	554	p-p	25,25	860	264	270	286
11	C3-2-40	300×300-4T25-35	3.54	29.3	554	p-p	40,40	680	250	262	280
12	C3-3-60	300×300-4T25-35	3.54	29.3	554	p-p	60,60	530	242	258	272

c) Parametric study on the effect of concrete cover

Having obtained good results from experimental validation, the proposal anlytical model is used for parametric study on the effect of concrete cover to the axial load resistance of RC column in fire. The information of the prototype column is shown in Table 2.

Table 2. *Prototype column for parametric study*

No	Ref.	$b{\times}h$ (mm)	Rebars $n\mathrm{T}\Phi$	**c'** (mm)	ρ_l (%)	f_{ck} (MPa)	f_{yk} (MPa)	f_{cd} (MPa)	f_{yd} (MPa)	Fire curve
1	C3-1	300×300	4T25	25	2.18	30	500	25.5	500	ISO 834

In the parametric study, all the properties of the columns are kept unchanged, whereas only the concrete cover c' is varied from 25 to 75mm.

When subjected to a standard fire scenario, the cross-sectional ultimate axial load resistance of RC columns, which is the intersection between the interaction diagram and N-axis in Fig. 6, is shown in Eq. (3).

$$N_{ud}^{\theta} = k_{ud}^{c'}(t)\left(A_c \frac{0.85 f_{ck}}{\gamma_{c,fi}} + A_s \frac{f_{yk}}{\gamma_{s,fi}} \right) \tag{3}$$

In Eq. (3), the cross-sectional ultimate axial resistance N_{ud}^{θ} of an RC column at a certain time t of standard fire can be determined based on the concrete gross cross-section A_c and the gross section A_s of longitudinal rebars, the ambient characteristic compressive and yield strengths of concrete and reinforcement (f_{ck} and f_{yk}, respectively) and a reduction factor $k_{ud}^{c'}(t)$. The coefficient of 0.85 is to account for the long-term effect on compressive cylinder strength and for unfavorable effects resulted from the way a load is applied, as specified in EC2 Pt.1.1 [11]. Besides, the partial safety factors of concrete and reinforcement for the fire situation, which are $\gamma_{c,fi}$ and $\gamma_{s,fi}$, respectively, are set to unity, in accordance with EC2 Pt.1.2 [2].

Fig. 8(a) shows the reduction factor influenced by concrete cover, so-called $k_{ud}^{c'}(t)$. This factor is obtained from the analytical model introduced in Item 3.a of this paper which accounts for the rational relationship between the load resistance and the reduction of material properties at elevated temperatures.

It can be seen that the reduction in ultimate axial resistance is more significant as concrete cover (c') decreases. This is mainly due to the contribution of steel reinforcement, which has a greater reduction in strength at higher elevated temperatures occurred by thinner concrete covers. Based on the curves plotted in Fig. 8(a), a simplified model is proposed to determine the c'-dependent reduction factor $k_{ud}^{c'}(t)$ of ultimate axial resistance of the column as shown in Fig. 8(b) and Eq. (4).

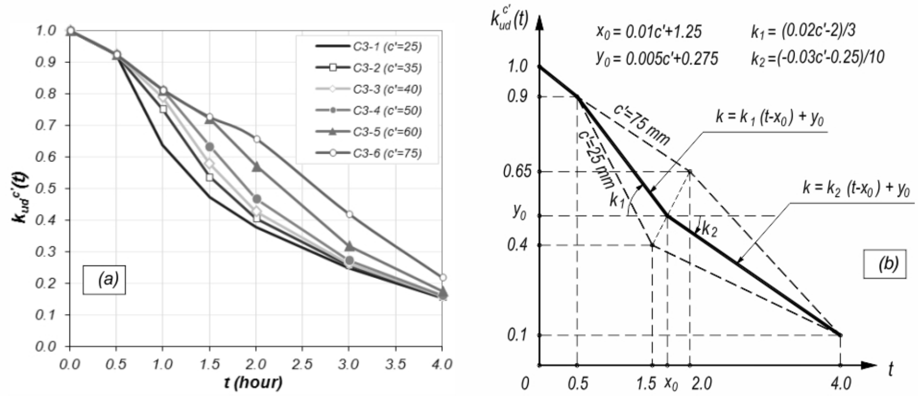


Figure 8. *(a) Results of parametric study on $k_{ud}^{c'}$ (t) (b) Simplified model*

$$k_{ud}^{c'}(t) = 1 - 0.2t \qquad \text{for } t \le 0.5\text{h} \qquad (4a)$$

$$k_{ud}^{c'}(t) = \left(\frac{0.02c' - 2}{3}\right)\left(t - 0.01c' - 1.25\right) + \left(0.005c' + 0.275\right) \qquad \text{for } 0.5\text{h} < t \le (0.01c'+1.25)\text{h} \qquad (4b)$$

$$k_{ud}^{c'}(t) = \left(\frac{-0.03c' - 0.25}{10}\right)\left(t - 0.01c' - 1.25\right) + \left(0.005c' + 0.275\right) \qquad \text{for } (0.01c'+1.25) < t \le 4\text{h} \qquad (4c)$$

where concrete cover c' is in mm and fire exposure time t is in hour.

