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P-M interaction curve for reinforced concrete columns exposed to elevated temperature

Hyun Kang[1a], Na-Rae Cheon[2b], Deuck Hang Lee[1c], Jungmin Lee[1d], Kang Su Kim[*1] and Heung-Youl Kim[3e]

[1]*Department of Architectural Engineering, University of Seoul, 163 Seoulsiripdae-ro, Dongdaemun-gu, 02504, Seoul, Republic of Korea*
[2]*Metallic Materials and Mechanical Engineering Team, Korea Testing & Research Institute, 98 Gyoyukwon-ro, Gwacheon-si, 13810, Gyeonggi-do, Republic of Korea*
[3]*Fire Safety Research Division, Korea Institute of Construction Technology, 182-64 Mado-ro, Mado-myeon, Hwaseong-si, 18544, Gyeonggi-do, Republic of Korea*

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Abstract. The strength and deformational capacity of slender reinforced concrete (RC) columns greatly rely on their slenderness ratios, while an additional secondary moment (i.e., the P-δ effect) can be induced especially when the RC column members are exposed to fire. To evaluate the fire-resisting performances of RC columns, this study proposed an axial force-flexural moment (i.e., P-M) interaction curve model, which can reflect the fire-induced slenderness effects and the nonlinearity of building materials considering the level of stress and the magnitude of temperature. The P-M interaction model proposed in this study was verified in detail by comparing with the fire test results of RC column specimens reported in literature. The verification results showed that the proposed model can properly evaluate the fire-resisting performances of RC column members.

Keywords: reinforced concrete; column; fire resistance; P-M interaction curve; secondary moment; slenderness ratio

1. Introduction

Column members are the most important core structural components in reinforced concrete (RC) buildings and infrastructures, and thus, their sufficient reserved strengths and deformational capacities shall be secured even when they are exposed to fire; otherwise, it may cause not only collapse of the column members but also the stability of a whole structure, and a proper fire-resisting design is thus very important (Lee *et al*. 2013, Raut and Kodur 2011, and Caldas *et al.* 2010). The current fire design standards are undergoing a transition from the prescriptive design approach to the performance-based fire design (PBFD) methods (Heo *et al.* 2016, KCI-M-07 2007, EN 1992-1-2 2004, and ACI Committee 216 2007), and many researchers are developing fire-resisting performance evaluation methods for RC columns based on the PBFD. Yao *et al.* (2008) proposed an axial strength model for RC columns exposed to fire using Rankine method, (Rankine 1908) and Kodur and Raut (2012) also proposed an evaluation model of the fire resistance performance for RC columns considering the effects of slenderness ratio, load ratio, effective cover thickness, longitudinal reinforcement ratio, eccentricity of axial force, and type of aggregate. Caldas *et al*. (2010) and El-Fitiany *et al*. (2009, 2014) also presented axial force-flexural moment (P-M) interaction curve approaches for RC column members exposed to fire considering their thermal deformation. This study presented a new P-M interaction curve estimation method for RC columns exposed to fire, in which the second-order effect induced by large deformations was considered in a simple but rational manner.

∗Corresponding author, Professor
E-mail: kangkim@uos.ac.kr
[a]Ph.D. Candidate
[b]Researcher
[c]Ph.D. Research Professor
[d]Graduate Student
[e]Research Fellow

2. P-M interaction curve for RC columns exposed to fire

This study aimed to evaluate the fire-resisting performances of RC columns using the P-M interaction curve approach considering the material strength degradations and the additional secondary effect (P-δ effect) induced by fire damages.

Fig. 1 shows typical strength degradation behaviors of an RC column depending on the fire exposure time through P-M interaction curves. With an increase in the fire exposure time (t), the potential axial and flexural capacities of the RC column member rapidly decrease due to fire damages induced by the temperature rise. In other words, the column behavior changes because the column becomes slender compared to that at room temperature due to the loss of cross-section area caused by fire damage (Tan and Yao 2003). By using the P-M interaction curve, all the

