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CASE STUDIES ON DETERMINING FIRE RESISTANCE OF UNIAXIAL BENDING REINFORCED CONCRETE COLUMNS SUBJECTED TO 4-SIDE STANDARD FIRE EXPOSURE

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Abstract

This article introduces technical specifications and calculation instructions for fire resistance of uniaxial bending rectangular reinforced concrete (RC) columns under 4-side heating by standard fire. Although a number of research works have been conducted following the Eurocodes to meet the long-term requirement of the country's development in construction, it is shown that the Russian design standard SP 468.1325800.2019 (SP 468) is also applicable for concrete structures subjected to fire in the Vietnam current context. Tabulated data and simplified calculation methods according to SP 468 are introduced, followed by two practical case studies for demonstration. The calculation results show that by using the simplified calculation method the RC columns' fire-resistant limit can be explicitly determined with a particular safety factor ku,T, and a more economical design solution can be obtained compared to that of the tabulated data method. Additionally, RC columns loaded with small eccentricities exhibit greater fire resistance advantages compared to those with large-eccentricity loading. A number of other useful observations are also discussed in the latter part of this article.

Keywords: reinforced concrete; rectangular column; uniaxial bending; eccentric; fire resistance.

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1. Background

Being considered hazardous always, fire incidents potentially claim heavy casualties and huge financial losses, especially for areas having high-density occupants. With an increasing number of fires occurred in civil and industrial buildings recently, researching on fire safety engineering for application in reality is becoming an urgent requirement in Vietnam, where the concrete buildings are relatively more dominant compared to the other types of construction.

Although the national building code and technical regulations on fire safety of buildings and construction have been continuously updated in the past few years [1–4], there is still a lack of Vietnamese design standard for concrete structures in fire, as the current standard TCVN 5574:2018 is only applicable for those at temperatures ranging from −70 °C to +50 °C [5, 6]. It has been planned by the government to prepare for the new direction of construction codes and standards in Vietnam, of which the Eurocodes are to be taken into consideration [7]. Towards this orientation, various number of studies on concrete structures in fire have been conducted by local and regional researchers. Significant attentions have also been paid on reinforced concrete (RC) column, as it is among the

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most critical members and can be considered as the last line of defense when the building structural system is threatened in fires. Numerous fire tests were performed [8–10], based on which a number of analytical approaches [11–18] were proposed to determine fire resistance of rectangular RC columns subjected to eccentric loads and axial restraint using specifications of EC2-1.1 [19] and EC2-1.2 [20]. Finite element program and artificial intelligence approach were also used for investigations of RC members at elevated temperatures [21, 22]. However, it requires a long-term effort to synchronize all the standards in the construction field for the application of the Eurocodes in Vietnam. Hence, during a short-term period of time, it is rational to utilize newly-updated Russian standards as an intermediate stage owing to the following reasons: (i) A significant part of the current Vietnamese standards has been developed from the corresponding ones of Russia in the past; and (ii) Recently, many of Russian standards have been updating in the harmonized manner with the Eurocodes. As a result, the Russian design standard for concrete structures in fire SP 468.1325800.2019 (SP 468) [23] has also been studied for RC beams in Vietnam recently [24–26]. As a continuity of the research series funded by the Ministry of Construction (MOC) and Hanoi University of Civil Engineering (HUCE), this article introduces SP 468's technical specifications and calculation instructions for fire resistance of uniaxial bending rectangular RC columns subjected to 4-side standard fire exposure. The tabulated data and simplified calculation methods according to SP 468 are introduced and applied in two practical case studies to obtain useful observations and discussions.

2. SP 468's specifications for calculation of RC columns' fire resistance

2.1. Standard fire exposure

In SP 468, all the calculations are based on a standard fire exposure, which is expressed as the temperature-time relationship shown In Eq. (1):

$$T = 20 + 345\log_{10}(8t + 1) \tag{1}$$

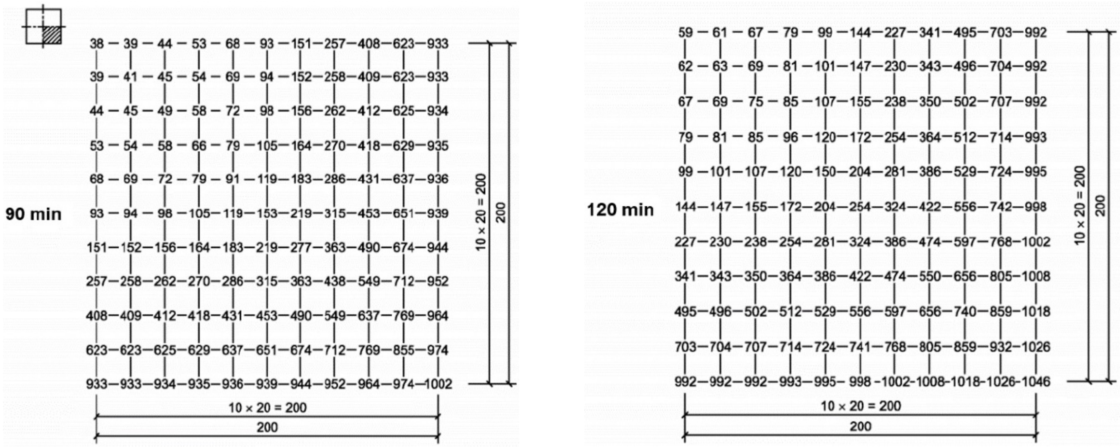
where T (°C) is the temperatures and t is the time in minute counted from the fire starts.

