The current issue and full text archive of this journal is available on Emerald Insight at:
https://www.emerald.com/insight/2040-2317.htm

Member and structural fragility of reinforced concrete structure under fire

Ranjit Kumar Chaudhary and Tathagata Roy
Department of Civil Engineering, Indian Institute of Technology Delhi, New Delhi, India, and
Vasant Matsagar
Indian Institute of Technology Delhi, New Delhi, India

Received 14 February 2019
Revised 22 February 2020
Accepted 14 April 2020

Abstract

Purpose – Despite recognizing the significance of risk-based frameworks in fire safety engineering, the usual approach in structural fire design is largely member/component level, wherein effect of uncertainties influencing the fire resistance of structures are not explicitly considered. In this context, a probabilistic framework is presented to investigate the vulnerability of a reinforced concrete (RC) members and structure under fire loading scenario.

Design/methodology/approach – The RC structures exposed to fire are modeled in a finite element (FE) platform incorporating material and geometric nonlinearity, in which the transient thermo-mechanical analysis is carried out by suitably incorporating the temperature variation of thermal and mechanical properties of both concrete and steel rebar. The stochasticity in the system is considered in structural resistance, thermal and fire model parameters, and the subsequent fragility curves are developed considering threshold limit state of deflection.

Findings – The fire resistance of RC structure is reported to be significantly lower in comparison to the RC members, thereby illustrating the current prescriptive design approaches based on studies of structural member behavior to be crucial from a safety and reliability point of view.

Practical implications – The framework developed for the vulnerability assessment of RC structures under fire hazard through FE analysis can be effectively used to estimate the structural fire resistance for other similar structure to enhance safety and reliability of structures under such extreme threats.

Originality/value – The paper proposes a novel methodology for vulnerability assessment of three-dimensional RC structures under fire hazard through FE analysis and provides comparison of the structural fragility with fragility developed for structural members. Moreover, the research emphasizes to assume 3D behavior of the structure rather than the approximate 2D behavior.

Keywords Vulnerability, Concrete, Fire, Fragility, Thermal, Stochastic

Paper type Research paper

1. Introduction

Despite recognizing the significance of risk-based frameworks in fire safety engineering, the usual approach in structural fire design has been largely at member/component level using traditional prescriptive methodology, wherein effects of uncertainties influencing the fire resistance of structures are not explicitly considered (Zha, 2003; Zhou and Vecchio, 2005; Knox *et al.*, 2008). The measured data obtained from various experimental programs and relevant case studies show existence of significant degree of uncertainty, which consequently affects structural performance under fire loading. The existing methodologies have resulted in practice in which structural reliability under fire is relatively undefined and

Journal of Structural Fire Engineering

2040-2317
DOI 10.1108/JSFE-02-2019-0015

vague (Shi *et al.*, 2013). Moreover, results from empirical models are in complete disagreement with comparatively modern risk-based approaches, indicating the existence of significant uncertainties in the system, which resulted in obtaining the response to be highly sensitive to these uncertain parameters (Van Coile *et al.*, 2013; Huang and Delichatsios, 2010). Considering these aspects, probabilistic approaches are observed to have the potential to provide appropriate understanding for assessing the variabilities and uncertainties to achieve safe and economic design of structures susceptible to catastrophic fire (Khorasani *et al.*, 2015) as is enormously proved in the field of earthquake engineering (Roy and Matsagar, 2017; Saha *et al.*, 2016a). The quantification of structural vulnerability is a critical objective in structural fire engineering as the risk assessment strategies enable designers and key stakeholders to study tradeoffs in utility and cost for determining best engineered solution for a given target performance level (Van Coile *et al.*, 2014). Therefore, to measure the uncertainties associated with the random variables, the development of fragility functions becomes an attractive procedure for assessing the vulnerability of structures under fire (Khorasani *et al.*, 2015; Gernay *et al.*, 2018).

The use of probabilistic frameworks by developing fragility functions has been in use over a significant period to evaluate vulnerability at structural and community level under seismic hazard (Ellingwood and Kinali, 2009; Saha *et al.*, 2016b). In recent years, methods for investigating performance of structures exposed to fire have similarly started to advance in probabilistic domains (Gernay *et al.*, 2016). The probabilistic frameworks are useful as the process involved from development of fire to required mitigation techniques involve a complex approach that includes structural fire resistance, fire mitigation system and protective measures for human life and property. Such complexities allow probabilistic frameworks to identify the importance of uncertainties and variabilities to evaluate the performance of structural systems exposed to catastrophic fire, which is almost inaccessible under the lesser rational deterministic methods (Balaji *et al.*, 2016). Moreover, the fire models in the design guidelines are unable to provide the required confidence level for achieving desired structural performance exposed to fire (ElMohandes and Vecchio, 2016), which necessitated adopting the probabilistic approaches to investigate the fire performance for structural fire safety. Hence, the probabilistic approach provides relatively worthwhile knowledge about the reliability of structural systems, which is a crucial component of more resilient built environment (Khorasani *et al.*, 2015).

Application of the probabilistic concepts to investigate the structural fire safety requires assessing the sources of uncertainties (Khorasani *et al.*, 2016). The primary source of uncertainties is the fire loading itself, which includes distribution, nature and quantity of combustible materials in the compartment, nature of fire development because of the presence of ventilation factor and attained gas temperature. Moreover, the structural responses are also dependent on structural member size, subsequent exposure category (IS 456, 2000; ASCE/SEI/SFPE 29–05, 2006; ACI 216.1–07, 2007), temperature and time-dependent nonlinear thermal and mechanical properties of material and complexity in the structural behavior because of the nature of materials. The extent of uncertainties associated with the structural fire resistance has forced to address structural reliability at member and community level. Initially, probabilistic methodology in fire safety engineering was used to investigate load and resistance factors for designing structures exposed to fire (Ellingwood, 2005). Thereafter, Monte Carlo (MC) simulation and reliability techniques were used to investigate member level performance of RC structures considering uncertainties in structural capacity and fire loading (Sidibé *et al.*, 2000; Shi *et al.*, 2013; Guo *et al.*, 2013; Balogh and Vigh, 2016; Kho *et al.*, 2017). It was concluded that the proposed methodologies provided insight into more practical approach to assess the reliability for more complex structural systems. Moreover, procedures for conducting reliability analysis of RC beams and columns and prestressed concrete beams exposed to fire load were presented (Eamon and Jensen, 2012, 2013a, 2013b). It was concluded that the reliability decreased nonlinearly with respect to time, while the most effective parameters influencing the reliability of RC members were reinforcement cover, load ratio, fire exposure and boundary conditions. Similarly, Ioannou *et al.* (2017) quantified uncertainties for slabs and column in mid-rise RC frame building and subsequently used expert judgement to derive the fire fragility curves for this building. Most of the studies conducted, although limited, to investigate the vulnerability of the structure exposed to fire were merely based on approximate assumptions of the structural model considering lumped mass plasticity (Lange *et al.*, 2014; Jiang and Usmani, 2018). Such assumptions have the chances in overestimating the design of structures under fire because the fire spread in a structure is 3D in nature, which will require appropriate elements for obtaining accurate responses in each direction. The accuracy of the response obtained is compromised significantly for numerical investigations of structures under fire hazard carried out by idealizing the structural members with either one dimensional elements (1-D) or two-dimensional elements (2-D) such as plane stress, plane strain, and axisymmetric elements for reduced computational time and convergence issues because these analyses involve limited set of variables, which are insufficient to represent the actual three-dimensional (3-D) behavior of structures. Obtaining the solution with relatively approximate 2-D method limits the assumptions that may affect the state of the responses. In a nutshell, the literature available till date suggests that most of the studies were conducted to determine the performance of RC structures at member (local) level; however, the performance at structural/system (global) level has not been addressed so far, especially on the probabilistic aspects. As limited research is available to determine the structural fire performance for individual members such as beams and columns, relatively lesser knowledge is available on the use of stochastic approach to investigate the performance of structural frame under fire. Therefore, the advantages of developing probabilistic analysis can be used for evaluating the structural fire performance under a set of given uncertainties to develop performance-based design guidelines for the structures under fire.

In this study, RC members are modeled as 2D and 3D members to compare the effectiveness of modeling strategy and accuracy of the modeling techniques adopted. Moreover, fragility curves are derived for 3D beam and column members, and are subsequently compared with a RC portal frame exposed to fire. The RC members and structure exposed to fire is modeled in a finite element (FE) platform incorporating material and geometric nonlinearity, in which the transient thermo-mechanical analysis is carried out by suitably incorporating the temperature variation of thermal and mechanical properties of concrete and steel rebar. The stochasticity in the system is considered in structural, thermal and fire model parameters, and the subsequent fragility curves are developed based on the threshold limit state of deflections of beams and columns. Therefore, from the above-mentioned gaps, the objectives of the present study are as follows: (i) to compare the responses of the RC beams and columns modeled as 2D and 3D FE members, (ii) to study the variation of the responses of the RC members and structure considering uncertainties in the system parameters and (iii) to develop fragility curves based on failure probability of the structure under fire at member and structural level.

2. Material modeling of reinforced concrete elements

The behavior of RC structure at elevated temperature is mainly governed by the material properties of the structural members. To predict the structural performance under fire, suitable temperature-dependent material models are adopted, which involves modeling of the behavior of concrete and reinforcing steel bars at an elevated temperature.

2.1 Modeling of concrete

As concrete is highly nonlinear and non-homogeneous material, it is difficult to precisely predict the behavior of RC structure, especially at an elevated temperature. For thermo-mechanical analysis of RC structure, it is essential to determine the temperature dependent mechanical and thermal properties of concrete. In this study, the behavior of concrete in compression and tension at elevated temperature is modeled based on the relations proposed by Aslani and Bastami (2011) and Chang *et al.* (2006), respectively. These proposed models are based on the comparison with the experimental results and existing literature, which are observed to be effective in predicting the structural performance under fire:

$$\sigma'_{cT} = \sigma'_{c}\begin{bmatrix} 1.012 - 0.0005T \leq 1.0 & ; & 20°C \leq T \leq 100°C \\ 0.985 + 0.0002T - 2.235\times10^{-6}T^2 + 8\times10^{-10}T^3 & ; & 100°C < T \leq 800°C \\ 0.44 - 0.0004T & ; & 800°C < T \leq 1000°C \\ 0 & ; & T > 1000°C \end{bmatrix} \tag{1}$$

$$\sigma'_{tT} = \sigma'_{t}\begin{bmatrix} 1.05 - 0.0025T \leq 1.0 & ; & 20°C \leq T \leq 100°C \\ 0.8 & ; & 100°C < T \leq 200°C \\ 1.02 - 0.0011T \geq 0 & ; & 200°C < T \leq 800°C \end{bmatrix} \tag{2}$$

where, σ'_c, σ'_{cT} and σ'_t, σ'_{tT} are compressive and tensile strengths at room and elevated temperature *"T"*, respectively. The elastic modulus is a key factor governing the deformation of structural elements, which is either expressed as a function of compressive strength or in the form of empirical relations. The empirical relation proposed by Aslani and Bastami (2011) for estimating reduction in elastic modulus of the concrete at an elevated temperature can be expressed as follows:

$$E_{ciT} = E_{ci}\begin{bmatrix} 1.0 & ; & 20°C \leq T \leq 100°C \\ 1.015 - 0.00154T + 2\times10^{-7}T^2 + 3\times10^{-10}T^3 & ; & 100°C < T \leq 1000°C \\ 0 & ; & T > 1000°C \end{bmatrix} \tag{3}$$

where $E_{ci} = 5000 \checkmark \sigma'_c$ is the elastic modulus of concrete at room temperature. Based on the models of maximum compressive strength and elastic modulus proposed, the uniaxial stress–strain relation depicting the temperature dependent behavior of concrete is given by Aslani and Bastami (2011):

$$\frac{\sigma_{cT}}{\sigma'_{cT}} = \frac{\beta_{mT}\left(\frac{\varepsilon_{cT}{}^{cT}}{\varepsilon_{max}{}^{max}}\right)}{\beta_{mT} - 1 + \left(\frac{\varepsilon_{cT}{}^{cT}}{\varepsilon_{max}{}^{max}}\right)^{\beta_{mT}}} \tag{4}$$

$$\varepsilon_{max} = ^{max}2\sigma'_c/E_{ci} + 0.21\times10^{-4}(T-20) - 0.9\times10^{-8}(T-20)^2 \quad (5)$$

$$\begin{aligned}
\beta_{mT} &= \beta_{mT,a}(\text{fitted}) = \left[1.02 - 1.17\left(E_{pi}/E_{ci}\right)\right]^{-0.74} \quad ; \quad \varepsilon_{cT} \le {}^{cT}\varepsilon_{max}{}^{max} \\
\beta_{mT} &= \beta_{mT,d}(\text{fitted}) = \beta_{mT,a}(\text{fitted}) + (a + 28b) \quad ; \quad \varepsilon_{cT} > {}^{cT}\varepsilon_{max}{}^{max} \\
a &= 2.7\times\left(12.4 - 1.66\times10^{-2}\sigma'_{cT}\right)^{-0.46} \quad ; \quad b = 0.83\exp\left(-911/\sigma'_{cT}\right)
\end{aligned} \quad (6)$$

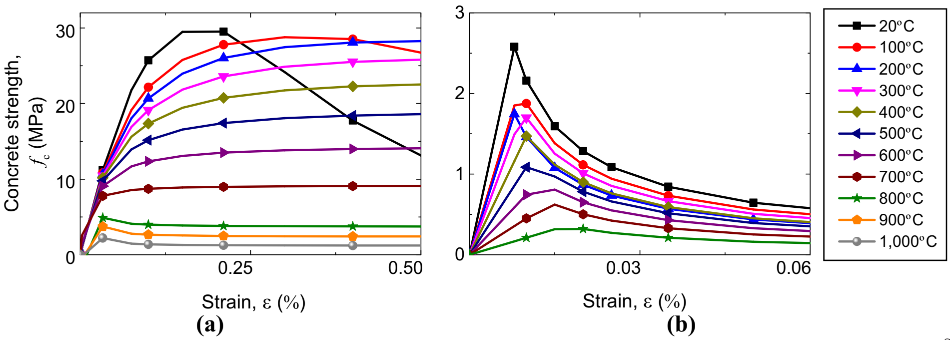
where β_{mT} is shape parameter of the stress–strain curve of concrete; E_{pi} is the secant modulus of concrete at peak stress; E_{ci} is the initial tangent modulus of concrete at ambient temperature; ε_{cT} is the strain at temperature, T; ε_{max} is the maximum strain at peak stress for temperature, T and a and b are curve fitting parameters. It is extremely important to model the transit creep effects for concrete (Gernay and Franssen, 2012); however, for the current study, this transient creep effect is not considered.

Although the tensile strength of concrete is significantly lower (about 10%–15% of compressive strength), it is an essential property influencing the development of cracks under particular load. The cracks developed in the structure have the potential to significantly alter the behavior of the structure, making it important to consider the effect of tension in concrete. In this study, the tensile stress–strain relationship considered can be expressed as follows:

$$\sigma_{tT} = \begin{bmatrix} \varepsilon_{ct}E_{ciT} & ; & \varepsilon_{ct} \le \varepsilon'_{ctT} \\ \sigma'_{tT}\left(\varepsilon'_{ctT}/\varepsilon_{ct}\right)^{0.75} & ; & \varepsilon_{ct} > \varepsilon'_{ctT} \end{bmatrix} \quad (7)$$

where ε'_{ctT} is the tensile strain at the peak stress at temperature, T, and σ'_{tT} is the maximum tensile strain in concrete at temperature, T. Figure 1 represents the uniaxial stress–strain relationship for the concrete of mean strength (30 MPa) at an elevated temperature based on the above relations.

In addition to the mechanical strength parameters, the thermal properties of concrete have a significant effect on the response of structure under fire loading, namely, specific heat



Notes: (a) Compression; (b) tension

Figure 1. Stress–strain relationship of concrete with characteristic strength of 30 MPa at elevated temperature

(c_c), thermal conductivity (λ_c), mass/density (ρ_c) and coefficient of expansion (α_c). As per the recommendations of EN 1991–1-2 (2004), the lower and upper limits for thermal conductivity of concrete are as follows:

$$\lambda_{c,lower} = 1.36 - 0.136(T/100) + 0.0057(T/100)^2; \qquad 20^\circ C \leq T \leq 1200^\circ C \tag{8}$$

$$\lambda_{c,upper} = 2 - 0.2451(T/100) + 0.0107(T/100)^2; \qquad 20^\circ C \leq T \leq 1200^\circ C \tag{9}$$

Similarly, the reduction in mass/density (ρ_c) because of the loss of water from the concrete can be expressed as follows:

$$\begin{aligned}
\rho_c(T) &= \rho_c(20^\circ C) &;\quad & 20^\circ C \leq T \leq 115^\circ C \\
\rho_c(T) &= \rho_c(20^\circ C)\left(1 - 0.02(T-115)/85\right); &;\quad & 115^\circ C \leq T \leq 200^\circ C \\
\rho_c(T) &= \rho_c(20^\circ C)\left(0.98 - 0.03(T-200)/200\right) &;\quad & 200^\circ C \leq T \leq 400^\circ C \\
\rho_c(T) &= \rho_c(20^\circ C)\left(0.98 - 0.03(T-200)/200\right) &;\quad & 400^\circ C \leq T \leq 1200^\circ C
\end{aligned} \tag{10}$$

The coefficients of thermal expansion and heat capacity of concrete are obtained from the studies conducted by Ruan *et al.* (2015), which are expressed as follows:

$$\alpha_c = (0.008T + 6) \times 10^{-6} \quad (\text{in}\ ^\circ C^{-1}) \tag{11}$$

$$c_c = -4\left(\frac{T}{120}\right)^2 + 80\left(\frac{T}{120}\right) + 900 \quad (\text{in Jkg}^{-1\circ}\text{K}^{-1}) \tag{12}$$

2.2 Modeling of steel bars

Because of the significant degradation of strength in steel bars at a high temperature, the thermo-mechanical behavior of RC structure is considerably affected, making it essential to consider the temperature dependent behavior of steel bars for the structural fire design. Hence, in this study, the Eurocode (EN 1992–1-2, 2004) elasto-plastic constitutive model for the reinforcing steel bars at elevated temperature is used, which is given by the following:

$$\begin{aligned}
\sigma(T) &= \varepsilon E_{s,T} &;\quad & \varepsilon \leq \varepsilon_{sp,T} \\
\sigma(T) &= f_{sp,T} - c + (b/a)\left[a^2 - (\varepsilon_{sy,T} - \varepsilon)^2\right]^{0.5} &;\quad & \varepsilon_{sp,T} \leq \varepsilon \leq \varepsilon_{sy,T} \\
\sigma(T) &= f_{sy,T} &;\quad & \varepsilon_{sy,T} \leq \varepsilon \leq \varepsilon_{st,T} \\
\sigma(T) &= f_{sy,T}\left[1 - (\varepsilon - \varepsilon_{st,T})/(\varepsilon_{su,T} - \varepsilon_{st,T})\right] &;\quad & \varepsilon_{st,T} \leq \varepsilon \leq \varepsilon_{su,T} \\
\sigma(T) &= 0 &;\quad & \varepsilon = \varepsilon_{su,T}
\end{aligned} \tag{13}$$

where the parameters a, b and c are as follows:

$$a = \sqrt{(\varepsilon_{sy,T} - \varepsilon_{sp,T})(\varepsilon_{sy,T} - \varepsilon_{sp,T} + c/E_{s,T})} \tag{14}$$

$$b = \sqrt{c(\varepsilon_{\mathrm{sy,T}} - \varepsilon_{\mathrm{sp,T}})E_{\mathrm{s,T}} + c^2} \tag{15}$$

$$c = \frac{\left(f_{\mathrm{sy,T}} - f_{\mathrm{sp,T}}\right)^2}{(\varepsilon_{\mathrm{sy,T}} - \varepsilon_{\mathrm{sp,T}})E_{\mathrm{s,T}} - 2\left(f_{\mathrm{sy,T}} - f_{\mathrm{sp,T}}\right)} \tag{16}$$

where $f_{\mathrm{sp,T}}$ and $f_{\mathrm{sy,T}}$ are the proportional limit and maximum stress level in the bar at temperature, T; $\varepsilon_{\mathrm{sp,T}} = f_{\mathrm{sp,T}}/E_{\mathrm{s,T}}$; $\varepsilon_{\mathrm{sy,T}} = 0.02$; $\varepsilon_{\mathrm{st,T}} = 0.15$ and $\varepsilon_{\mathrm{su,T}} = 0.20$. The above constitutive model represents the stress–strain behavior of reinforcing steel bars in both tension and compression. The reduction factor for the temperature dependent constitutive relationship for the steel bars at an elevated temperature is considered based on guidelines for cold worked steel bars in Eurocode.