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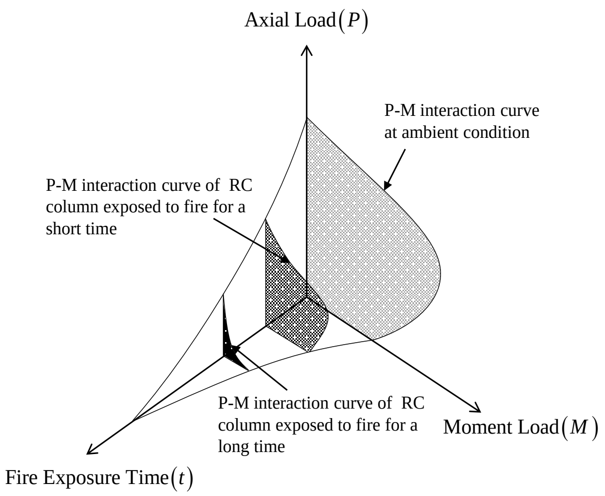
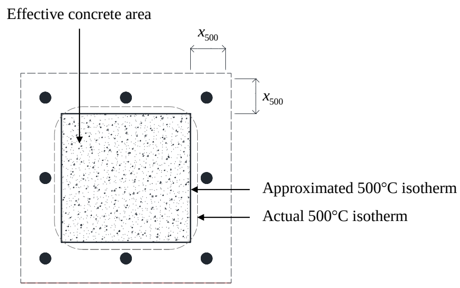
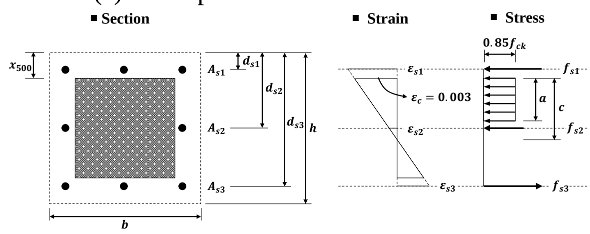


Fig. 1 Strength degradations of a reinforced concrete column due to fire damage



(a) Concept of 500℃ isotherm method



(b) Strain and stress distribution of a reinforced concrete column exposed to fire

Fig. 2 Sectional analysis based on 500℃ isotherm method

maximum load combinations of the RC column at the target fire exposure time required in the performance-based fire design can be estimated, and the maximum fire-resisting time of the RC column under a specific load combination can be determined.

2.1 Temperature distribution in reinforced concrete section

The complicated temperature distribution in the concrete cross-section exposed to fire is one of the difficulties for simple estimation of the fire-resisting performances of RC members (Tan and Yao 2003). In this study, therefore, the 500℃ isotherm method presented in EN 1992-1-2 Annex B was utilized to estimate the axial and flexural capacities of RC cross-sections. As shown in Fig. 2(a), the 500℃ isotherm method assumes that the concrete section exposed to a higher temperature than 500℃ loses all of its structural resistance, while the section exposed to below 500℃ does not have any degradation in its strength and stiffness (Anderberg 1978). The temperature distribution on the reinforced concrete cross-section exposed to fire can be closely calculated using the finite difference method or the finite element method. However, these analysis methods require technical expertise in the numerical analysis including deep knowledge on heat transfer, etc., which means that they may not be easy to be used in practice (Caldas *et al.* 2010, Kang *et al.* 2015). Thus, this study adopted the simplified temperature estimation model presented by Wickstrom (1986) that was derived based on several analysis results by TASEF-2, a software for analyzing the temperature of structures exposed to fire. Based on the Wickstrom's formula, the temperature distribution in an RC cross-section can be easily calculated (Wickstrom 1986 and Purkiss 2007), and assuming that the temperature of the reinforcement is identical to that of concrete at the same position, the temperature of the reinforcements can be calculated as follows

$$T_s = \Delta T + 20 = n_x n_w \Delta T_f + 20 \tag{1}$$

where ΔT is temperature increase at the level of the reinforcing bar, n_x is the factor to take into account for heat transfer through the concrete section, calculated as $0.18\ln(\gamma t/x^2)-0.81$, t is the fire exposure time, x is the depth of the reinforcing bar from the fire exposure surface, γ is the ratio of the thermal diffusivity of concrete, which is 1.0 for normal-weight concrete, n_w is the factor for calculating the surface temperature of the concrete section that is taken to be $1-0.0616t^{-0.88}$, and ΔT_f is the temperature rise of the fire that is taken as $345\log(8t+1)$ based on the standard fire curve. (ISO 834-1:1999(E) 1999) Assuming that ΔT is 480℃ (Purkiss 2007) and the thermal diffusivity (a) is 0.417×10^{-6} as commonly used for normal-weight concrete, the 500℃ penetration depth (x_{500}) shown in Fig. 2(a) can be derived from Eq. (1), as follows

$$x_{500} = \left[\frac{1}{\exp\left(4.5 + \dfrac{480}{0.18 n_w \Delta T_f}\right)}\right]^{0.5} \tag{2}$$

2.2 P-M interaction curves using 500°C isotherm method

This study utilized the 500℃ isotherm method to reflect the influence of fire damages on the axial and flexural strengths of RC column members in a simple manner. Fig. 2(b) shows the sectional analysis method for estimating the strengths of an RC compression member exposed to an elevated temperature based on the concept of the 500℃ isotherm method. The effective sectional area of the concrete reduced by fire damage ($A_{reduced}$) can be calculated as follows

$$A_{reduced} = (b-2x_{500})(h-2x_{500}) \quad \text{for } x_{500} > (c_c + d_b) \tag{3a}$$

$$A_{reduced} = (b-2x_{500})(h-2x_{500}) - A_s \quad \text{for } x_{500} \le (c_c + d_b) \tag{3b}$$