It should be highlighted that Eq. (1) is also similar to that of the standard ISO 834 fire exposure [27], which is commonly used to determine temperature distribution in the cross-section of structural components in the Eurocodes [19, 20]. This sets the basis for the compatibility between SP 468 and EC2-1.2 in calculation of temperature distribution in cross-sections of RC members.

2.2. Temeperature distribution on column cross-section

In fire tests using standard fire curves, so-called the standard fire tests, while beams and slabs are usually heated from beneath, columns are generally heated on all four sides. In real situations, depending on its position on building layout, as well as on the arrangement of partition walls, columns with rectangular cross-sections may be subjected to 1-, 2-, 3- or 4-side fire attacks. The temperatures at different points in RC columns can be determined by heat balance analysis. Those temperatures are lower than the gas temperature since it takes time for the heat transfer process to take place. This process in RC columns can be determined based on the material's thermal properties and heat transfer methods such as radiation, convection, and conduction. Compared to structural steel, concrete has much better fire-resistant properties, such as lower thermal conductivity and greater specific heat capacity.

The appendices of SP 468 provide temperature profiles at various times in a standard fire on a number of typical member cross-sections made of heavy weight concrete having density of 2350 kg/m^3 and the humidity in the range of 2.5-3.0%, siliceous (granite, syenites, diorites) or calcareous aggregates. Figs. 1(a) and 1(b) show the examples of temperature distributions on a quarter of a column having a square cross-section of $b \times h = 400 \times 400$ (mm) with the meshing dimensions of 20×20 (mm) at 90 and 120 min of 4-side standard fire exposure, respectively.



(a) R 90 (b) R 120

Figure 1. Temperature distribution on a quarter of 400×400 (mm) column [23]

2.3. Temperature-dependent mechanical properties of materials

In SP 486 [23], the temperature-dependent mechanical properties of concrete and reinforcing steel can be determined by multiplying those at ambient condition with working condition factors as shown in Eqs. (2) and (3), respectively.

$$R_{b,T} = R_b \gamma_{b,T};\ R_{bn,T} = R_{bn}\gamma_{b,T};\ R_{bt,T} = R_{bt}\gamma_{t,T};\ R_{btn,T} = R_{btn}\gamma_{t,T} \text{ and } E_{b,T} = \beta_{b,T} E_b \tag{2}$$

$$R_{s,T} = R_s \gamma_s;\ R_{sc,T} = R_{sc}\gamma_{s,T};\ R_{sn,T} = R_{sn}\gamma_{s,T};\ R_{scn,T} = R_{scn}\gamma_{s,T} \text{ and } E_{s,T} = \beta_{s,T} E_s \tag{3}$$

In Eq. (2), $R_b(R_{bt})$ and $R_{bn}(R_{btn})$ are the respective design and specified values of compressive (the terms in parentheses are for tensile) strengths of concrete for ultimate limit states at room temperature; $R_{b,T}(R_{bt,T})$ and $R_{bn,T}(R_{btn,T})$ are the corresponding values of the above strengths at elevated temperature T; $\gamma_{b,T}$ and $\gamma_{t,T}$ are the corresponding working condition factors given in Table 1. In this table, the concrete temperature can be taken at a point which locates at a distance of 20% the effective height from the heated surface of the compression zone of the column cross-section.

In Eq. (3), $R_s, R_{sc}, R_{sn}, R_{scn}$ are the design and specified tensile and compressive strengths of the longitudinal rebars for ultimate limit states at room temperature; $R_{s,T}, R_{sc,T}, R_{sn,T}, R_{scn,T}$ are the corresponding values of the above strengths at elevated temperature T, respectively; and $\gamma_{s,T}$ is the corresponding working condition factor given in Table 1, of which the temperature is taken from the centroid of the rebars. It is also noted that the $\gamma_{s,T}$ values in parentheses are for reinforcing steel in CB500 group, whereas the others are for reinforcement groups of CB240, CB300 and CB400.

In Eqs. (2) and (3), E_b and E_s are the respective elastic moduli of concrete and reinforcing steel at ambient condition. All the terms $E_{b,T}$ and $E_{s,T}$ having subscript "T" are the corresponding temperature-dependent elastic modulus. The working condition factors $\beta_{b,T}$ and $\beta_{s,T}$ are also given in Table 1. It is noted that linear interpolation can be applied for any intermediate value of temperature to determine the corresponding working condition factor in this table.

Table 1. Working condition factors of materials to SP 468 [23]

Materials	Factors	Temperature (°C)							
		20	200	300	400	500	600	700	800
Siliceous aggregate concrete	$\gamma_{b,T}$	1.00	0.98	0.95	0.85	0.80	0.60	0.20	0.10
	$\gamma_{t,T}$	1.00	0.65	0.50	0.35	0.20	0.05	-	-
	$\beta_{b,T}$	1.00	0.70	0.50	0.40	0.30	0.20	0.10	0.05
Calcareous aggregate concrete	$\gamma_{b,T}$	1.00	1.00	0.95	0.90	0.85	0.65	0.30	0.15
	$\gamma_{t,T}$	1.0	0.70	0.55	0.40	0.25	0.10	-	-
	$\beta_{b,T}$	1.00	0.75	0.55	0.45	0.35	0.25	0.15	0.10
Hot-rolled reinforcement CB240, CB300, CB400 (CB500)	$\gamma_{s,T}$	1.00 (1.00)	1.00 (1.00)	1.00 (0.90)	0.85 (0.70)	0.60 (0.50)	0.37 (0.30)	0.22 (0.20)	0.10 (0.10)
	$\beta_{s,T}$	1.00	0.92	0.90	0.85	0.80	0.77	0.72	0.65