The ultimate resistance of longitudinal rebars in an RC column subjected to a standard fire condition is shown in Eq. (5).

$$T_{ud}^{\theta} = k_{Tud}^{c'}(t)\left(A_s \frac{f_{yk}}{\gamma_{s,fi}}\right) \qquad (5)$$

The c'-dependent reduction factor $k_{Tud}^{c'}$ *(t)* obtained from the analytical model proposed in Item 3.a is shown in Fig. 9(a).

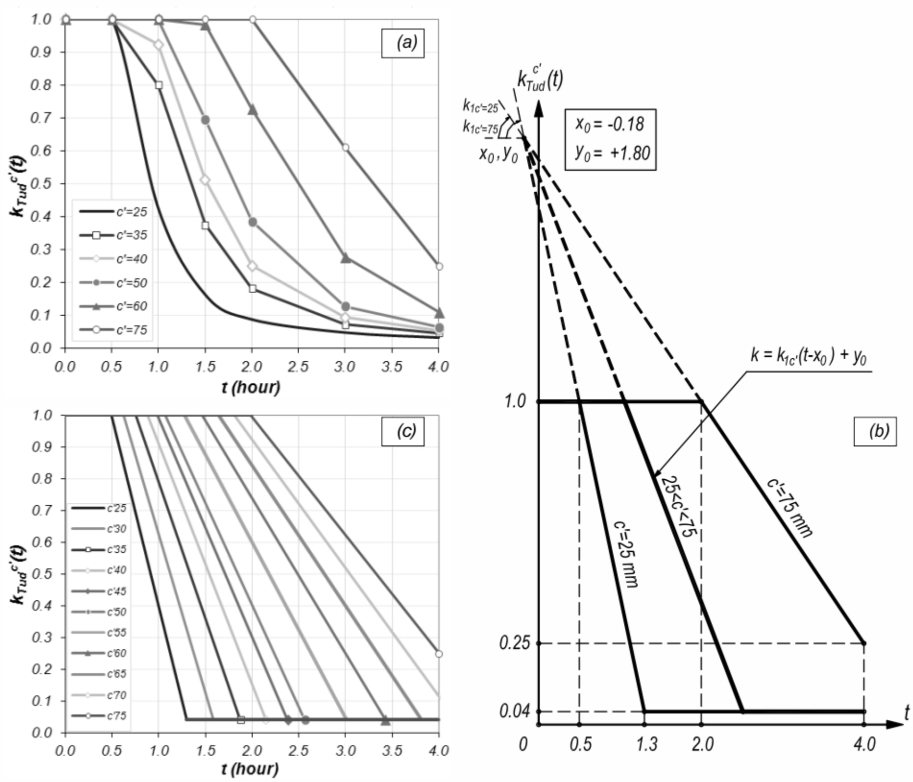


Figure 9. *(a) Results of parametric study on $k_{Tud}^{c'}$ (t) (b) Simplified model (c) Generated model*

It is shown that the reduction in ultimate resistance of longitudinal rebars is more significant (reduction factor decreases) as concrete cover (c') decreases. Based on the curves obtained from Fig. 9(a), a simplified model is proposed to determine the c'-dependent reduction factor $k_{Tud}^{c'}(t)$ as shown in Figs. 9.(b,c) and Eqs. (6).

$$k_{Tud}^{c'}(t) = \left(-0.72\times10^{-3}c'^2 + 75.78\times10^{-3}c' - 2.63\right)\left(t+0.18\right) + 1.8 \qquad \text{for } c'\leq 50 \quad (6a)$$

$$k_{Tud}^{c'}(t) = \left(-0.28\times10^{-3}c'^2 + 45.46\times10^{-3}c' - 2.21\right)\left(t+0.18\right) + 1.8 \qquad \text{for } c'>50 \quad (6b)$$

$$k_{Tud}^{c'}(t) \leq 1.0;\ k_{Tud}^{c'}(t) = \left(85.623e^{-0.061c}\right)\times10^{-4}\times\left(\frac{c}{t}\right)^{0.02c+1.1} \qquad \text{if } k_{Tud}^{c'}(t)\leq 0.25 \quad (6c)$$

where concrete cover c' is in mm, and fire exposure time t is in hour.

4. Conclusions

The general principle of performance-based approach in structural fire-resistant analysis is introduced in this paper. In order to investigate the effect of concrete cover on the axial load resistance of reinforced concrete (RC) columns in fire, the following issues shall be taken in to consideration: fire model, heat transfer and temperature distribution, mechanical properties of materials at elevated temperatures, rational analytical model, fire tests for validation and parametric study. It is quantitatively shown by rational derivations that an increament of the concrete cover leads to less reduction of the axial load resistance of RC columns in fire. In the other words, an explicit expression of the axial load resistance of RC columns in fire based on a reduction factor due to concrete cover is proposed in this paper. The similar approaches shall also be applied to investigate the effects of other parameters on the structural response of RC columns in fire. They are: shape and dimensions of the column cross-section, reinforcement ratio, compressive strength of concrete, type of aggregate, initial load level, etc. The results of the above parametric studies can be efficiently used for simplified assessment procedures for load resistance of RC columns in fire condition. Besides, this approach can also be applied for the RC columns subjected to eccentric loads in fire.

Acknowledgement

The author would like to thank Professor Tan Kang Hai and the Nanyang Technological University (NTU, Singapore) for providing research fund for the experimental study presented herein. The author's thankfulness is also extended to the National University of Civil Engineering (NUCE, Vietnam) for their support on administration issues in writing this paper.

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