Table 1 Reduction coefficients for steel according to temperature (EN 1992-1-2 2004)

Reduction coefficients		
Yield strength	for 20℃ ≤ T ≤ 100℃	$K_s(T) = 1.0$
	for 100℃ < T ≤ 400℃	$K_s(T) = 0.7 - 0.3(T - 400)/300$
	for 400℃ < T ≤ 500℃	$K_s(T) = 0.57 - 0.13(T - 500)/100$
	for 500℃ < T ≤ 700℃	$K_s(T) = 0.1 - 0.47(T - 700)/200$
	for 700℃ < T ≤ 1200℃	$K_s(T) = 0.1(1200 - T)/500$
Elastic modulus	for 20℃ ≤ T ≤ 100℃	$K_E(T) = 1.0$
	for 100℃ < T ≤ 500℃	$K_E(T) = 1.1 - 0.001T$
	for 500℃ < T ≤ 600℃	$K_E(T) = 2.05 - 0.0029T$
	for 600℃ < T ≤ 700℃	$K_E(T) = 1.39 - 0.0018T$
	for 700℃ < T ≤ 800℃	$K_E(T) = 0.41 - 0.0004T$
	for 800℃ < T ≤ 900℃	$K_E(T) = 0.25 - 0.0002T$
	for 900℃ < T ≤ 1000℃	$K_E(T) = 0.34 - 0.0003T$
	for 1000℃ < T ≤ 1200℃	$K_E(T) = 0.24 - 0.0002T$

where c_c is the net concrete cover thickness and d_b is the diameter of the reinforcing bar. To avoid double counting of the reinforcing bar area in the calculation of the effective sectional area when the 500℃ penetration depth (x_{500}) was less than or equal to the sum of the net concrete cover thickness and the reinforcing bar diameter (i.e., $x_{500} \leq c_c + d_b$), the effective area of concrete ($A_{reduced}$) was calculated considering the reduction of the reinforcing bar area. The yield strength and the elastic modulus of the reinforcing bar exposed to the temperature (T), $f_y(T)$ and $E_s(T)$, were adopted from Eurocode 2 (EN 1992-1-2 2004), respectively as follows

$$f_y(T) = K_s(T) \times f_y \tag{4}$$

$$E_s(T) = K_E(T) \times E_s \tag{5}$$

where f_y and E_s are the yield strength and the elastic modulus of the steel reinforcing bar at room temperature, respectively, while $K_s(T)$ and $K_E(T)$ are the reduction factors of the yield strength and the elastic modulus of the steel reinforcing bar exposed to the temperature (T), respectively, as shown in Table 1. To derive the P-M interaction curves, as shown in Fig. 2(b), all the load combinations (P-M) satisfying the force equilibrium condition need to be calculated by fixing the maximum compressive strain (ε_c) of the effective sectional area of concrete at 0.003 and changing the strain at the level of the tensile reinforcing bar (ε_{s3}). Based on this computational procedure, as shown in Fig. 3, the failure envelope curve of all the maximum load combinations (P-M) of the RC column member at each specific fire exposure time can be finally determined. The axial strength without an eccentricity (P_0) and the flexural strength without an axial force (M_0) of the RC column member can be calculated without iterative calculations. The axial strength without an eccentricity (P_0) is calculated, as follows

$$P_0 = 0.85 f_{ck} A_{reduced} + A_s f_y(T_s) \tag{6}$$

where T_s is the temperature of steel reinforcement. The flexural strength (M_0) can be also estimated, assuming that all the reinforcing bars in the cross-section reached their yield strength (f_y), as follows

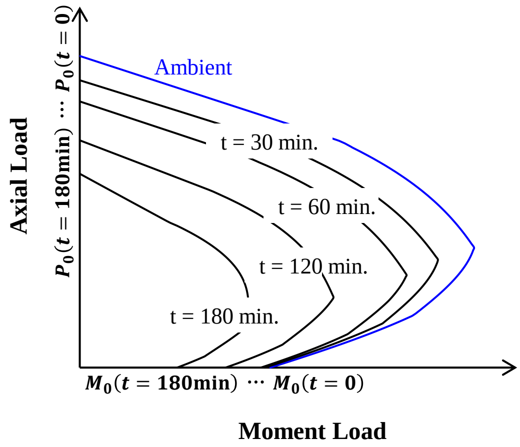


Fig. 3 P-M interaction curves of an RC column section according to fire exposure time

$$M_0(T) = \sum A_s f_y(T_s)\left(d_s - \frac{h}{2}\right) + 0.85 f_{ck} a\left(b - 2x_{500}\right)\left(\frac{h - 2x_{500}}{2} - \frac{a}{2}\right) \tag{7}$$

where f_{ck} is the compressive strength of concrete, A_s is the area of the reinforcing bar, d_s is the depth of the reinforcing bar from the extreme compressive fiber of the RC cross-section, b is the width of the gross section, h is the height of the gross section, and a is the depth of the rectangular concrete compressive stress block.