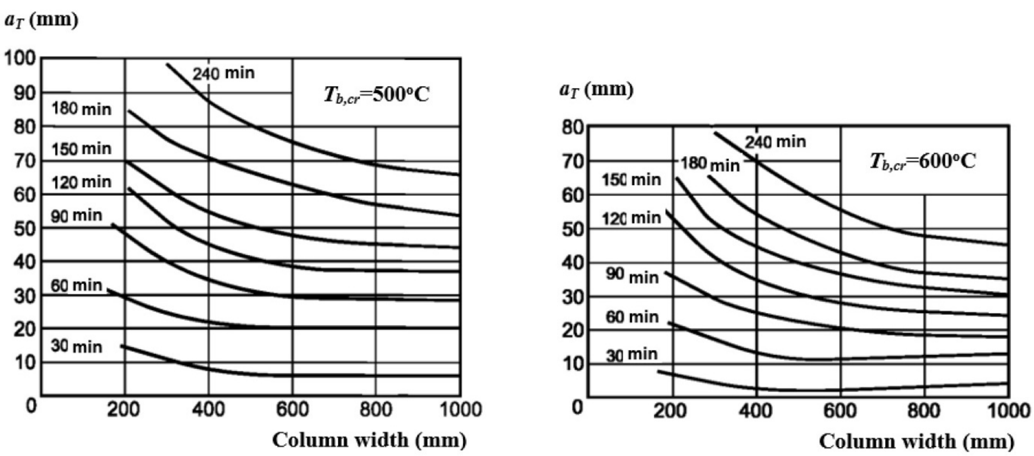
2.4. Reduction of column effective cross-section under standard fire

It is specified in SP 468 [23] that when exposed to 4-side heating of the standard fire, the sufficient column cross-sectional dimensions are constantly reduced with elevated temperature T as follows:

$$b_T = b - 2a_T; \quad h_T = h - 2a_T; \quad h_{0,T} = h_0 - a_T \tag{4}$$

where b, h, h_0 $(b_T, h_T, h_{0,T})$ are respectively the width, the height and the effective height of the column cross-section at room temperature (the variables with subscript T in parentheses are the corresponding dimensions at elevated temperature T); a_T is the depth of the outer concrete layer having temperature higher than a critical temperature $T_{b,cr}$ - this is also the depth of the isothermal contour at temperature $T_{b,cr}$ from the heated surfaces of the column, that can be determined in Fig. 2.

It can be seen in Fig. 2 that the values of $T_{b,cr}$ are 500 °C and 600 °C for siliceous and calcareous concrete, respectively. Besides, the value of a_T is also dependent on the width of the column's cross-section.



(a) Siliceous concrete (b) Calcareous concrete

Figure 2. Determination of a_T based on critical temperature $T_{b,cr}$ [23]

2.5. Calculation methods for columns' fire resistance

a. Tabulated data method

It is specified in SP 468 that in order to satisfy a specific fire resistance R, a column subjected to 4-side standard fire exposure shall meet the requirement in the minimum values of a number of cross-sectional dimensions shown in Table 2. The critical dimensions include the cross-section width b and the distance a from the heated surface to the centroid of longitudinal rebars, so-called the axis distance. In case there are various rebars placed at different levels in the tensile or compression zone, an average axis distance should be calculated based on the cross-sectional area of each rebar $(A_1, A_2, ..., A_n)$ and the corresponding axis distance $(a_1, a_2, ..., a_n)$ as follows:

$$a = \frac{A_1 a_1 + A_2 a_2 + ... + A_n a_n}{A_1 + A_2 + ... + A_n} = \frac{\sum_{i=1}^{n} A_i a_i}{\sum_{i=1}^{n} A_i} \tag{5}$$

Table 2. Minimum dimensions for fire resistance of RC columns to SP 468 [23]

Fire exposure	Column dimensions	Minimum values with fire resistance (mm)					
		R30	R60	R90	R120	R150	R180
4-side heating	Cross-section width b Axis distance a	150 10	200 25	240 35	300 40	400 50	450 50

It is noted that the data given in Table 2 are only applicable for RC columns under uniaxial or biaxial bending having longitudinal rebars ratios $\mu_s = (A_s + A'_s)/bh_0$ not being higher than 3%, otherwise the simplified calculation method shall be alternatively used for determination of column fire resistance [23].

b. Simplified calculation method

At room temperature, uniaxial bending columns are initially designed based on the ultimate limit state, which requires that the design internal forces (M, N) acting on the column's normal sections due to combinations of factored loads do not exceed the load resistance (M_u, N_u) calculated from the design strengths at ambient condition of concrete and reinforcing steel [6]. In the standard fire condition, this column is exposed to heating from the surfaces and the temperatures at each point in the cross-section constantly increase with time and distribute as specified in Section 2.2, leading to the reduction of materials' strengths at individual positions following specification in Section 2.3. As a result, the resultant resistance forces of the cross-section also decrease. Then, the column fire resistance will be determined based on the accidental limit states. If after a period of time R (in minute), the column load resistances $(M_{u,T}, N_{u,T})$ - calculated from the specified strengths of concrete and reinforcing steel at elevated temperature T - are not lower than the internal forces $(M_{n,T}, N_{n,T})$ calculated from that of (M_n, N_n) due to unfactored long-term loads, the column is considered to be capable of qualifying the fire resistance R [23]. It is noteworthy that this simplified method adopts calculation expressions similar to those at ambient condition, ignoring the thermal strain which is only associated in the advanced calculation method.