The temperature attained by steel reinforcing bars under fire loading primarily depends on its thermal properties, namely, density (ρ_{s}), specific heat (c_{s}), thermal conductivity (λ_{s}) and coefficient of expansion (α_{s}). These parameters of thermal properties for the thermo-mechanical analysis of reinforcing steel bars is estimated based on the relationship proposed for thermal strains in Eurocode (EN 1994–1-2, 2005):.

$$\begin{aligned} &\lambda_{\mathrm{s}}(T) = 54 - 3.33 \times 10^{-2} T \quad ; \quad 20°\mathrm{C} \le T \le 800°\mathrm{C} \\ &\lambda_{\mathrm{s}}(T) = 27.3 \quad ; \quad 800°\mathrm{C} \le T \le 1200°\mathrm{C} \end{aligned} \tag{17}$$

$$\begin{aligned} &c_{\mathrm{s}}(T) = -425 + 0.773T - 0.00169T^2 + 2.22 \times 10^{-6} T^3 \quad ; \quad 20°\mathrm{C} \le T \le 600°\mathrm{C} \\ &c_{\mathrm{s}}(T) = 666 - \left(13002/(T - 738)\right) \quad ; \quad 600°\mathrm{C} \le T \le 735°\mathrm{C} \\ &c_{\mathrm{s}}(T) = 545 - \left(17820/(T - 731)\right) \quad ; \quad 735°\mathrm{C} \le T \le 900°\mathrm{C} \\ &c_{\mathrm{s}}(T) = 650 \quad ; \quad 900°\mathrm{C} \le T \le 1200°\mathrm{C} \end{aligned} \tag{18}$$

The coefficient of the expansion can be estimated from the equations developed for thermal strains in Eurocode, which can be expressed as follows:

$$\begin{aligned} &\varepsilon_{\mathrm{s}}(T) = -2.416 \times 10^{-4} + 1.2 \times 10^{-5} T + 0.4 \times 10^{-8} T^2 \quad ; \quad 20°\mathrm{C} \le T \le 750°\mathrm{C} \\ &\varepsilon_{\mathrm{s}}(T) = 11 \times 10^{-3} \quad ; \quad 750°\mathrm{C} \le T \le 860°\mathrm{C} \\ &\varepsilon_{\mathrm{s}}(T) = -6.2 \times 10^{-3} + 2 \times 10^{-5} T \quad ; \quad 860°\mathrm{C} \le T \le 1200°\mathrm{C} \end{aligned} \tag{19}$$

3. Modeling of fire loading on reinforced concrete structures

The fire developed in a compartment is generally idealized through different time-temperature curves. There are different time-temperature curves available, which are used for research such as ISO 834 (2014), hydrocarbon fire curve and external fire curve; however, these idealized fire curves are incapable of completely representing the realistic behavior of fire. In this regard, parametric fire curve proposed in "Appendix A" of the Eurocode 1 (EN 1991–1-2, 2002) is found to be suitable to because this model has the provision for the type of structural materials and occupancy levels in a compartment. Moreover, this model is helpful to accommodate desired opening factor, thermal inertia and fire load density available, which can further be used as uncertain parameters for the probabilistic assessment under fire loading. In this regard, the heating phase of the parametric fire curve is as follows:

$$T = 20 + 1325\left(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}\right) \tag{20}$$

where $t^* = t\Gamma$ is a parameter controlled by ventilation and properties of materials of enclosures with $\Gamma = (O/b)^2/(0.04/1160)^2$, in which O is opening factor and b is thermal inertia. The thermal inertia (b) depends on density, specific heat and thermal conductivity of the enclosure in a compartment.

The duration of heating phase of fire may either be ventilation-controlled or fuel-controlled, which is considered as a maximum of ($0.2 \times 10^{-3} \times q/O$) or 15 min, where "$q$" refers to the design fire load density for the structure. In this regard, 15 min refers to minimum duration if there is no ventilation; however, if the fire is fuel-controlled depending only on the type of fire for heating phase duration, minimum value of opening factor as prescribed in the code needs to be adopted. The decay phase of the fire is further given as follows:

$$\begin{aligned} T &= T_{\max} - 625\left(t^* - t^*_{\max}x\right) && ; \quad t^*_{\max} \leq 0.5 \\ T &= T_{\max} - 250\left(3 - t^*_{\max}\right)\left(t^* - t^*_{\max}x\right) && ; \quad 0.5 \leq t^*_{\max} \leq 2 \\ T &= T_{\max} - 250\left(t^* - t^*_{\max}x\right) && ; \quad t^*_{\max} \geq 2 \end{aligned} \tag{21}$$

where $t^*_{\max} = t_{\max}\,\Gamma$ and x is represented as follows:

$$\begin{aligned} x &= 1 && ; \quad t_{\max} > t_{\lim} \\ x &= t_{\lim}.\Gamma/t^*_{\max} && ; \quad t_{\max} = t_{\lim} \end{aligned} \tag{22}$$

The developed parametric fire curve is helpful to simulate the realistic fire developed within the compartment after assuming homogeneous temperature distribution. However, the parametric fire curve developed is limited to the fire compartments up to 500 m^2 of floor area with a maximum compartment height of 4 m and having no openings in the roof.

4. Thermo-mechanical analysis of three-dimensional reinforced concrete structures

The current study aims to investigate the behavior of the RC structures under fire loading, which involves mechanical and thermal loading acting on the structure simultaneously. Because the strength of concrete and steel bars is considerably reduced at a high temperature, the response of the structure at an elevated temperature compared to that at room temperature is substantially increased. The fire loading in the RC structure is primarily attributed to the heat energy generated from convectional currents of the burning fluid in the chamber and heat radiation emitted from combustion of available fuel components. The total heat flux imposed on the fire exposed surface in the compartment comprises net heat flux because of convection and radiation, which can be written as $Q_{net} = Q_{net,c} + Q_{net,r}$. The thermal gradient developed between the unexposed face and the internal elements leads to the temperature distribution in the RC structure. The temperature distribution in the RC structure primarily depends on thermal conductivity (λ) and specific heat capacity (c) of concrete and steel bars. The present study adopts FE analysis procedure for predicting the performance of RC structure at an elevated temperature, which is carried out in a commercial FE software package, ABAQUS®.

The FE model of the RC structure is developed using the 3D part available in the software using the extrusion feature. The concrete is highly nonlinear and brittle material, which is effectively modeled using the concrete damaged plasticity (CDP) model. The CDP model is a plasticity continuum damage model of concrete, capable of capturing the material nonlinearity and multi-axial behavior for temperature or strain-rate dependency.

The main failure mode of concrete is either by compressive crushing or tensile cracking of concrete, which can be easily predicted through the nonlinear CDP model. The CDP model is a smeared crack approach in which individual micro-cracks cannot be tracked and constitutive calculations are independently performed at each integration point of FE model. For the RC structure, the CDP model can be used with reinforcing bars to model reinforcements. The reinforcing steel bars being elasto-plastic ductile material are modeled using the classical theory of plasticity, which uses von Mises yield criterion with associated plastic flow and isotropic hardening. The reinforcing bars are modeled as 1-D truss element smeared in the concrete region, which transmits only axial force. This model similarly has a provision of defining the temperature sensitivity of steel bars to predict its behavior under thermo-mechanical loading. The bonding between the concrete and reinforcing bars is achieved through a TIE constraint, which is capable of transmitting the forces or reactions, deformations and temperature between the structural elements. As shear failure is not a governing criteria of failure for RC structure under fire loading, the contribution of secondary reinforcements (stirrups and lateral ties) towards the fire resistance is ignored in the present study.

The behavior of the RC structure under fire can be assessed through coupled-temperature displacement analysis, in which the thermal and mechanical responses are simultaneously estimated. The thermo-mechanical analysis is a two-step procedure, in which the response of the structure under mechanical loading at room temperature is determined first, followed by assessment of the behavior under thermal loading. The mechanical loading on the structure primarily comprises dead and live loads, which are applied to the RC structure in the form of distributed loading. During mechanical loading, the structure is predefined to a room temperature of 20°C, which gradually increases because of the heat flux gained by the fire exposed surface on exposure of fire loading. The fire-exposed surface of the RC structure is subjected to thermal loading in the form of convection and radiation with temperature variation in the compartment simulated via a natural fire curve. For thermo-mechanical analysis of RC structures, the concrete is discretized into 3-D brick C3D8RT elements used for heat transfer and mechanical stress analysis with reduced integration points to reduce computation costs. Similarly, the reinforcing steel bars are discretized using T3D2T elements, which is a three dimensional 2-noded truss element for coupled-temperature displacement analysis. The nonlinear temperature-dependent material properties discussed in the preceding sections simulates the material nonlinearity of RC structure. Finally, the geometrical nonlinearity of the RC structure is considered using the NLGEOM option available in the software.

5. Probabilistic assessment

Most of the traditional design approaches are based on prescriptive strategies as a design guideline for structures subjected to fire. However, the fire resistance estimated for a structure is uncertain depending on the number of random parameters such as fire exposure scenario, geometrical configurations and material property. In this regard, the probabilistic approaches have been found to be effective in incorporating all such uncertain parameters and are helpful in decision-making for safe and reliable structural fire design. In this study, the probabilistic assessment is performed using the Pacific Earthquake Engineering Research Center framework for stochastic assessment of structures under seismic hazard (Lange *et al.*, 2014), which is given by:

$$F(DV) = \iiint p(DV|DM)p(DM|EDP)p(EDP|IM)p(IM)d(IM)d(EDP)d(DM) \tag{23}$$

where *IM* refers to intensity measure (such as fire load density and duration of fire loading); *EDP* refers to engineering demand parameters (such as stresses, deflections, temperature and deformation rate); *DM* refers to damage measure (such as slight damage, moderate damage and extensive damage) and *DV* refers to decision variables (such as casualty rate and downtime). The framework proposed for seismic hazard can be used for the probabilistic assessment of RC structure under fire hazard, considering respective analogies. However, the current study intends to develop the fragility function for RC structures under a fire hazard, which can be used for decision-making based on performance levels. The vulnerability of the structure exposed to fire hazard is modified as follows:

$$F(DM) = \iint p(DM|EDP)p(EDP|IM)p(IM)d(IM)d(EDP) \tag{24}$$

In equation (24), the damage measure depends on the capacity of structure (*C*) and demand of the structure, defined in terms of intensity measure (*IM*). Thus, the probability of failure (p_f) based on the capacity of RC structure for a particular *IM* parameter is as follows:

$$p_f = P(D \geq C|IM) \tag{25}$$

The probabilities of failure determined at different *IM* levels using equation (25) are fitted to develop fragility function by optimizing maximum likelihood function (MLE) (Shinozuka *et al.*, 2000).

The estimation of structural capacity is the first step towards deriving fragility function for structures under fire hazard. The structural resistance against a particular hazard primarily depends on the intensity of hazard and structural behavior, which can be defined in terms of moment capacity, maximum temperature and threshold deflections. Usually, the structural capacity is predicted by performing analytical or experimental investigations. In this study, the empirical relations proposed by ISO 834 (2014) are considered as limiting criteria to predict structural failure under fire hazard. For flexural members such as beams, the limiting criteria based on structural deflection are as follows:

$$\begin{gathered} \delta_b = \frac{L^2}{400d} \quad (\text{in mm}) \\ \frac{d\delta_b}{dt} = \frac{L^2}{9000d} \quad (\text{in mm/min}) \end{gathered} \tag{26}$$

where δ_b is the allowable vertical deflection; $d\delta_b/dt$ is the limiting rate of vertical deflection.; L is the length of the clear span of the test specimen; d is the distance between extreme fiber of compression and tension zone. Similarly, the failure criteria for the axially loaded structural members such as columns is as follows:

$$\begin{aligned} \delta_c &= \frac{h}{100} \quad (\text{in mm}) \\ \frac{d\delta_c}{dt} &= \frac{3h}{1000} \quad (\text{in mm/min}) \end{aligned} \tag{27}$$

where δ_c is axial contraction in columns and $d\delta_c/dt$ is the limiting rate of axial contraction. Subsequently, the *IM* parameters for analysis of structures under fire hazard are considered in terms of fire load density (q) and duration of fire (t). Based on these prescribed failure limits, the probability of failure of the structural member at different *IM* levels is estimated.