2.3 Effect of secondary moment (P-δ Effect)

As the exposure time to high temperature increases, the cross-sectional area loss of the RC member increases, and consequently, the effective slenderness ratio (λ_{se}) of the RC column member also gradually increases (Yeo 2012). Fig. 4 shows the effect of fire damages on the effective slenderness ratio (λ_{se}) of RC column members according to the fire exposure time (t), in which the initial effective slenderness ratios (λ_{se}) in Fig. 4 were calculated using the effective cross-sectional area ($A_{reduced}$) estimated by the 500℃ isotherm method. As the fire exposure time (t) increases, the effective slenderness ratio (λ_{se}) shows an increasing trend. Such a tendency appeared more clearly in the small sized sections and in the members with larger initial slenderness ratio at room temperature (λ_s). Thus, the additional secondary effect (i.e., P-δ effect), depending on the effective slenderness ratio (λ_{se}), should be considered in evaluating the fire-resisting performances of RC column members. As shown in Fig. 5(a), when an RC column member is subjected to the combined axial force (P) and flexural moment (Pe) at both ends with a single curvature, the flexural moment (M_{mid}) at the mid-height of the member can be expressed as the sum of the primary moment (M_{1st}) and the secondary moment (M_{2nd}), as follows (Collins and Mitchell 1991, MacGregor *et al.* 1970, and MacGregor and Wight 2005).

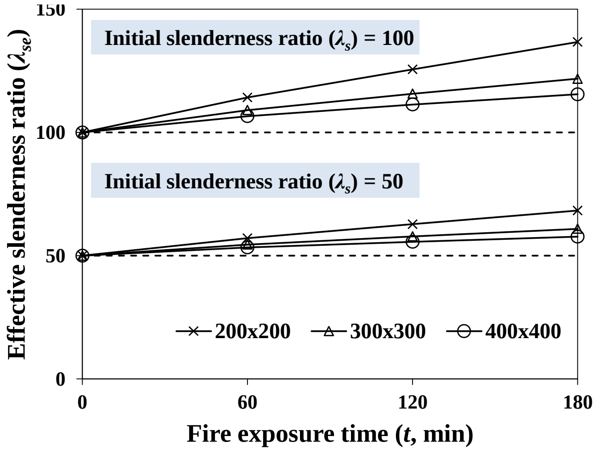
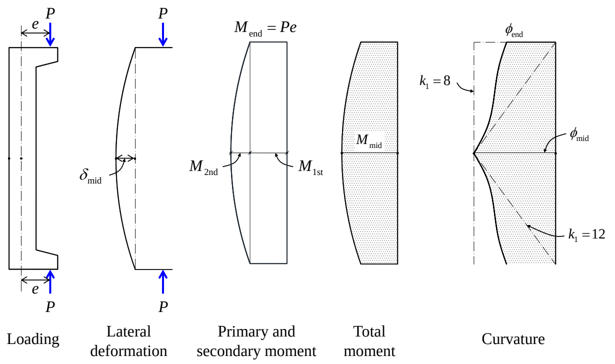
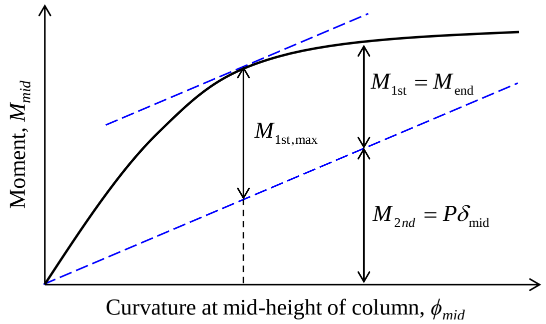


Fig. 4 Effective slenderness ratios of RC columns according to fire exposure time



(a) Moment, curvature, and lateral deformation of an RC column member



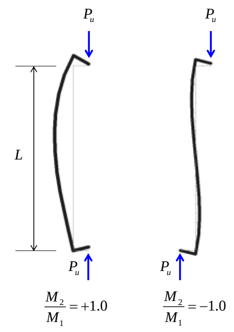
(b) Determination of maximum moment considering the secondary effect