It is specified in SP 468 that the fire resistance based on strength criteria R in minute (i.e., R 30, R 60, R 90, R 120, R 180 and R 240) of RC columns in uniaxial bending is determined as follows:

$$\text{If } N_{n,T} = N_{u,T} \text{ then } M_{n,T} \leq M_{u,T} \tag{6}$$

where $M_{n,T}, N_{n,T}$ are the respective bending moment and axial load on the column cross-section and $M_{u,T}, N_{u,T}$ are the respective temperature-dependent bending moment and axial load resistance at the corresponding time of 30, 60, 90, 120, 180 and 240 min in the standard fire exposure introduced in Section 2.1, which can be calculated based on the materials' specified strengths shown in Eqs. (2), (3) as well as the effective dimensions of the column's cross-section specified in Section 2.4.

For 4-side heated columns, Eq. (6) can be expressed in more details based on the forces diagram shown in Fig. 3.

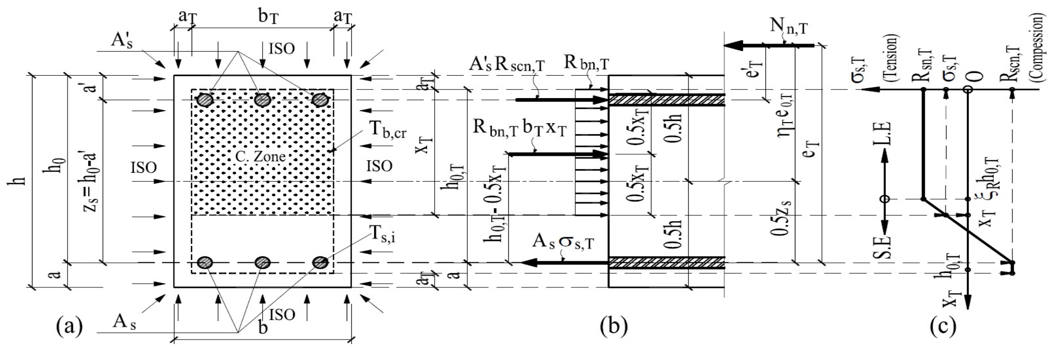


Figure 3. Force analysis of uniaxial bending column in fire
a) Column cross-section; b) Forces diagram; c) Determination of stress $\sigma_{s,T}$ in A_s

It can be seen in Fig. 3(b) that when taking moment with the axis going through the cross-sectional centroid of the reinforcement A_s and perpendicular to the flexural plane, Eq. (6) can be derived as follows:

$$N_{n,T}e_T \leq R_{bn,T}b_T x_T(h_{0,T} - 0.5x_T) + R_{scn,T}A'_s(h_0 - a') \tag{7}$$

where the height x_T of the compression zone is determined from the condition that $N_{n,T} = N_{u,T}$. When $x_T \leq \xi_R h_{0,T}$, the column is loaded in large eccentricity (L.E), it is shown in Fig. 3(b) that the stress in rebars A_s reaches

$$x_T = \frac{N_{n,T} + R_{sn,T}A_s - R_{scn,T}A'_s}{R_{bn,T}b_T} \tag{8}$$

It is noted in Eq. (8) that when $R_{sn,T} = R_{scn,T}$ and $A_s = A'_s$, one has $x_T = \dfrac{N_{n,T}}{R_{bn,T}b_T}$. Then the condition $x_T \leq \xi_R h_{0,T}$ can be rewritten as $\alpha_{n,T} = \dfrac{N_{n,T}}{R_{bn,T}b_T h_{0,T}} \leq \xi_R$.

When $x_T > \xi_R h_{0,T}$, the column is loaded in small eccentricity (S.E), Fig. 3(c) shows that the stress $\sigma_{s,T}$ in rebars A_s can be calculated by linear interpolation, then:

$$x_T = \frac{N_{n,T} + R_{sn,T}A_s\dfrac{1+\xi_R}{1-\xi_R} - R_{scn,T}A'_s}{R_{bn,T}b_T + \dfrac{2R_{sn,T}A_s}{h_0(1-\xi_R)}} \tag{9}$$

In Eq. (6), e_T is the eccentricity from the axial load $N_{n,T}$ to the centroid of rebars A_s and can be determined as follows:

$$e_T = \eta_T e_{0,T} + 0.5(h_0 - a') \tag{10}$$

where $e_{0,T}$ is the static eccentricity $e_{0,T} = \dfrac{M_n}{N_n}$; η_T is the moment magnification factor $\eta_T = \dfrac{1}{1 - \dfrac{N_n}{N_{cr,T}}}$; critical axial load $N_{cr,T} = \dfrac{\pi^2 D}{l_0^2}$; l_0 is the column effective height; cross-sectional stiffness of the column $D = \dfrac{0.15 E_{b,T} I_{b,T}}{\varphi_l (0.3 + \delta_e)} + 0.7 E_{s,T} I_s$; coefficient taking account of the long-term effect of loading at fire condition $\varphi_l = 2$, coefficient $\delta_e = \max\left(\dfrac{e_0}{h_T}, 0.15\right)$; $I_{b,T} = b_T h_T^3/12$ and $I_s = (A_s + A'_s)(0.5h - a)^2$ are respectively the moment inertias of effective concrete and cross-section of all the rebars to the centroid of the column concrete cross-section.