6. Validation of modeling strategy

The validation of the FE modeling strategy and thermo-mechanical solution procedure adopted is performed by comparing the response of the RC structural member, namely, beams and columns under thermo-mechanical loading with that of the of the result available in the literature. In this regard, experimental studies conducted by Rafi *et al.* (2007) and Kodur *et al.* (2003) on RC beams and columns is considered. In this study, the 3D FE modeling of structures over 2D is deliberately adopted for thermo-mechanical analysis to incorporate 3D thermal strains developed in the structure and precisely predict the structural fire performance. Here, the 3D element used is an eight-node thermally coupled brick element having trilinear displacement and temperature degrees of freedom (DOF), with reduced integration and hourglass control, whereas the 2-D element used is a four-node bilinear element having displacement and temperature DOF with reduced integration and hourglass control.

To validate the behavior of RC structural member at an elevated temperature under mechanical loading, FE models (2-D and 3-D) of RC beam are developed in accordance with Rafi *et al.* (2007). The beam considered of 2 m clear span for validation has rectangular cross-sections of size 120 mm × 200 mm and is simply-supported at its ends. The RC beam is reinforced with 4-longitudinal steel bars, 2–8 mm diameter (ϕ) at the top and 2–10ϕ at the bottom, as main reinforcement and stirrups of 6ϕ diameter spaced at 100 mm c/c as lateral reinforcement. The mechanical loading applied on the beam is in the form of two concentrated loads of 15.5 kN each at 1/3rd of the length of beam, L from both the sides, while the fire loading on the beam is governed by ISO 834 (2014) fire curve at soffit of the beam and the adjacent sides of it. Figure 2 shows the comparison of the response obtained through the modeling strategy adopted in this study, which is in agreement with that obtained by Rafi *et al.* (2007). However, the result obtained using the 2D FE modeling approach shows significant deviation from actual results, thereby illustrating the inefficiency of simulating the actual behavior of structure under thermo-mechanical loading.

Similarly, the response of axially loaded RC structural member under fire loading is validated with the results obtained by Kodur *et al.* (2003). The RC column is subjected to an axial load of 930 kN at the upper surface with both the ends of the column fixed and fire loading applied in the form of time-temperature curve at all four sides based on the experimental study conducted for the furnace. The length of the column adopted in experimental study was 3.81 m, with cross-sections of size 305 mm × 305 mm and reinforced with 4-main steel bars of 25ϕ. The mechanical properties and thermal properties considered for validation is based on the experimental study conducted by Kodur *et al.* (2003). The thermo-mechanical sequential analysis is considered for the FE analysis of RC column under

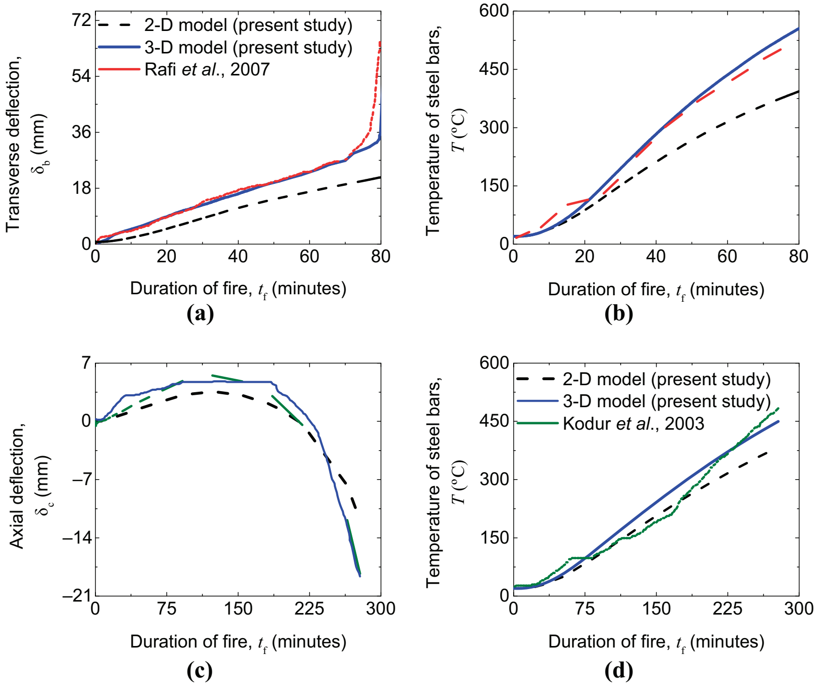


Figure 2.
Comparison of responses of 2-D and 3-D model of RC beam and column with results of experimental studies in literature

thermo-mechanical loading. Figure 2 shows the comparison of the axial contraction at the center of the column and temperature in the main reinforcing bars obtained for the 2D and 3D models of the present study with that obtained by Kodur *et al.* (2003). The maximum axial contraction observed in the 3D FE model of RC column is 18.32 mm, which is close to about 98% obtained in the case of the experimental study. However, the response obtained in adopting the 2-D FE modeling approach is approximately 90% of the result obtained in the literature. Thus, the 3-D FE modeling approach adopted in the current study can be effectively used for simulating the behavior of axially loaded members under fire loading.

The deviation in the response obtained for the 2D modeling approach compared to the 3D model is primarily attributed to the volumetric strains, which is developed in the RC structure for 3-D models. Such bilinear or planar elements used in the 2D model cannot capture the induced volumetric strain, which otherwise neglects the Poisson effect in the 2D model. Figure 3 shows the comparison of maximum tensile strains developed in the 2D and 3D models of RC beam and column developed for validation of FE modeling strategy. The strains developed for the 3D models of RC beam and column is observed to be significantly higher than the corresponding 2D models. Thus, 3D modeling approach is recommended based on this observation for investigating the behavior of RC structures under thermo-mechanical loading.

7. Numerical study

A three-storey residential building (story height, h = 3.3 m) as shown in Figure 4 with three bays in either direction (bay length, L = 5 m) is considered in the current study to assess the performance of RC structure under fire hazard. The mechanical loading on the structure is

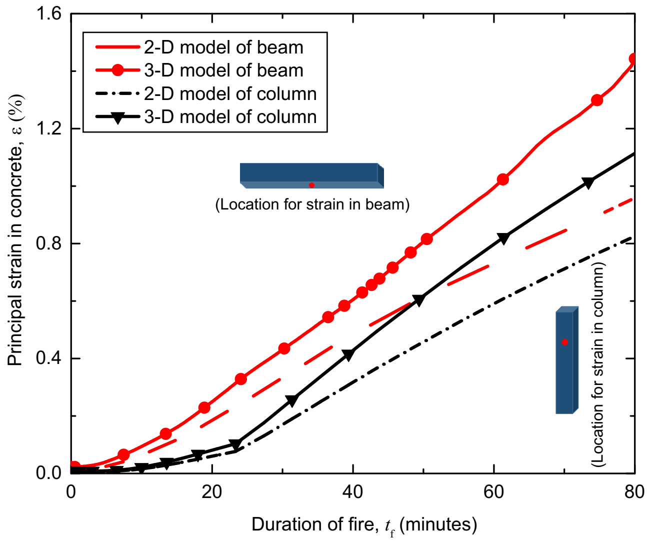


Figure 3. Comparison of principal strain developed in RC beam and column for 2-D and 3-D models

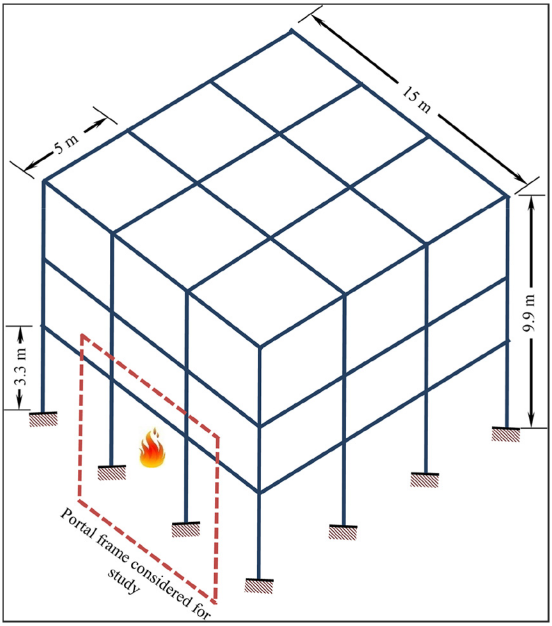


Figure 4. Residential building considered for the study of RC structure under fire hazard

estimated based on the code provisions of IS 875 - Part 2 (1987). The residential building is designed by adopting the limit state method, considering dead loads, imposed loads and seismic load acting on the structure. However, the structural design does not consider the accidental load because of fire hazard. The current study involves investigation of the RC members and structure under fire loading through a nonlinear 3D FE analysis. In this regard, most of the studies predict the behavior of such structures by simplifying the larger complex structure into simpler structural members such as beams, columns and slabs. However, the behavior of the structure obtained through such approaches is different from actual performance. Thus, the current study is aimed to assess this discrepancy in the actual behavior of RC structure by comparing the structural fragility of the RC portal frame with the member fragility of RC beams and columns.