Fig. 5 Primary and secondary moments in a slender RC column

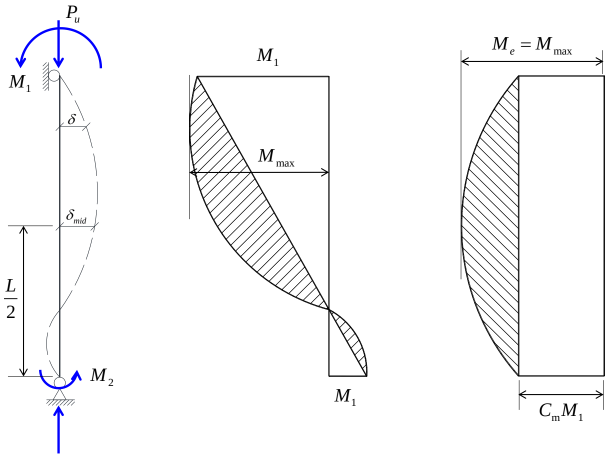
$$M_{mid} = M_{1st} + M_{2nd} = Pe + P\delta_{mid} \tag{8}$$

where P is the axial force acting on the column member, and δ_{mid} is the lateral deformation at the mid-height of the column member. Consequently, as shown in Fig. 5(b), the primary moment (M_{1st}) of the RC column member can be calculated by subtracting the secondary moment ($P\delta_{mid}$) from the moment-curvature response curve (M_{mid}–ϕ_{mid} curve), as follows

$$M_{1st} = M_{mid} - M_{2nd} = M_{mid} - P\delta_{mid} \tag{9}$$



(a) End moment ratio and its sign convention



(b) Equivalent uniform moment factor

Fig. 6 Equivalent uniform moment approach for RC column subjected to unequal end moments (Leite *et al.* 2014)

For the RC column subjected to unequal end moments, the equivalent uniform moment factor (C_m), as recently presented by Leite *et al.* (2014), was adopted in this study, as follows

$$C_m = 1 - \frac{v\lambda_g^2}{600} \geq C_{min} \tag{10}$$

$$C_{min} = 0.6 + 0.4\frac{M_2}{M_1} \geq 0.4 \tag{11}$$

where v is the axial force ratio (P_u/P_0), λ_g is the geometrical slenderness ratio (kL/h), kL is the effective length of the column member, M_1 and M_2 are the larger and smaller values of the end moments, respectively, as shown in Figs. 6(a) and 6(b), and M_2/M_1 has a positive value for the column with a single curvature and a negative value for the one with a double curvature. Then, the equivalent flexural moment (M_e) in the column member subjected to the unequal end moments can be expressed using the equivalent uniform moment factor (C_m), as follows

$$M_e = C_m\left(M_1 + P\delta\right) \tag{12}$$

where the lateral displacement (δ) caused by the column effect can be equivalently replaced by the lateral displacement at the mid-height of the column member (δ_{mid}), as shown in Fig. 6(c). The larger end moments (M_1) can be then calculated, as follows

$$M_1 = \frac{M_e}{C_m} - P\delta_{mid} \tag{13}$$

where δ_{mid} can be calculated, as follows

$$\delta_{mid} = \frac{\varphi_{mid} L^2}{k_1} \tag{14}$$

where ϕ_{mid} is the curvature at the mid-height of the column, L is the length of the column. Also, k_1 is the shape coefficient of the curvature distribution, which is taken to be 8.0 and 12.0 when flexural moments act only at both ends and when lateral forces act between the member ends, respectively, as illustrated in Fig. 5(a). According to Collins and Mitchell (1991), MacGregor *et al.* (1970), and MacGregor and Wight (2005), sufficient analysis accuracy can be obtained when the k_1 value is taken as 10.0. As shown in Fig. 5(b), the flexural strength of the RC column member subjected to the arbitrary axial force (P) can be defined by the maximum value of the primary moment ($M_{1st,max}$), which is calculated by either Eq. (9) or Eq. (13) for the RC column member subjected to equal or unequal end moments, respectively. The moment-curvature analysis of the RC cross-section subjected to the axial force P shall be essentially performed to determine the maximum moment ($M_{1st,max}$), for which this study adopted the flexural behavior analysis model presented in the authors' previous studies (Han *et al.* 2014, Ju *et al.* 2014, Kim and Lee 2011, Kim *et al.* 2011, Kim and Lee 2012a, Kim and Lee 2012b, and Lee and Kim 2011). The equivalent stress block model for concrete presented by Collins and Mitchell (1991) was applied in this study, and the constitutive model of steel reinforcement specified in Eurocode 2 was adopted to consider the effect of temperature, as shown in Table 1.