Based on Eq. (10), Eq. (7) can be further derived as below:

$$N_{n,T}\left[\eta_T e_{0,T} + 0.5(h_0 - a')\right] \leq R_{bn,T} b_T x_T (h_{0,T} - 0.5x_T) + R_{scn,T} A'_s (h_0 - a') \tag{11}$$

$$N_{n,T} \eta_T e_{0,T} \leq R_{bn,T} b_T x_T (h_{0,T} - 0.5x_T) + (R_{scn,T} A'_s - 0.5N_{n,T})(h_0 - a') \tag{12}$$

In Eq. (12), the left-hand-side term can be referred to as $M_{n,T} = N_{n,T}\eta_T e_{0,T}$, whereas the right-hand-side term is also the resistance $M_{u,T}$ of the column at T and can be defined as $M_{u,T} = R_{bn,T} b_T x_T (h_{0,T} - 0.5x_T) + (R_{scn,T} A'_s - 0.5N_{n,T})(h_0 - a')$, so that the expression in Eq. (6) is clearly demonstrated. Set the safety factor $k_{u,T} = M_{u,T}/M_{n,T}$. At a certain time R (in minute) of the standard fire, the column is said to be capable of reaching the fire resistance R as long as $k_{u,T} \geq 1$.

3. Case studies

3.1. Case study No. 1

Example 4.6 of [28] is used for this case study. A statically indeterminate RC column having square cross-section of $b \times h = 400 \times 400$ (mm) and effective length of $l_0 = 3600$ mm is reinforced by 3Φ25 in tension ($A_s = 1471$ mm^2) and other 3Φ25 in compression ($A'_s = 1471$ mm^2). The concrete cover to longitudinal rebars is $c = 25$ mm. The siliceous concrete compressive class is B25 and reinforcing steel class is CB400-V. Hence, one has $R_b = 14.5$ MPa, $R_{bn} = 18.5$ MPa, $R_s = R_{sc} = 350$ MPa, $R_{sn} = R_{scn} = 400$ MPa. The concrete casting is in vertical direction at the height of higher than 1.5 m. with the working-condition factor of $\gamma_{b3} = 0.85$. The column is in uniaxial bending with design internal forces of $M = 200$ kNm and $N = 750$ kN. The corresponding long-term values are $M_l = 99$ kNm and $N_l = 562.5$ kN. It is determined in Example 4.6 of [28] that at room temperature the column is uniaxially loaded in small eccentricity (S.E) with the moment resistance to the cross-sectional centroid of the tensioned rebars of $M_u = 260.45$ kNm whereas the corresponding acting moment is $M_E = 232.94$ kNm, meaning that the column has sufficient load-bearing capacity with a safety factor of $k_u = M_u/M_E = 1.12 > 1$. It is required to determine the fire resistance of the column when heated from all sides based on SP 468 specifications introduced in Section 2.

a. Tabulated data method

It is determined from Section 2.5.a that the axis distance to the centroid of the longitudinal rebars is $a = c + 0.5\Phi = 37.5$ mm. Then, the column critical dimensions are both larger than the corresponding required minimum values of $b_{\min} = 240$ mm and $a_{\min} = 35$ mm of R 90 in Table 2, respectively. Besides, the column fire resistance cannot reach R 120 since although the column width satisfies $b = 400 > b_{\min} = 300$ mm, but the axis distance $a = 37.5$ mm is lower than the required value of $a_{\min} = 40$ mm at R 120. Hence, if the column is statically determinate, its fire resistance is R 90.

b. Simplified calculation method

Fig. 4 illustrates the cross-section of the RC column in this case study. The fundamental dimensions determined from Fig. 4(a) are $a = a' = 37.5$ mm, $h_0 = h-a = 362.5$ mm, $z_s = h_0-a' = 325$ mm. Besides, for siliceous concrete and the width $b = 400$ mm, it is determined from Fig. 2(a) that $a_T = 45$ mm at R 120 (Fig. 4(d)).

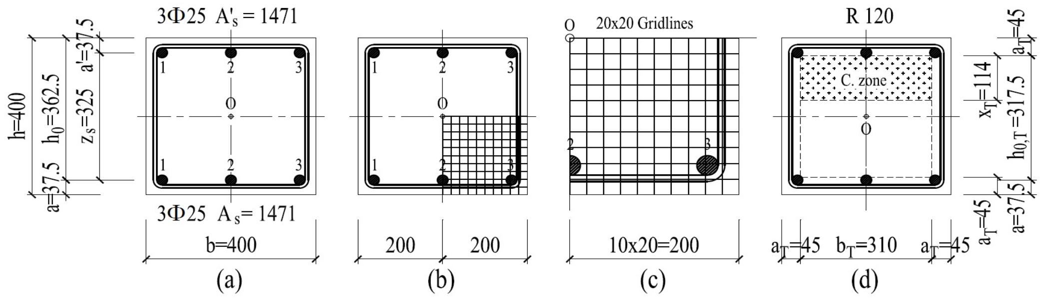


Figure 4. Cross-section of column in Case study No. 1

In order to calculate the safety factor $k_{u,T}$ at R 120, temperatures at rebars No. 1, 2, 3 (Figs. 4(b) and 4(c)) are determined based on Fig. 1(b). It can be seen that rebars No. 1 and No. 3 have the same relative positions to the nearest surfaces then will have identical temperatures. Fig. 1(b) shows that at R 120, the temperatures of the four nearest points on gridlines are 740, 859, 859, and 932 °C. Using linear interpolation principal based on the relative distance from the centroid of rebars No. 3 to the four nearest reference points on gridlines, it can be calculated that $T_1 = T_3 = 769$ °C. Similarly, with the temperatures at the two nearest points on gridlines are 495 and 703 °C (Fig. 1(b)), the temperature at rebar No.2 can also be linearly interpolated as $T_2 = 521$ °C. The results of the rebars' temperature at R30, 60, 90, 120, 180 and 240 are also shown in Table 3.