The numerical study is carried out on a portal frame from the RC residential building described above, which has beams and columns of cross-sections 300 mm $\times$ 300 mm and span length of 5 m and 3.3 m, respectively. Figure 5 shows the geometrical details of the beam and column of RC portal frame considered for the study. The axially loaded column of RC portal frame consists of 8-steel bars of 25ϕ as main reinforcing bars and lateral ties of 10ϕ at spacing of 150 mm c/c as lateral reinforcement. Similarly, the beam consists of 3 main reinforcing bars of 25ϕ at top and bottom both with stirrups of 8ϕ spaced at 100 mm

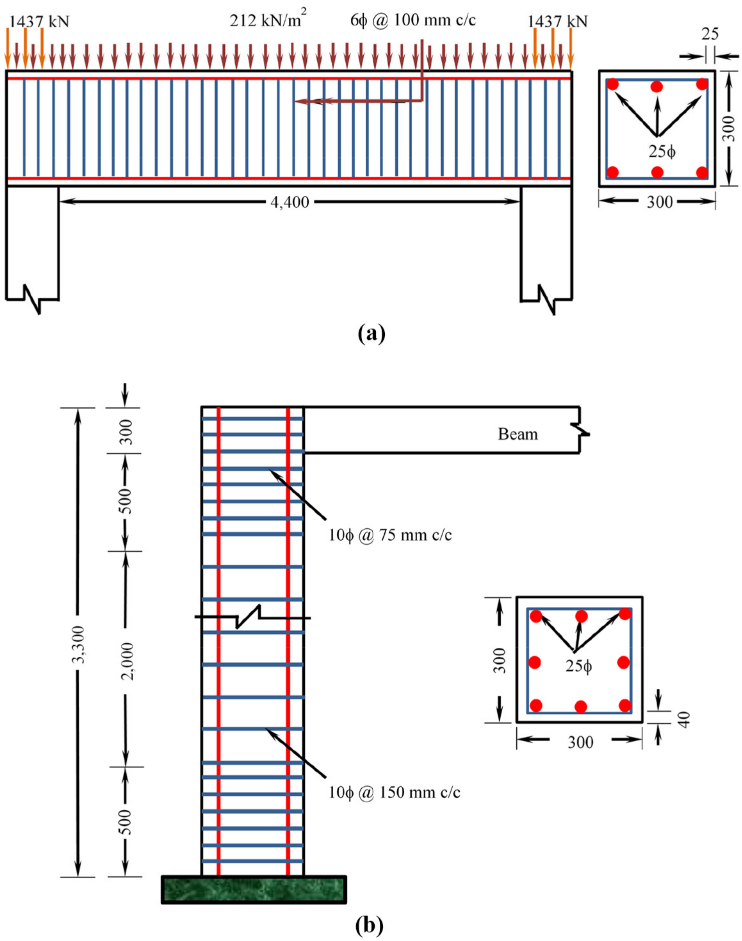


Figure 5.
Details of the reinforcement in (a) beam and (b) column of RC portal frame

c/c. The column is subjected to an axial load of 1437 kN, which comprises of all the loads from upper storey, while beam is under uniformly distributed mechanical loading of 212 kN/mm^2. The axial load in RC column members is assumed to be applied at an eccentricity of 20 mm away from the face exposed to fire based on the minimum eccentricity criteria considered during the design of RC structures (IS: 456, 2000). The RC portal frame is assumed to be fixed at its supports for column, thereby restricting all the DOF. In a similar way, the study for beams is performed considering both fixed and simply supported ends, while the column is assumed to be fixed at both of the ends. Since the portal frame, part of the RC structure, has lateral restraints at its ends, the restraint effect is neglected under the effect of thermo-mechanical loading because the study intends to mainly focus on difference in actual behavior of structure and member under fire loading. The analysis of the RC portal frame under the combined effect of thermal and mechanical loading is carried out independent of the whole structure in which the building is considered only for calculating dead and live loads acting on the portal frame. For thermal boundary conditions, the fire loading is applied on the structure because of the heat fluxes from conduction and radiation. The portal frame under compartment fire is exposed to thermal loading at its internal faces, primarily interior faces of columns and soffit of beam along with its adjacent sides. Thus, the beams and columns of the RC portal frame are subjected to three-side and single-side fire loading, respectively. However, while investigating the RC beam and column members, theoretical scenarios are assumed having single-side and three-side fire exposure for beams, and single-side and four-side fire exposure for columns. Thus, the cases in the present study are grouped as follows:

- beam with simply-supported ends and three-sides fire exposure (BSS-3F);
- beam with simply-supported ends and single-side fire exposure (BSS-1F);
- beam with fixed ends and three-sides fire exposure (BF-3F);
- beam with fixed ends and single-side fire exposure (BF-1F);
- column with fixed ends and four side fire exposure (CF-4F);
- column with fixed ends and single-side fire exposure (CF-1F);
- beam of portal frame with three-sides fire exposure (PFB-3F); and
- column of portal frame with single-side fire exposure (PFC-1F).

Figure 6 illustrates the possible end support and fire exposure conditions of RC beam and column for study of behavior of RC structure under thermo-mechanical loading.

Initially, the study of RC members and structure under thermo-mechanical loading is carried out at a deterministic level. The deterministic study is carried out considering concrete of mean strength of 30 MPa (f_{ck}) and reinforcing steel bars of yield strength of 500 MPa (f_y), which are modeled using CDP and classical plasticity model, respectively. The parameters of CDP model adopted for the study are shown in Table 1. The mean values of parameters for mechanical and thermal properties of steel and reinforcing bars at room temperature are shown in Table 2 and 3, which are adopted for the deterministic study. The fire load for the RC structure of 75 m^2 of total floor is modeled in the form of parametric fire curve prescribed in the Eurocode 1 (EN 1991–1-2, 2002). The thermal inertia influencing the fire curve is estimated as 1800 $J/m^2s^{1/2}K$ for normal weight concrete. Similarly, the fire loading in residential building is adopted as 780 MJ/m^2 based on Eurocode 1. Similarly, the opening factor for the compartment is assumed as 0.05 $m^{1/2}$ based on the available air ventilation because of doors and windows.

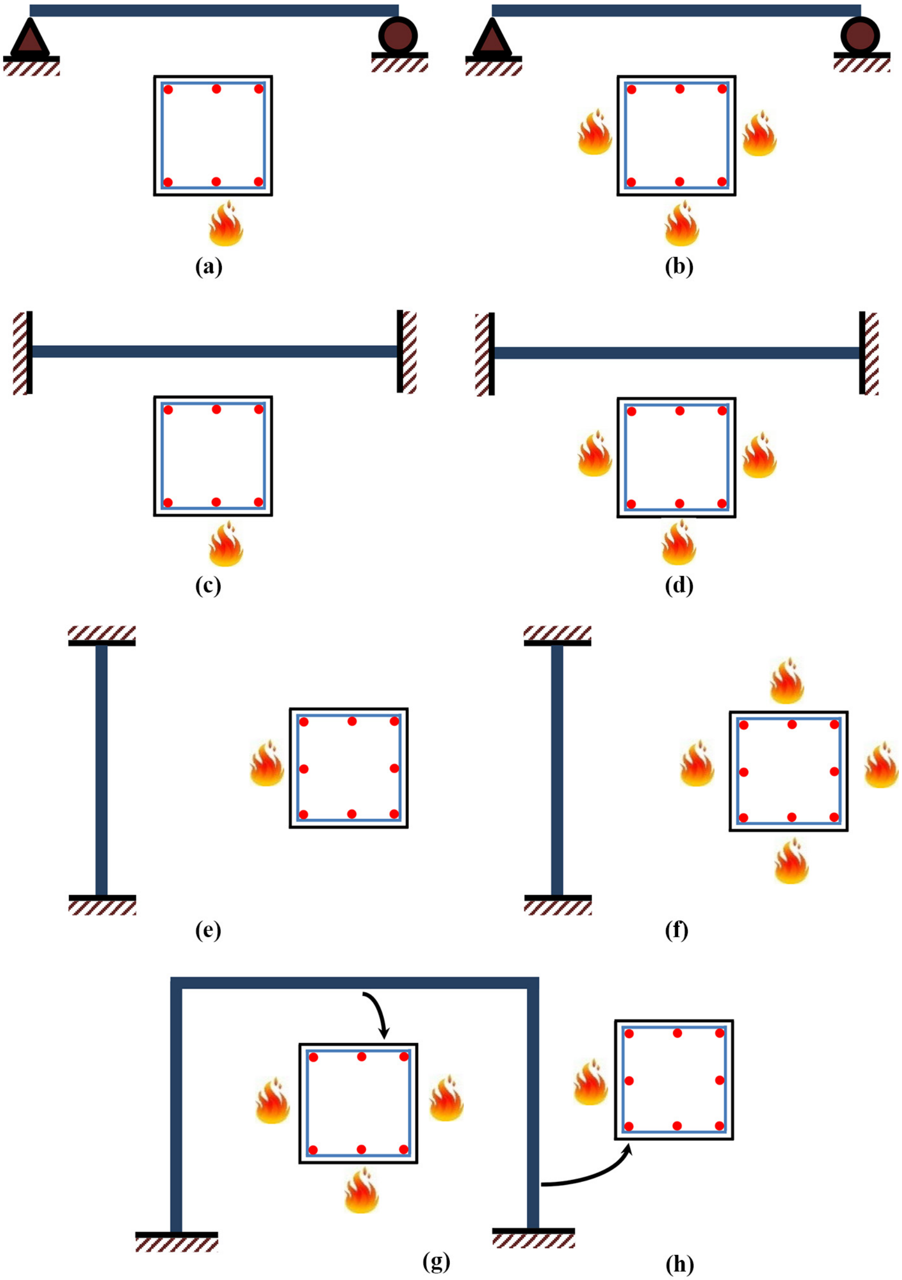


Figure 6.
Cases considered for study based on the end support conditions and number of sides of fire exposure of RC structure

Notes: (a) BSS-1F; (b) BSS-3F; (c) BF-1F; (d) BF-3F; (e) CF-1F; (f) CF-4F; (g) PFB-3F; (h) PFC-1F

The probabilistic study is carried out to consider the effect of various uncertain parameters on the performance of RC members and structure to enhance the safety and reliability of the structure. The probabilistic investigation is carried out using PYTHON scripting based on MC simulations for 1000 realizations. Figure 7 represents the methodology adopted for the probabilistic study adopted in the current study for vulnerability of assessment of structures under fire hazard. The stochastic parameters for the probabilistic analysis are shown in Tables 2–4, where the randomness is primarily considered in the material properties, heat transfer, mechanical and thermal loading parameters. Moreover, the distribution of these parameters are assumed based on literature. In the present study, fire load density (q) is considered as the IM measure parameters, whereas deflections (δ_b and δ_c) and their rate of change ($d\delta_b/dt$, and $d\delta_c/dt$) are considered as the EDP to determine the probability of failure of RC structure under thermo-mechanical loading.

8. Results and discussions

In this study, the behavior of RC members and structure, namely, beams, columns, and portal frame under the thermo-mechanical loading is investigated through 3D FE analysis considering all the associated material and geometrical nonlinearity. Parametric study is

Table 1. Parameters of concrete damaged plasticity (CDP) model

Dilation angle (ψ)	Eccentricity (e)	f_{b0}/f_{co}	K_c	Viscosity (μ_e)
36	0.1	1.16	0.667	0

Table 2. Stochastic parameters for mechanical and thermal properties of concrete

	Parameters	Unit	Distribution	Mean values	COV
Mechanical properties	Strength of concrete (f_{ck})	MPa	Lognormal	30	0.07 (Van Coile and Bisby, 2017)
	Elastic modulus of concrete (E_c)		–	27386.12	–
Thermal properties	Density (ρ_c)	kg/m^3	Deterministic	2300	–
	Conductivity (k_c)	W/(m °C)	Normal	1.95	0.1 (Iqbal and Harichandran, 2010)
	Specific heat (c_c)	J/(kg K)		901	
	Coefficient of expansion (α_c)	/°C		9.0×10^{-6}	

Table 3. Stochastic parameters for mechanical and thermal properties of steel bars

	Parameters	Unit	Distribution	Mean values	COV
Mechanical properties	Strength of steel (f_y)	MPa	Lognormal	500	0.07 (Van Coile and Bisby, 2017)
	Elastic modulus of steel (E_s)		–	2×10^5	–
Thermal properties	Density (ρ_s)	kg/m^3	Deterministic	7850	–
	Conductivity (k_s)	W/(m °C)	Normal	52.73	0.1 (Iqbal and Harichandran, 2010)
	Specific heat (c_s)	J/(kg °C)		439.80	
	Coefficient of expansion (α_s)	/°C		11×10^{-6}	