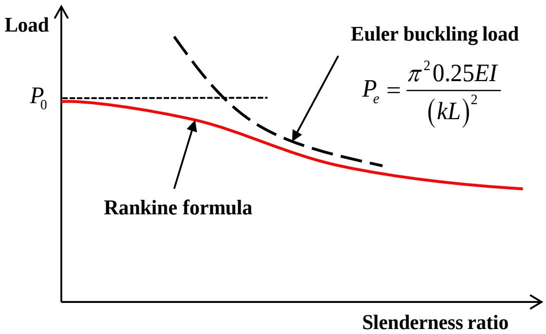
2.4 Rankine load

Sufficient structural safety factors are commonly applied for the design of column members, because there are many uncertainties in practical situations such as unexpected loading, on-site construction errors, etc. Thus, in this study, the maximum axial strength was limited by Rankine load (P_R), as follows

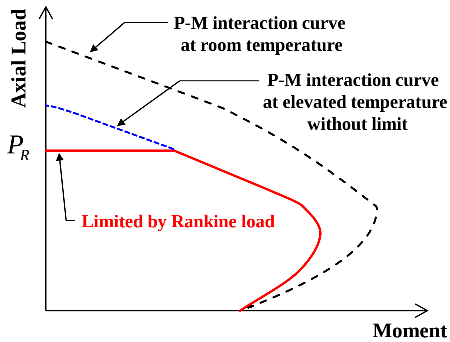
$$\frac{1}{P_R} = \frac{1}{P_o} + \frac{1}{P_e} \tag{15a}$$

$$P_e = \frac{\pi^2 (EI)_{eff}}{(kL)^2} \tag{15b}$$

where kL and $(EI)_{eff}$ are the effective length and the effective flexural stiffness of the column member. As shown in Fig. 7(a), Rankine load (P_R) is the function of the axial strength (P_o) and Euler's buckling load (P_e) of the RC column, and it can be applied to both short and long column members with various slenderness ratios (Rankine 1908 and Tang *et al.* 2001). Recent studies (Tan and Tang 2004) also showed that the Rankine formula could reasonably estimate the strengths of the column members exposed to high temperature. The P-M interaction curve shown in Fig. 7(b)



(a) Description of Rankine load



(b) P-M interaction curve with axial capacity limit

Fig. 7 Maximum limit of axial capacity of RC column based on Rankine formula

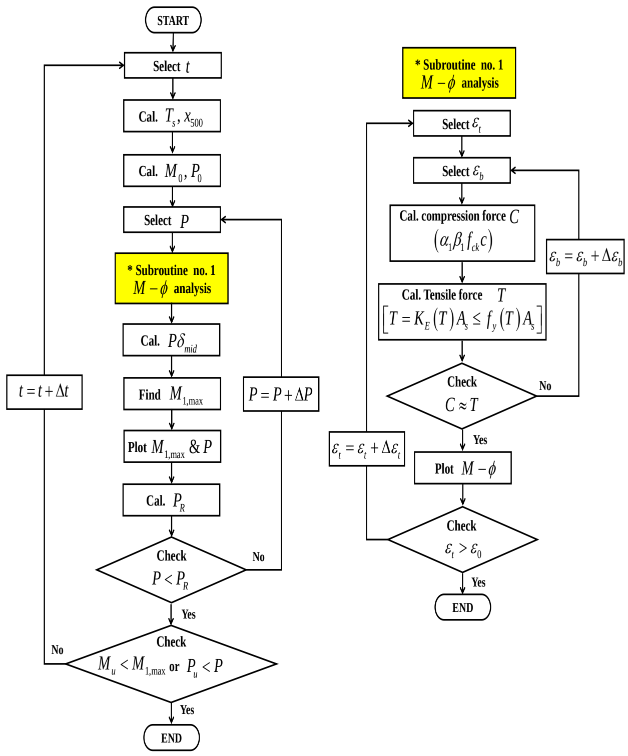
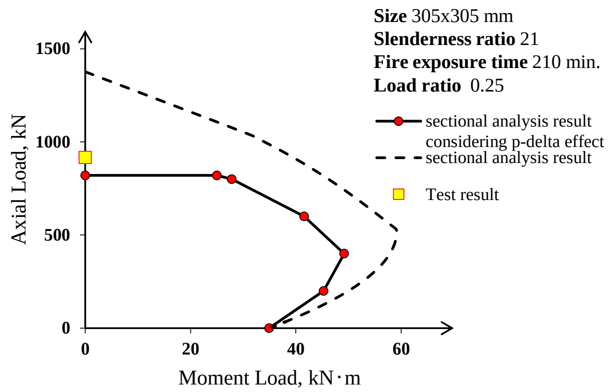


Fig. 8 Computational procedures of the proposed model

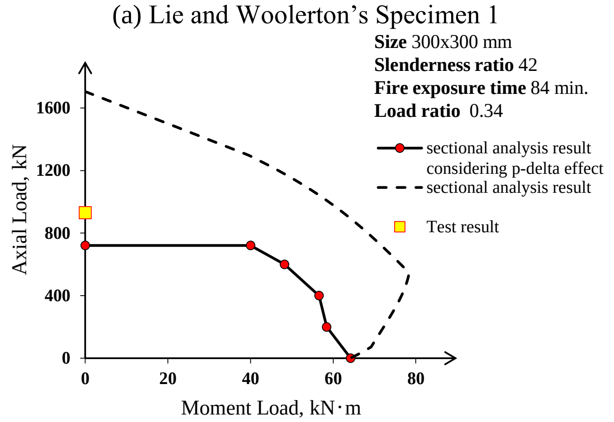
can be then derived by limiting the axial compressive strength of the column member exposed to fire by Rankine load (P_R). Fig. 8 shows the detailed computational procedures of the proposed method. The P-M interaction curve of the RC column with the consideration of the P-δ