Table 3. Case study No. 1 Temperature of rebars (°C)

Point	R30	R60	R90	R120	R180	R240
1, 3	260	500	669	769	900	986
2	152	320	345	521	693	754

Based on the data shown in Table 3 and Table 1, the working condition factors of rebars for strength $\gamma_{s,T}$ and elastic modulus $\beta_{s,T}$ are determined as shown in Table 4. It is noted that the values of $\beta_{s,T}$ are in parentheses. Besides, linear interpolation principal is applied for intermediate values and average factors are also determined for calculation.

Table 4. Case study No.1 - Working condition factors $\gamma_{s,T}$ ($\beta_{s,T}$) of rebars

Point	R30	R60	R90	R120	R180	R240
1, 3	1.0 (0.908)	0.650 (0.800)	0.267 (0.736)	0.144 (0.676)	0 (0)	0 (0)
2	1.0 (0.941)	0.963 (0.888)	0.780 (0.833)	0.591 (0.794)	0.231 (0.724)	0.155 (0.682)
Ave.	1.0 (0.919)	0.754 (0.829)	0.438 (0.768)	0.293 (0.715)	0.071 (0.241)	0.052 (0.227)

The long-term unfactored internal forces acting on the column cross-section at fire limit states are $M_n = M_l/1.1 = 90.0$ kNm and $N_n = N_l/1.1 = 511.364$ kN. This is because SP 468 specified that the win load is not considered in fire design situation. Besides, the long-term imposed loads are relatively lower than the long-term permanent loads. Then, calculations based on Section 2.5.b are conducted by the authors to give results shown in Table 5. It is noted in the calculation that the coefficient $\xi_R = 0.533$.

Table 5. Case study No. 1 - Calculation results

Parameters	R30	R60	R90	R120	R180	R240
a_T (mm)	8	22	35	45	70	88
b_T (mm)	384	356	330	310	260	224
h_T (mm)	384	356	330	310	260	224
$h_{0,T}$ (mm)	354.5	340.5	327.5	317.5	292.5	274.5
$0.2h_{0,T}$ (mm)	70.9	68.1	65.5	63.5	58.5	54.9
T_c (°C)	49	125	228	330	490	630
$\gamma_{b,T}$	0.997	0.998	0.972	0.920	0.850	0.480
$\beta_{b,T}$	0.952	0.825	0.644	0.470	0.310	0.170
$N_{cr,T}$ (kN)	11517.801	9258.579	7679.529	6645.676	2208.154	1941.229
η_T	1.046	1.058	1.071	1.083	1.301	1.358
$\alpha_{n,T}$	0.239	0.268	0.301	0.330	0.428	0.529
Eccentricity	L.E	L.E	L.E	L.E	L.E	L.E
x_T (mm)	84.9	92.5	101.4	114.0	155.4	302.4
$M_{u,T}$ (kNm)	267.905	211.794	142.305	106.200	41.493	−10.166
$M_{n,T}$ (kNm)	94.181	95.261	96.420	97.503	117.123	122.185
$k_{u,T}$	2.845	2.223	1.476	1.089	0.3554	−0.083

The results for R 120 in Table 5 are shown in Fig. 4(d) for demonstration. With $\alpha_{n,T} = 0.330 < \xi_R = 0.533$, the column is in large-eccentricity (L.E) loading, the compression zone height x_T = 114 mm based on Eq. (8). Then, at R 120 one has $M_{u,T}$ = 106.2 kNm, $M_{n,T} = N_{n,T}\eta_T e_{0,T}$ = 97.503 kNm and $k_{u,T} = M_{u,T}/M_{n,T} = 1.089 > 1$. Hence, it can be concluded that the column fire resistance is R 120.

3.2. *Case study No. 2*

Example 4.9 of [28] is used for this case study. Another statically indeterminate RC column having rectangular cross-section of $b \times h = 400 \times 700$ (mm) and the effective length of l_0 = 6000 mm is reinforced by 8Φ22 equally divided into both sides of the cross-section ($A_s = A'_s$ = 1520 mm^2) with the concrete cover to longitudinal rebars of c = 25 mm. The materials are similar to those in Case study No. 1 (B25 and CB400-V). The working-condition factor of concrete is $\gamma_{b3} = 1$. The column is uniaxially loaded by the design internal forces of M = 255 kNm, N = 3905 kN and the corresponding long-term values of M_l = 114.8 kNm and N_l = 2928.75 kN. It is also determined in Example 4.9 of [28] that at room temperature the column is in uniaxial bending with large eccentricity. The moment resistance to the cross-sectional centroid of the tensioned rebars is M_u = 351.88 kNm and the acting moment is M_E = 293.25 kNm. It means that at ambient condition the column also has sufficient load-bearing capacity with a safety factor of $k_u = M_u/M_E = 1.2 > 1$. The column fire resistance when subjected to 4-side heating will also be determined according to SP 468.

a. Tabulated data method

It can be seen that similar to the column in Case study 1, with b = 400 mm and $a = c + 0.5\Phi$ = 36 mm, if being statically determinate the column in Case study 2 can only reach fire resistance of R 90 following the SP 468's tabulated requirements indicated in Table 2.

b. Simplified calculation method

The cross-section of the RC column in this case study is illustrated in Fig. 5.