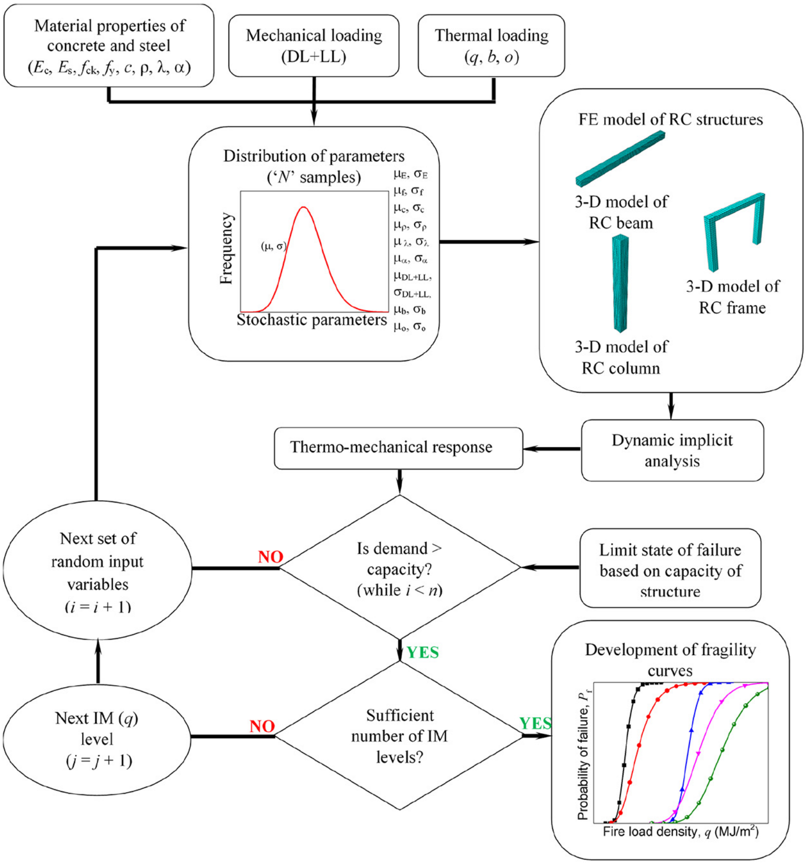


Figure 7.
Stochastic framework to quantify the vulnerability of the RC structure under fire hazard

Table 4.
Stochastic parameters for mechanical and thermal loading on RC structure

	Parameters	Unit	Distribution	Mean values	COV
Mechanical loading	Load on beam surface	kN/m^2	Normal	212	0.1 (Iqbal and Harichandran, 2010)
	Axial load on column	kN		1437	
Fire loading	Thermal inertia (b)	J m^{-2}°C^{-1}s$^{-1/2}$	Normal	1800	0.1 (Iqbal and Harichandran, 2010)
	Fire load density (q)	MJ/m^2	Deterministic	400–1000	-
	Duration of fire loading (t_f)	hour		3 hours	
	Opening factor (O)	m$^{1/2}$	Normal	0.075	0.1 (Iqbal and Harichandran, 2010)
	Fire growth (t_{lim})	min	Deterministic	Slow (25)	–

conducted for the considered RC members and structure for the different end support conditions and fire exposure scenario.

Figure 8 shows the deflection profile and temperature distribution in the RC members and structure under fire loading. The maximum transverse deflection (δ_b) observed for the RC members and structure are 295.43, 185.60, 99.26, 27.78 and 159 mm for BSS-3F, BSS-1F, BF-3F, BS-1F, and PFB-3 F, respectively. Based on the failure criteria for flexural members under fire, the permissible value for maximum transverse deflection and its rate of change are 208.33 mm and 9.26 mm/min, respectively. Accordingly, the fire resistance for the BSS-3F and BF-3F are 0.7 h (hours) and >3 h for natural fire curve, respectively, which shows a significant decrease of >76% in fire resistance of beams with simply-supported ends. However, the beam of the RC portal frame shows a deflection of 159 mm at 1.72 h, time where there is sudden significant increase in deflection and strain, leading to the termination of program. The beam of RC portal frame thus shows considerable decrease in fire resistance (~60%) to natural fire curve compared to fixed beams with three-side fire exposure. In Figure 8(a), it is observed that the effect of number of sides of fire exposure on beams with simply supported ends is negligible because the fire resistance of BSS-1F and BSS-3F, respectively, are 0.75 h and 0.70 h. Similarly, transverse displacement observed for RC columns are 62.78 mm, 14.84 mm, 25.1 mm for CF-4F, CF-1F and PFC-1F, respectively, as shown in Figure 8(b). In this regard, the allowable maximum deflection and deflection rate of axially loaded member are 30 mm and 9 mm/min, respectively. Based on this estimated structural capacity of RC columns, the fire resistance of CF-4F, CF-1F and PFC-1F are found to be 1.69-h, >3-h and 1.72-h for natural fire curve, respectively. Thus, there is decrease in fire resistance of >44% for columns with four-sided fire exposure compared to columns with single-side fire exposure. However, the fire performance for column of portal frame with similar fire exposure condition as column is 43% lesser compared to column with fixed

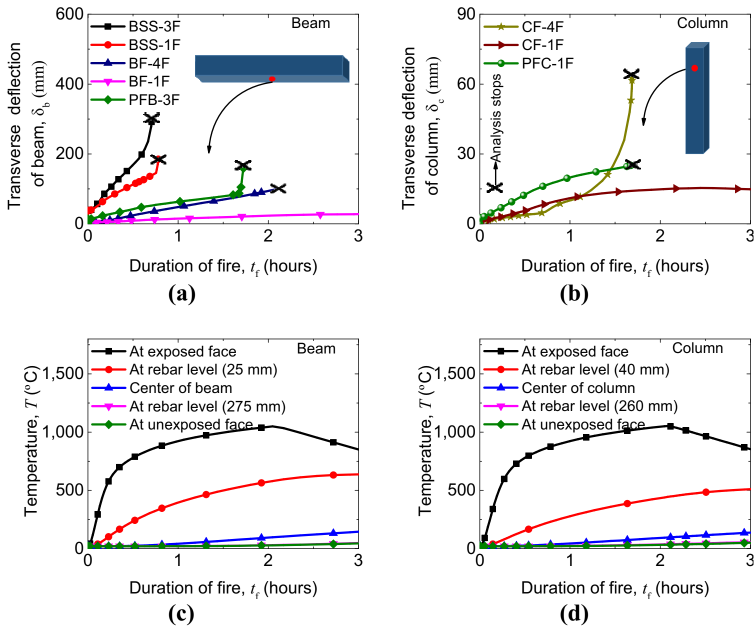


Figure 8. Peak deflection and temperature-time history for considered cases of RC beams and columns under fire loading

ends. The deflection of beam induces a degree of rotation in the beam-column joint, which in turn increases the deflection of column at a relatively higher rate. The displacement for CF-1F and PFC-1F is significantly higher than CF-4F at initial stages of fire loading because of the dominant eccentric loading effect on column, which at later stages is exceeded by CF-4F because of the higher effect of increase in temperature of reinforcing steel bars for four-side fire exposure. Figures 8(c) and 8(d) show the temperature distribution in the RC beam, column and portal frame for 3 h of fire loading. The exposed surface of beams and column is found to have attained temperature of 1024°C after 2-h of fire loading, which is substantially higher to decrease the structural strength considerably.

Figure 9 shows the peak strain-time history of concrete for RC structural member under thermo-mechanical loading. As shown in the figure, under fire exposure the failure in BSS-3F, BSS-1F and PFB-3F have started at a tensile strain of about 1.2%, after which the strain abruptly increases, indicating the point where structural collapse initiates. Similarly, the RC columns (CF-4F and PFC-1F) shows collapse at tensile strain of about 2.4%, after which the rate of change of strain shows a substantial increase. However, no such collapse point is observed for the other cases considered for the study, i.e. BF-1F and CF-1F. The strain for these cases are comparatively lesser than other RC structural members considered in the study.

Thus, it can be concluded that end support conditions of the RC structural members have significant influence on fire resistance of RC members with decrease of about 76% in the fire resistance of beam with simply-supported ends compared to beams with fixed ends. The effect of number of sides of fire exposure on beams with simply supported ends is significantly lesser (~7%) for decrease in fire resistance because of three-sided fire exposure compared to single-sided fire exposure. However, the effect of number of sides of fire exposure in case of columns is quite influential in decreasing fire resistance, where RC columns with four-sided fire exposure have fire resistance of about 44% lesser compared to column with single-sided fire exposure. The beam and columns of portal frame are found have fire resistance of about 60% and 43% lesser compared to to the beams and columns with fixed ends and having similar fire exposure scenario. Thus, it can be concluded that RC structure are more vulnerable compared to the structural members with fixed ends. In addition, the structural members with simply supported ends are found mostly vulnerable compared to all other cases considered for study.

Figure 10 illustrates quantile-quantile plots (Q-Q) for the different cases of RC members and structure under characteristic fire load density of 780 MJ/m^2. In the present study, the deflection response of the RC structure (δ_b and δ_c) are considered as EDP parameters for probabilistic studies, which are found to follow normal distribution for BF-1F and CF-1F

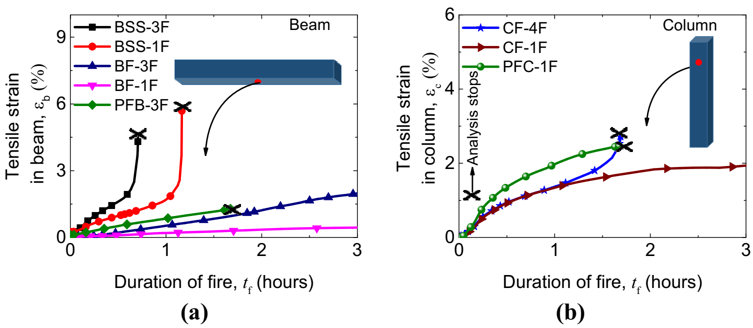


Figure 9.
Strain profile in RC members and structure under fire loading

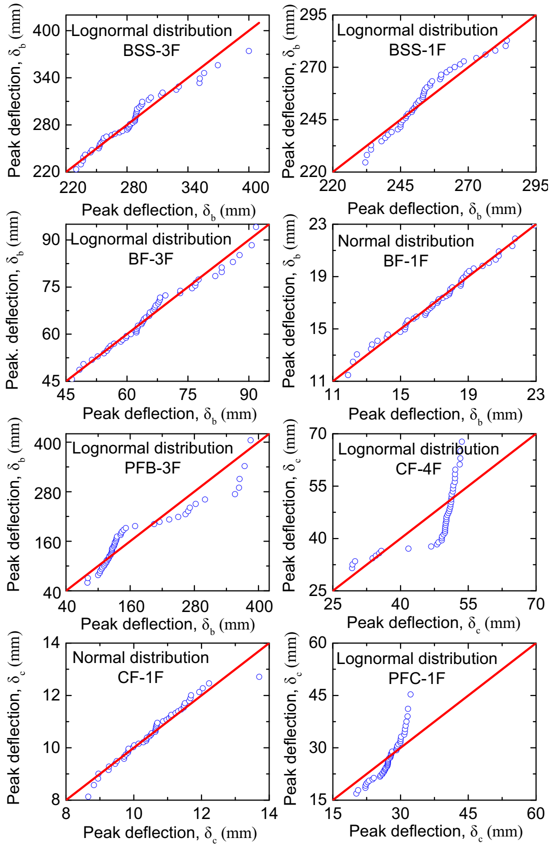


Figure 10. Quantile-quantile (Q-Q) plot for distribution of responses in considered cases of RC members and structure at fuel load of 780 MJ/m^2