Table 2 Dimensions and material properties of specimens collected from literature

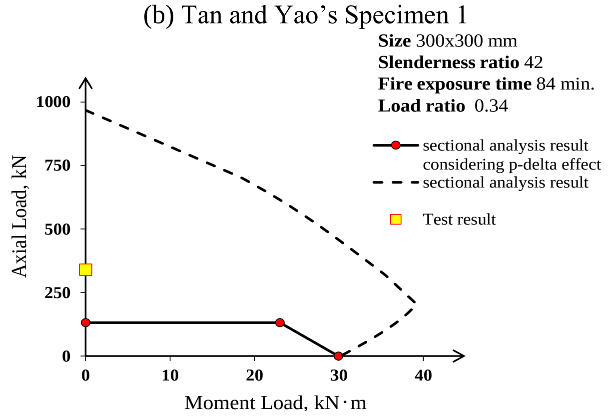
Investigator(s)	Number of specimen	b or h (mm)	ρ_s (%)	L (m)	f_{ck} (MPa)	f_y (MPa)	c_c (mm)	e (mm)	P_{app} (MPa)	t (min)
Yeo (2012)	3	250 to 350	2.5 to 4.95	3	24	400	61	0	1225 to 1837	170 to 210
Tan and Yao (2003)	39	200 to 300	2.1 to 3.1	3.8 to 5.8	24.1 to 42.3	418 to 544	30 to 38	0 to 150	122 to 1695	31 to 160
Lie and Woolerton (1988)	24	203 to 406	2.2 to 4.4	3.8	34.2 to 52.9	414 to 444	48 to 64	0 to 44	169 to 2978	146 to 285



(a) Lie and Woolerton's Specimen 1

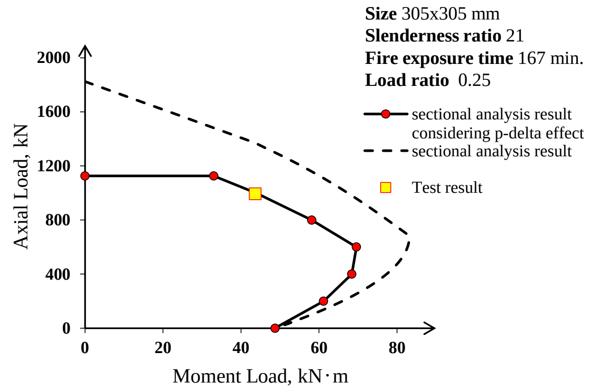


(b) Tan and Yao's Specimen 1

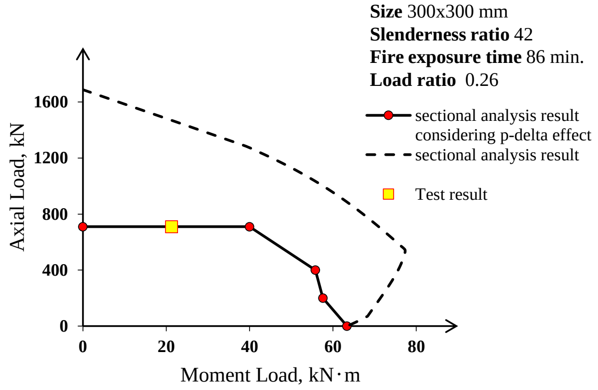


(c) Tan and Yao's Specimen 2

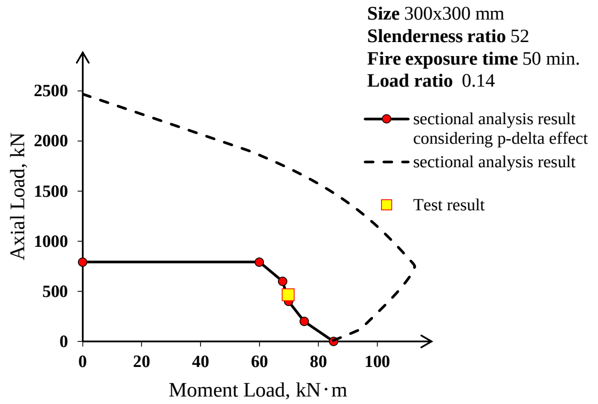
Fig. 9 Test and analysis results of concentrically loaded RC columns



(a) Lie and Woolerton's Specimen 2



(b) Tan and Yao's Specimen 3



(c) Tan and Yao's Specimen 4

Fig. 10 Test and analysis results of eccentrically loaded RC columns

effect can be obtained by following the calculation procedures for any target fire exposed time (t) or design magnitude of axial forces (P).