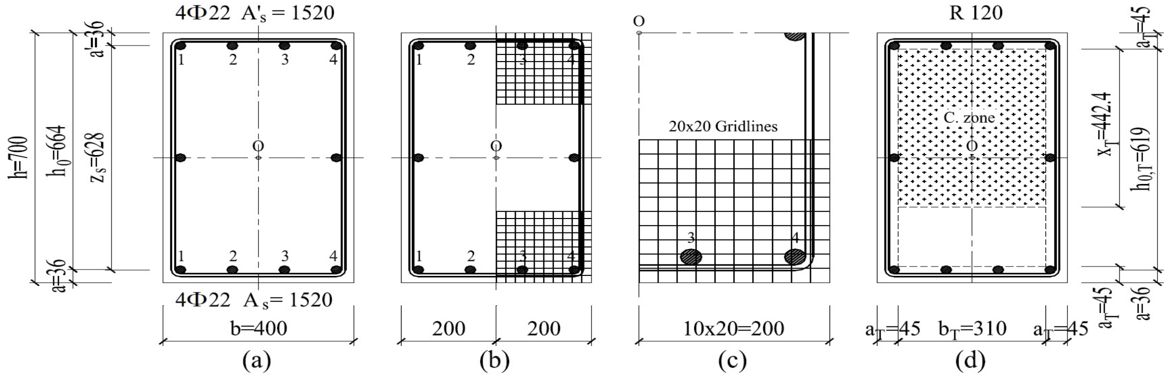


Figure 5. Cross-section of column in Case study No. 2

The temperatures and the corresponding working condition factors are determined similarly with Case study No. 1. The results are shown in Tables 6 and 7.

Table 6. Case study No. 2 - Temperature of rebars (°C)

Point	R30	R60	R90	R120	R180	R240
1, 4	307	511	694	768	914	997
2, 3	165	338	459	550	687	788

Table 7. Case study No. 2 - Working condition factors $\gamma_{s,T}$ ($\beta_{s,T}$) of rebars

Point	R30	R60	R90	R120	R180	R240
1, 4	0.99 (0.897)	0.619 (0.797)	0.229 (0.723)	0.117 (0.660)	0 (0)	0 (0)
2, 3	1.00 (0.936)	0.943 (0.881)	0.732 (0.821)	0.510 (0.785)	0.240 (0.727)	0.114 (0.658)
Ave.	0.995 (0.917)	0.791 (0.839)	0.481 (0.772)	0.314 (0.723)	0.120 (0.364)	0.057 (0.329)

The long-term unfactored internal forces acting on the column cross-section at fire limit states are $M_n = M_l/1.1 = 104.364$ kNm and $N_n = N_l/1.1 = 2662.5$ kN. The calculation results of fire resistance are shown in Table 8.

Table 8. Case study No. 2 Calculation results

Parameters	R30	R60	R90	R120	R180	R240
a_T (mm)	8	22	35	45	70	88
b_T (mm)	384	356	330	310	260	224
h_T (mm)	684	656	630	610	560	524
$h_{0,T}$ (mm)	656	642	629	619	594	576
$0.2h_{0,T}$ (mm)	131.2	128.4	125.8	123.8	118.8	115.2
T_c (°C)	23	40	68	99	204	339
$\gamma_{b,T}$	1.0	0.998	0.995	0.991	0.979	0.911

Parameters	R30	R60	R90	R120	R180	R240
$\beta_{b,T}$	0.995	0.967	0.920	0.868	0.692	0.461
$N_{cr,T}$ (kN)	28137.908	23541.612	19657.745	16933.780	8573.060	5823.901
η_T	1.105	1.128	1.157	1.187	1.450	1.842
$\alpha_{n,T}$	0.571	0.630	0.693	0.750	0.932	1.115
Eccentricity	S.E	S.E	S.E	S.E	S.E	S.E
x_T (mm)	365.8	384.6	4112.9	442.4	540.4	679.8
$M_{u,T}$ (kNm)	773.449	599.118	407.301	283.889	33.796	-208.337
$M_{n,T}$ (kNm)	115.271	117.672	120.713	123.834	151.376	192.258
$k_{u,T}$	6.710	5.091	3.374	2.292	0.223	−1.084

The results for R 120 in Table 8 are also shown in Fig. 5(d) for demonstration. With $\alpha_{n,T} = 0.75 > \xi_R = 0.533$, the column is in small-eccentricity (S.E) loading, the height of the compression zone $x_T = 442.4$ mm based on Eq. (9) and Eq. (12) is used to have $M_{u,T} = 283.889$ kNm and $M_{n,T} = N_{n,T}\eta_T e_{0,T} = 123.834$ kNm. At R 120 the safety factor is determined as $k_{u,T} = M_{u,T}/M_{n,T} = 2.292 > 1$, whereas at R 180 one has $k_{u,T} = 0.223 < 1$. Hence, it can be concluded that based on simplified calculation method, the column fire resistance is R 120.

3.3. Discussions on the calculation results

A number of observations and discussions can be made from the results of the conducted case studies as below.

- For both cases of columns under large and small eccentricities (L.E and S.E), it is shown that the fire resistance determined by simplified calculation method is R 120, which is higher than that of R 90 based on tabulated data method. However, it is noted that the latter method is only applicable for statically determinate structural elements.