and lognormal distribution for BSS-3F, BSS-1F, BF-3F, PFB-3F, CF-4F and PFC-1F, in lieu of different type of distributions assumed for the variabilities in input data. Figure 11 shows the box plot for the deflection response of different cases of RC structure under characteristic fire load density of 780 MJ/m^2 for 3-h fire duration. The box plots indicate the range of the responses obtained for this study, from which the skewness of the data can easily be achieved. The inter-quartile range (25%–75% range of data) for the distribution of responses for BSS-3F, BSS-1F BF-3F and BF-1F are 32 mm, 25 mm, 13 mm and 5 mm, respectively, whereas it is comparatively higher (about 85 mm) in case of beams of RC portal frame. Similarly, the inter-quartile range for CF-4F, CF-1F and PFC-1F are 4 mm, 2 mm and 4 mm, respectively, indicating that response of column is less affected by the variabilities and uncertainties considered in the study. Thus, the beams of portal frame with three-sided fire shows significant variation in response because of the uncertainties considered compared to other cases adopted for study.

Figure 12 shows the probability density function for distribution of responses obtained for different cases of RC structure subjected to fire hazard because of the characteristic fuel load of 780 MJ/m^2. The mean value of deflection observed for PFB-3F is 132.30 mm, whereas the mean deflections are 66.40 mm and 17.19 mm for BF-3F and BF-1F, respectively. Thus,

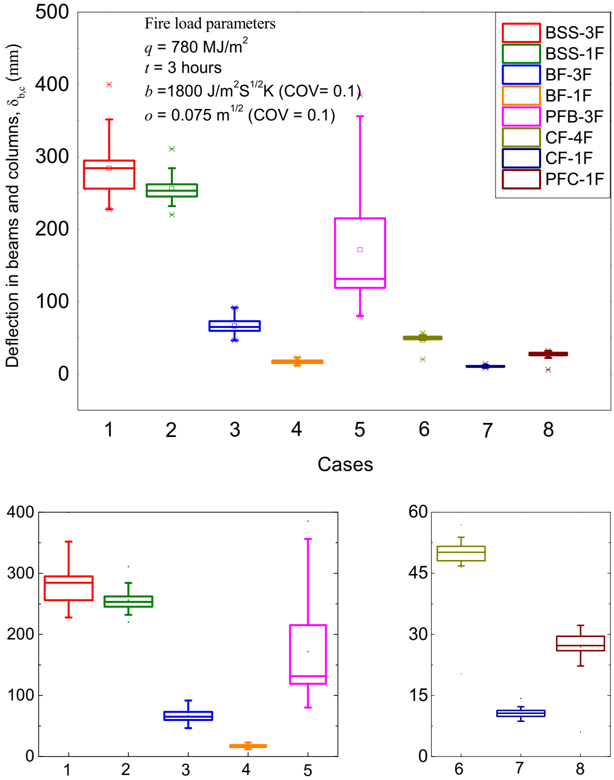


Figure 11.
Box plot for distribution of responses in considered cases of RC members and structure at fuel load of 780 MJ/m^2

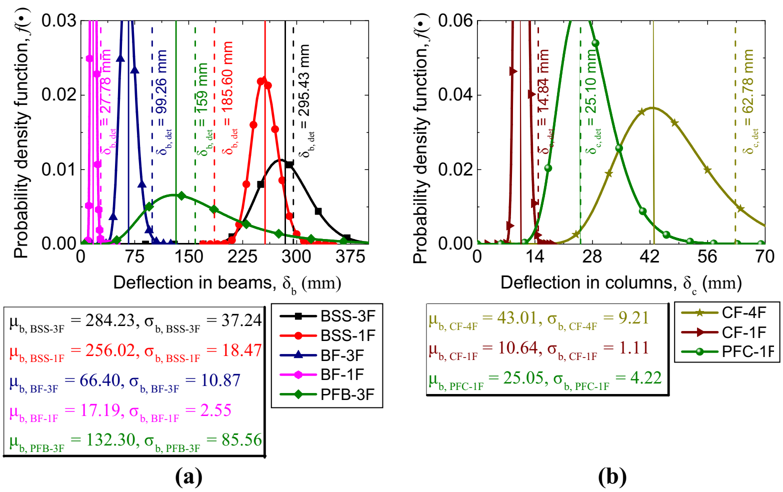


Figure 12.
Probability density function for distribution of responses in considered cases of RC members and structure at fuel load of 780 MJ/m^2

the mean deflections as observed for the PFB-3F is 50% higher than BF-3F, which has same fire exposure scenario, thereby showing its increased probability to failure during fire hazard. Similarly, the beams with simply supported ends show a mean displacement of 284.23 mm and 256.02 mm for BSS-3F and BSS-3F, respectively. Thus, end restraint conditions are found to be highly influential in affecting response of the structure under similar loading scenarios. In case of columns, the mean value for displacement obtained for PFC-1F is 25.05 mm, whereas it is 10.64 mm for CF-1F (~ 58% lower). In addition, the coefficient of variation (COV) of the responses obtained for the beams and columns of RC portal frame are 0.65 and 0.16, respectively, whereas it is significantly lower for other cases considered for study. The COV for responses of BSS-3F, BSS-1F, BF-3F, BF-1F, CF-4F and CF-1F are 0.13, 0.07, 0.16, 0.15, 0.21 and 0.10, respectively. Based on the distribution of responses, it can be concluded that beams with simply supported ends are least sensitive to the uncertain parameters for fire loading of structures, followed by the beams with fixed ends. The RC portal frame is found to be highly sensitive to parameters of uncertainties among the considered cases of study.

Figure 13 shows the probability of failure for RC beams under fire loading with fire load density as an *IM* parameter. The failure of beams with simply-supported ends starts at a fire load density of 150 MJ/m^2 for BSS-3F, which increased to 180 MJ/m^2 for BSS-1F because of the single-sided fire exposure. However, the failure for fixed beams is observed to start at 350 MJ/m^2 and 800 MJ/m^2 for BF-3F and BF-1F, respectively. The fire load density for 50% failure probability of BF-3F and BF-1F are 1418 MJ/m^2 and 1710 MJ/m^2, which decreased to 280 MJ/m^2 and 390 MJ/m^2 for BSS-3F and BSS-1F (~ 80% lower), respectively, for simply supported ends. In this regard, the failure of the portal frame based on the capacity of beam (local failure) starts at 300 MJ/m^2, which is lower compared to the beams with fixed ends and higher than the beams with simply supported ends. The fire load density for the 50% probability of failure obtained for beam of portal frame is 527 MJ/m^2, which increased to 1418 MJ/m^2 and 1710 MJ/m^2 for BF-3F and BF-1F, respectively. This decrease in the fire load resistance (~63%), when considering structural fragility rather than member fragility is highly significant and needs proper consideration in the structural fire design from safety point of view. Furthermore, the fire load density of 780 MJ/m^2 is highly vulnerable to the structures consisting of beams with simply supported ends, irrespective of the exposure scenario. Thus, it is highly advisable to avoid the use of beams with the simply supported ends for fire resistant design of structures because of its relatively low fire resistance.

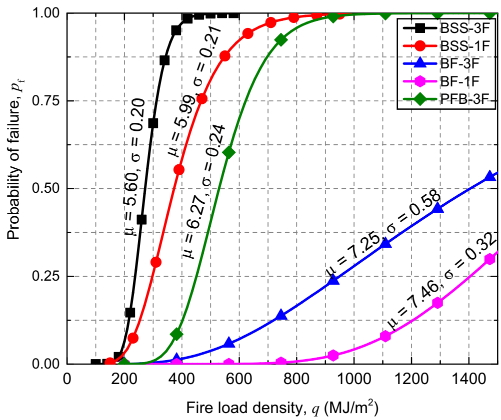


Figure 13.
Fragility curves for RC beams subjected to fire loading

Figure 14 shows the member and structural fragility of RC columns under fire loading with fire load density as an *IM* parameter. The mean fire load density for 50% failure probability of CF-4F, CF-1F, and PFC-1F are 600 MJ/m^2, 1490 MJ/m^2, and 1145 MJ/m^2, respectively. There is significant reduction of about 60% in the structural fire resistance of the RC column for four-sided fire exposure in comparison to the single-sided fire exposure. However, the portal frame can resist the fire because of fire load density of about 400 MJ/m^2 and has a mean structural capacity to resist characteristic fire load of 1145 MJ/m^2, which is about 23% higher compared to the individual column member with single-sided fire exposure. Based on the comparison of member and structural fragility of RC structure, it can thus be concluded that the RC portal frame is highly vulnerable to fire load compared to the beams and columns with fixed ends and similar fire exposure scenario.

9. Conclusions

Herein, the behavior of RC members and structure, i.e. beams, columns and portal frame, under thermo-mechanical loading is investigated through 3-D FE analysis. The study is carried out for the different possible end support conditions and fire exposure scenario. Probabilistic study is carried out by developing the fragility curves for the RC members and structure to assess the safety and reliability under extreme fire loading. Finally, the fragility function developed for the 3D beams and column is compared with the structural fragility developed for the RC portal frame based on the deflection criteria from literature. A framework for the probabilistic assessment of vulnerability of RC structure under fire hazard through FE analysis is developed, which can be effectively used to investigate the structural fire performance of other similar structure and to enhance safety and reliability of the structure under hazardous environment. Therefore, based on the current investigation, the following conclusions can be drawn:

- The response obtained using the 2-D FE modeling approach shows significant difference of about 10% from the actual behavior of structure, whereas the responses for 3-D FE models are quite in agreement (98% accuracy) with the results from experimental studies available in literature. Thus, 3-D FE analysis approach is recommended for study of behavior of RC structures under thermo-mechanical loading.