In addition, the flexural stiffness of RC columns decreases due to flexural cracking, and the deformation increases gradually due to the long-term creep effect of concrete. The elastic modulus also decreases due to the nonlinear behavior of concrete in compression. Thus, it is difficult to estimate the flexural stiffness of the RC members accurately. The current structural concrete design codes (KCI-M-07 2007 and ACI318-11 2011) provide a simple formula for calculating the flexural stiffness of the RC member considering the aforementioned influencing factors, and the effective flexural stiffness of the column members in a non-sway system ($(EI)_{eff}$) is estimated, as follows

$$(EI)_{eff} = \frac{0.4E_cI_g}{1+\beta_d} \tag{16}$$

where β_d is taken as 0.6, and thus $(EI)_{eff}$ becomes $0.25E_cI_g$.

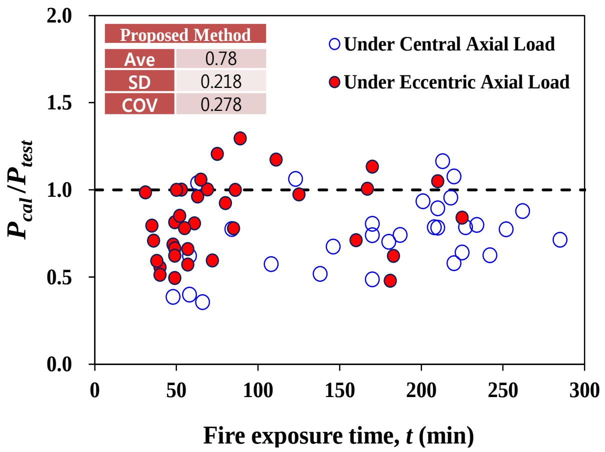


Fig. 11 Verification of the proposed model

Also, E_c is the modulus of the elasticity of concrete, and I_g is the secondary moment of inertia of the concrete section only neglecting the reinforcing bars.

3. Verification

To verify the analysis accuracy of the proposed fire performance evaluation method for RC columns, a total of 66 test results was collected from literature (Yeo 2012, Tan and Yao 2003, and Lie and Woolerton 1988), and their dimensions and material properties are summarized in Table 2. The collected specimens had a wide range of geometrical and material characteristics; the heights (h) of the cross-section ranged from 200 to 406 mm, the reinforcing bar ratios (ρ_s) ranged from 0.89 to 4.95, the member lengths (L) ranged from 2.1 to 5.76 m, the slenderness ratios (kL/r) ranged from 14.3 to 96, the compressive strengths (f_{ck}) of concrete ranged from 24 to 52.9 MPa, the yield strengths (f_y) of reinforcing bars ranged from 400 to 591 MPa, the load ratios (P_u/P_o) at ambient temperature were 0.07 to 0.5, and the fire exposure durations (t) were 31 to 285 minutes. Among the collected specimens, 28 were tested under axial loads only, and 38 were subjected to eccentric axial loads, i.e., axial loads and moments.

Fig. 9 shows the comparisons between the analysis and test results of the collected test specimens under the axial loads only. It is shown that the axial strength limitation based on the Rankine formula provide reasonable estimations on the axial strengths of the test specimens. Fig. 10 shows comparisons of the analysis and test results of the specimens subjected to the eccentric axial loads, and the proposed model provided good estimations close to the test results. In particular, the proposed method accurately estimated the test results of the RC column member with a large eccentricity. Fig. 11 shows the axial strength ratios (P_{cal}/P_{test}) of all the specimens collected in this study according to the fire exposure time (t). It can be found that the fire-resisting performances of the RC columns can be estimated by the proposed model with a consistent accuracy regardless of their slenderness ratio, load ratio, and fire exposure.

4. Conclusions

This study presented a estimation method of the fire-resisting performance for RC columns based on the P-M interaction curve approach. A total of 66 test results was collected from literature, and they were compared to those estimated by the proposed method. From this study, the following conclusions can be drawn:

- The P-M interaction curve model proposed in this study can estimate the axial and flexural strength degradations due to fire damages.
- According to the analysis results, the effective slenderness ratio of the RC columns exposed to fire increased due to the accumulation of fire damages, and the secondary moment effect were well reflected by the proposed P-M interaction curve in a rational manner.
- The axial strength of the RC column was limited by Rankine load in the proposed model to consider the accidental eccentricity and other uncertainties, and the equivalent uniform moment factor was also introduced to consider the influence of the unequal end moments on the behavior of the RC column exposed to fire.
- Compared to 66 test results collected from literature, it was confirmed that the proposed approach is simple but provides accurate analysis results.

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