- In the tabulated data method, it is not rational to use dimensions $b_{\min}$ and axis distance a as individual criterion for fire resistance. According to this method, both the columns in the above case studies having the width $b > b_{\min} = 300$ mm but the axis distance $a < a_{\min} = 40$ mm, they are still not be able to reach R 120, which is confirmed by the simplified method. Besides, it can be noted that other important factors such as the reinforcement ratio μ_s, the safety factor at room temperature k_u, the column cross-section height, material strengths, etc. are not considered in the tabulated data.

- In the simplified calculation method, the results can be explicitly obtained in the form of safety factor at elevated temperatures $k_{u,T}$. Based on the results shown in Table 5 and Table 8, the progressively-deteriorating moment resistance of $M_{u,T}$, the revolution of the acting internal bending moment in fire $M_{n,T} = N_{n,T}\eta_T e_{0,T}$, as well as the safety factor $k_{u,T}$of the columns in Case studies No. 1 and No. 2 can be graphically depicted in Figs. 6(a) and 6(b), respectively.

- It is clearly shown in Fig. 6(a) that at about 128 min of the standard fire, the column in Case study No. 1 has $k_{u,T} = 1$ and can be considered to qualify R 120. Similarly, Fig. 6(b) shows that at about 154 min, the column in Case study No. 2 has $k_{u,T} = 1$ and even qualifies a higher resistance of R 150 based on graphical analysis.

- It can also be observed in Fig. 6 that compared to the column in Case study No. 1 (large eccentricity - L.E), the column in small eccentricity - S.E (Case study No. 2) has higher safety factor in the initial stage of the standard fire. This is because in the S.E column, more concrete of the cross-section with slower development in temperature and slower reduction in strength are mobilized in load bearing capacity. Besides, in the column of Case study No. 2, the percentage to the overall ratio of the rebars at the cross-section corners (with higher temperature revolution and faster strength reduction) of $4/8 = 50\%$ is lower than that of $4/6 = 66.6\%$ of the column in Case study No. 1.

- The Russian standard SP 468 also specifies tabulated data for 1-side heated RC columns. In the simplified calculation method, if the columns are subjected to 1-, 2- or 3-side heating, and additional eccentricity $e_{\Delta T}$ accounting for the differential thermal effect will be considered in Eq. (10) as $e_T = \eta_T e_{0,T} + 0.5(h_0 - a') + e_{\Delta T}$. This so-called thermal eccentricity is dependent on the connecting conditions at the column ends and the thermal expansion coefficients of concrete and rebars, which are in the relationships with temperatures at the cooler and the hotter surfaces of the column, respectively.

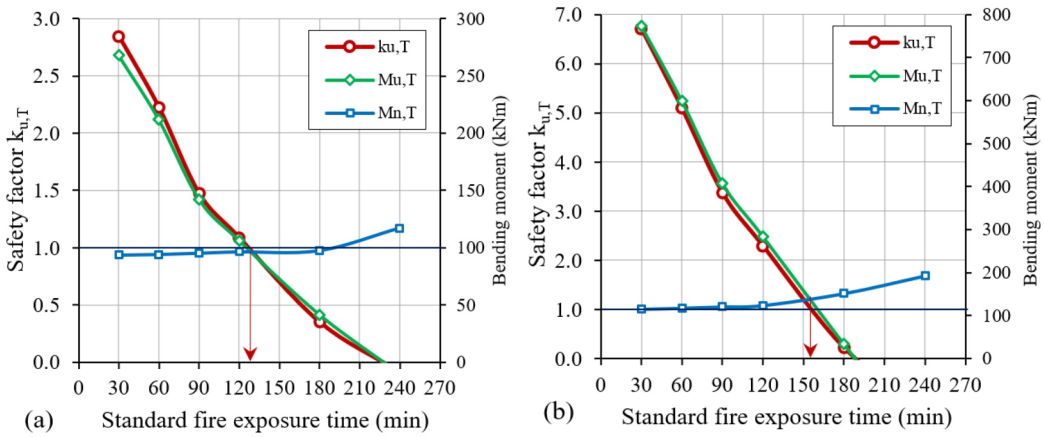


Figure 6. Graphic demonstrations of simplified calculation method results
a) Case study No. 1; b) Case study No. 2

4. Conclusions

Together with the long-term orientation towards applying the Eurocodes for Vietnamese codes and standards in construction field, the recently-updated Russian design standards are also the suitable materials for short-term application in Vietnam. The design standard SP 468.1325800.2019 provides comprehensive specifications of temperature distribution on cross-sections, temperature-dependent mechanical properties of materials and a number of calulation methods for concrete structures in fire that can be used in a compatible and sufficient manner in Vietnam conditions. Within the scope of the case studies performed in this article, the following observations can be made: (i) Compared to those of the tabulated data method, the result obtained from the simplified calculation method is more rational with an explicit safety factor $k_{u,T}$ and is capable of providing more economical design solutions for statically determinate reinforced concrete structures; and (ii) Reinforced concrete columns that are uniaxially loaded with small eccentricities have higher safety factor at the initial stage of the standard fire, compared to that of RC columns with large eccentricities of the uniaxial bending.

Although the aforementioned simplified approaches are practical, there are still future research works such as experimental studies in local conditions and finite element analysis that should be conducted to establish data library for the up-coming Vietnamese design standard as well as for more advanced approaches, so-called performance-based analysis of RC structures in fire. Besides, further studies are also needed for fire resistance of 1-, 2- and 3-side heated RC columns subjected to various ratios between variable loads and permanent loads. Furthermore, not only RC columns in uniaxial bending but also biaxially-loaded columns should be studied for actual fire design situations.

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