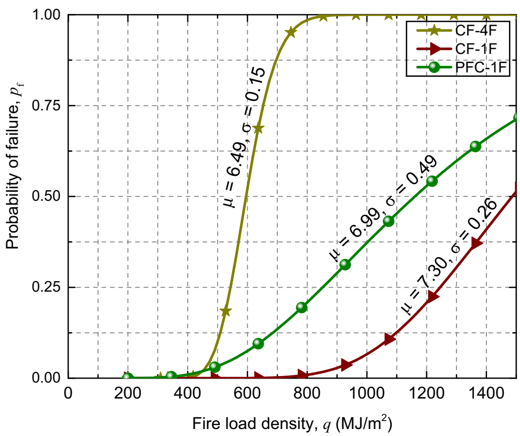


Figure 14.
Fragility curves for RC columns subjected to fire loading

- Based on the deflection criteria, the fire resistance of beams with simply supported conditions has a fire resistance of about 76% less compared to the beams with fixed ends, which shows that the end support conditions for RC structural members have significant influence on fire resistance of RC structures. Moreover, the beams with simply supported ends have 50% failure probability at a significantly lower fire load density of 280 MJ/m^2, which is about 80% lesser in comparison to the fixed beams. Thus, beams with simply supported end conditions are highly vulnerable to relatively lower fire load density and is hereby unadvisable for structures with the higher fire resistance requirements.
- Multiple-sided fire exposure is required to be considered in the assessment and design of axially loaded members.
- The beams with simply supported ends are least sensitive to the uncertain parameters of fire loading scenario, followed by the beams with fixed ends, whereas the RC portal frame is found to be highly sensitive to the same uncertain parameters.
- The fire resistance of beam and columns of the portal frame for natural fire curve is considerably lower compared to individual beams and columns with fixed end and similar fire exposure scenario. Moreover, structural vulnerability is higher compared to the member vulnerability. Thus, the design approaches based on consideration of studies on beams and columns as design criteria for RC structures under fire loading may lead to the assumptions, which will increase the vulnerability of the structure to hazards.

References

ABAQUS/Explicit User's manual, version 6.14, Dassault Systems, Simulia Corporation, Province, Rhode Island (RI), USA.

ACI 216.1-07 (2007), *Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies*, American Concrete Institute (ACI), Farmington Hills, MI (MI).

ASCE/SEI/SFPE 29-05 (2006), *Standard Calculation Methods for Structural Fire Protection*, American Society of Civil Engineers (ASCE), Reston, VT (VA).

Aslani, F. and Bastami, M. (2011), "Constitutive relationships for normal-and high-strength concrete at elevated temperatures", *ACI Materials Journal*, Vol. 108 No. 4, pp. 355-364.

Balaji, A., Aathira, M.S., Pillai, T.M.M. and Nagarajan, P. (2016), "Reliability studies on RC beams exposed to fire based on is 456: 2000 design methods", *Structural Engineering and Mechanics*, Vol. 59 No. 5, pp. 853-866.

Balogh, T. and Vigh, L.G. (2016), "Complex and comprehensive method for reliability calculation of structures under fire exposure", *Fire Safety Journal*, Vol. 86, pp. 41-52.

Chang, Y.F., Chen, Y.H., Sheu, M.S. and Yao, G.C. (2006), "Residual stress-strain relationship for concrete after exposure to high temperatures", *Cement and Concrete Research*, Vol. 36 No. 10, pp. 1999-2005.

Eamon, C.D. and Jensen, E.A. (2013a), "Reliability analysis of RC beams exposed to fire", *Journal of Structural Engineering (Engineering)*, Vol. 139 No. 2, pp. 212-220.

Eamon, C.D. and Jensen, E.A. (2012), "Reliability analysis of prestressed concrete beams exposed to fire", *Engineering Structures*, Vol. 43, pp. 69-77.

Eamon, C.D. and Jensen, E.A. (2013b), "Reliability analysis of reinforced concrete columns exposed to fire", *Fire Safety Journal*, Vol. 62, pp. 221-229.

Ellingwood, B.R. (2005), "Load combination requirements for fire-resistant structural design", *Journal of Fire Protection Engineering*, Vol. 15 No. 1, pp. 43-61.

Ellingwood, B.R. and Kinali, K. (2009), "Quantifying and communicating uncertainty in seismic risk assessment", *Structural Safety*, Vol. 31 No. 2, pp. 179-187.

ElMohandes, F. and Vecchio, F.J. (2016), "Reliability of temperature-dependent models for analysis of reinforced concrete members subjected to fire", *ACI Structural Journal*, Vol. 113 No. 3, pp. 481-490.

EN 1991-1-2 (2002), *Eurocode 1 Actions on structures - Part 1-2: General actions – Actions on Structures Exposed to Fire*, European Committee for Standardization.

EN 1992-1-2 (2004), *Eurocode 2: Design of Concrete Structures - Part 1-2: general Rules – Structural Fire Design*, European Committee for Standardization.

EN 1994-1-2 (2005), *Eurocode 4: Design of Composite Concrete and Steel Structures*, European Committee for Standardization.

Gernay, T. and Franssen, J.-M. (2012), "A formulation of the eurocode 2 concrete model at elevated temperature that includes an explicit term for transient creep", *Fire Safety Journal*, Vol. 51, pp. 1-9.

Gernay, T., Khorasani, N.E. and Garlock, M. (2016), "Fire fragility curves for steel buildings in a community context: a methodology", *Engineering Structures*, Vol. 113, pp. 259-276.

Gernay, T., Khorasani, N.E. and Garlock, M. (2018), "Fire fragility functions for steel frame buildings: sensitivity analysis and reliability framework", *Fire Technology*, Vol. 55 No. 4, pp. 1175-1210.

Guo, Q., Shi, K., Jia, Z. and Jeffers, A.E. (2013), "Probabilistic evaluation of structural fire resistance", *Fire Technology*, Vol. 49 No. 3, pp. 793-811.

Huang, P.T. and Delichatsios, M.A. (2010), "Quantitative risk analysis for the response of steel beams in fires", *Journal of Structural Fire Engineering*, Vol. 1 No. 4, pp. 231-241.

Ioannou, I., Aspinall, W., Rush, D., Bisby, L. and Rossetto, T. (2017), "Expert judgment-based fragility assessment of reinforced concrete buildings exposed to fire", *Reliability Engineering and System Safety*, Vol. 167, pp. 105-127.

Iqbal, S. and Harichandran, R.S. (2010), "Capacity reduction and fire load factors for design of steel members exposed to fire", *Journal of Structural Engineering*, Vol. 136 No. 12, pp. 1554-1562.

IS 456 (2000), *Plain and Reinforced Concrete – Code of Practice*, Bureau of Indian Standards (BIS), New Delhi.

IS 875 - Part 2 (1987), *Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures: imposed Loads*, Bureau of Indian Standards (BIS), New Delhi.

ISO 834 (2014), *Fire Resistance Tests – Elements of Building Construction*, International Organization for Standardization (ISO), Vol. 11, Geneva.

Jiang, L. and Usmani, A. (2018), "Towards scenario fires – modelling structural response to fire using an integrated computational tool", *Advances in Structural Engineering*, Vol. 21 No. 13, pp. 2056-2067.

Kho, T.S., Block, F.M. and Lowry, T.G. (2017), "Determining the fire rating of concrete structures: case study of using a probabilistic approach and travelling fires", *Applications of Structural Fire Engineering*,

Khorasani, N.E., Gardoni, P. and Garlock, M. (2015), "Probabilistic fire analysis: material models and evaluation of steel structural members", *Journal of Structural Engineering*, Vol. 141 No. 12.

Khorasani, N.E., Garlock, M. and Gardoni, P. (2016), "Probabilistic performance-based evaluation of a tall steel moment resisting frame under post-earthquake fires", *Journal of Structural Fire Engineering*, Vol. 7 No. 3, pp. 193-216.

Knox, J.J., Pankaj, P. and Usmani, A.S. (2008), "The behavior of Pre-Damaged reinforced concrete structural systems in fire", *14th World Conference on Earthquake Engineering, Beijing, China.*

Kodur, V., Cheng, F.P., Wang, T.C. and Sultan, M.A. (2003), "Effect of strength and fiber reinforcement on fire resistance of high-strength concrete column", *Journal of Structural Engineering*, Vol. 129 No. 2, pp. 253-259.

Lange, D., Devaney, S. and Usmani, A. (2014), "An application of the PEER performance based earthquake engineering framework to structures in fire", *Engineering Structures*, Vol. 66, pp. 100-115.

Rafi, M.M., Nadjai, A., Ali, F. and Talamona, D. (2007), "Aspects of behavior of CFRP reinforced concrete beams in bending", *Construction and Building Materials*, Vol. 22 No. 3, pp. 277-285.

Roy, T. and Matsagar, V.A. (2017), "Multi-Hazard assessment of steel buildings retrofitted using passive control devices", *16th World Conference on Earthquake (16WCEE), Santiago, Chile*.

Saha, S.K., Matsagar, V. and Chakraborty, S. (2016a), "Uncertainty quantification and seismic fragility of base-isolated liquid storage tanks using response surface models", *Probabilistic Engineering Mechanics*, Vol. 43, pp. 20-35.

Saha, S.K., Matsagar, V. and Jain, A.K. (2016b), "Seismic fragility of base-isolated water storage tanks under non-stationary earthquakes", *Bulletin of Earthquake Engineering*, Vol. 14 No. 4, pp. 1153-1175.

Shi, K., Guo, Q. and Jeffers, A. (2013), "Stochastic analysis of structures in fire by monte carlo simulation", *Journal of Structural Fire Engineering*, Vol. 4 No. 1, pp. 37-46.

Shinozuka, M., Feng, M.Q., Lee, J. and Naganuma, T. (2000), "Statistical analysis of fragility curves", *Journal of Engineering Mechanics*, Vol. 126 No. 12, pp. 1224-1231.

Sidibé, K., Duprat, F., Pinglot, M. and Bourret, B. (2000), "Fire safety of reinforced concrete columns", *ACI Structural Journal*, Vol. 97 No. 4, pp. 642-647.

Van Coile, R., Annerel, E., Caspeele, R. and Taerwe, L. (2013), "Full-probabilistic analysis of concrete beams during fire", *Journal of Structural Fire Engineering*, Vol. 4 No. 3, pp. 165-174.

Van Coile, R. and Bisby, L. (2017), "Initial probabilistic studies into a deflection-based design format for concrete floors exposed to fire", 6th international workshop on performance", *Procedia Engineering*, Vol. 210, pp. 488-495.

Van Coile, R., Caspeele, R. and Taerwe, L. (2014), "Lifetime cost optimization for the structural fire resistance of concrete slabs", *Fire Technology*, Vol. 50 No. 5, pp. 1201-1227.

Van Coile, R., Caspeele, R. and Taerwe, L. (2014), "Reliability-based evaluation of the inherent safety presumptions in common fire safety design", *Engineering Structures*, Vol. 77, pp. 181-192.

Zha, X.X. (2003), "Three-dimensional nonlinear analysis of reinforced concrete members in fire", *Building and Environment*, Vol. 38 No. 2, pp. 297-307.

Zhou, C.E. and Vecchio, F.J. (2005), "Nonlinear finite element analysis of reinforced concrete structures subjected to transient thermal loads", *Computers and Concrete*, Vol. 2 No. 6, pp. 455-479.

Corresponding author
Vasant Matsagar can be contacted at: matsagar@civil.iitd.ac.in