Framework for fragility assessment of reinforced concrete portal frame subjected to elevated temperature

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ABSTRACT

Despite realizing the importance of risk-based frameworks in fire engineering, current fire safety guidelines are largely prescriptive and over-conservative in nature, which do not consider the effect of uncertainties influencing the fire resistance of the structural systems. In this context, a probabilistic framework is presented to investigate vulnerability of a reinforced concrete (RC) portal frame subjected to fire. Here, a three-dimensional (3-D) nonlinear finite element (FE) model for the RC portal frame exposed to elevated temperature is developed, and transient thermo-mechanical analysis is carried out by duly incorporating temperature variation of thermal and mechanical properties of concrete and steel rebar. Probabilistic study is conducted by developing fragility curves based on limit state of deflection to quantify the vulnerability of the structural members exposed to fire loading. Parametric studies are conducted by varying the material properties of the RC portal frame to investigate the influence of uncertainties in the fire loading on its structural performance and fragility. The failure probability starts at a higher fire load density (450 MJ/m²) for the structure with higher concrete grade as compared to the relatively lower fire load density (200 MJ/m²) for lower grade. Therefore, the fire resistance of the RC structures has been recommended to be based on the strength of the member, rather than merely sizes of the members. The proposed stochastic analysis framework, thereby, provides a rational approach to improve the performance-based fire design of RC structures at member and structure levels by identifying the parameters affecting the structural fire resistance.

1. Introduction

Fire outbreaks are one of the most catastrophic manmade or accidental hazards causing devastations to structure and infrastructure systems [11,52]. The devastations include failure of structural and non-structural components through progressive collapse, multiple fire-induced structural failures, substantial collapse involving other complex challenges, such as, fire following earthquake and explosion [37]. As a result, it is extremely important to understand the level of risk associated with such hazards and the following catastrophe [56]. In a typical fire event, the development of temperature with time and the subsequent structural behavior at the elevated temperatures involve significant uncertainties in the system properties and loading. Such randomness in the system must be addressed carefully to recognize the critical parameters affecting the structural fire resistance. In this regard, probabilistic approaches are effective to address the influence of the uncertainties affecting the structural behavior; thereby, providing a new perspective to investigate the performance of structures exposed to fire [27]. The use of probabilistic frameworks provides an in-depth assessment of randomness in the system to facilitate the design of structures for a safer built environment under extreme fire threats.

The complexities involved in fire engineering include potential fire scenarios directly or indirectly affecting the performance of structures against fire, such as, structural fire resistance, fire mitigation system, as well as protective measures for human life and property. The structural fire resistance for a typical reinforced concrete (RC) member is achieved from a prescribed reinforcement cover for a given structural member size, aggregate type, and the subsequent exposure category (IS 456:2000 [33]; ASCE/ SEI/ SFPE 29-05 [3]; ACI 216.1-07 [2]; ASTM E119-16 [5]). Alongside, the fire mitigation strategies include minimum provisions and standards for escape routes in buildings, use of passive and active fire protection systems, such as, shape and size of structural and non-structural members, use of fire-resistant materials and sprinklers to limit the severity caused by the fire threat. On the other hand, the damage levels of structures under the fire loading depend significantly on several factors including, the duration of the fire, quantity of fuel, the
maximum temperature attained during the process and total heat energy 
[18,19]. Therefore, the extent of uncertainties influencing the structural fire design is evident from the degree of variables and the level of scatter associated with the process. Moreover, fundamental information is lacking in the current literature regarding the spatial and temporal distribution of temperature in a fire event, as till the date models based on existing standard fire test are being used to assess the fire severity [54]. Hence, based on these variabilities, adopting the probabilistic approach in fire design has been gaining importance mainly to incorporate the uncertain parameters for improving the structural resistance exposed to fire [39] and contributing towards a more improved performance-based design of structures against fire.

Early researches were mostly prescriptive in nature to obtain the fire resistance rating of structures by investigating their behavior at the member level [16,29,47]. Subsequently, the need of experimental studies was felt to determine the realistic behavior of structural members subjected to elevated temperature [9,35,43,50,72]. The newer experimental results mandated changes in the existing design code provisions for achieving desired safety levels in the structures under fire and develop future design guidelines to predict and enhance the fire endurance of structural members. Recently, advanced finite element (FE) approaches have been introduced to predict the behavior of reinforced concrete (RC) structures under fire [10,12,44,49,57-59]. However, the existing research approaches do not incorporate the complexities associated with the fire models provided in the design codes and standards that is required for achieving desired structural performance subjected to fire. Barnett and Clifton [8] and Liew and Ma [48] investigated and compared the behavior of steel structures, adopting parametric fire curves, which provided conservative results. Likewise, the study conducted by Elmohandes and Vecchio [21] showed that the temperature-dependent structural-fire models do not provide adequate confidence for a performance-based design for a fire exposed structure. These studies recommended that the probabilistic approaches are necessary to assess the fire performance for safe and economic design of structures.

Rush and Lange [64] have stated that the application of the probability and statistics in engineering is not new; however, its application in structural fire engineering is relatively new, which requires fundamental insight to understand the influence of uncertainties affecting the structural performance and its failure probability under fire. Initially, the probabilistic methodology was applied to the structural fire engineering mainly for investigating load and resistance factors to be included in structural designs [20]. Thereafter, Monte Carlo (MC) simulation and reliability methods were used to investigate the failure probability of individual RC members by considering the structural capacity and fire loading as the uncertain parameters [66,71]. Few studies were also conducted on the probabilistic aspects of fire loading on steel structures by using varied reliability techniques [7,26,40,41]. The influence of uncertain parameters on the vulnerability and reliability index for the structures was observed to be significant [6,69,70]. More recently, advanced methodologies were presented to calculate the failure probability and estimate the reliability of structural members through simplified numerical/ FE approaches [14,62,68]. The research conducted until the date has mostly addressed fire behavior at member (local) level, however limited issues concerning the fire behavior at structural/system (global) level have so far been addressed, especially on the probabilistic aspects. While significant research has been conducted to establish relative fire-resistance ratings for individual members such as beams and columns, relatively lesser knowledge on the behavior of the thermo-mechanical response of structural frame under fire is available. Therefore, the need to research in developing probabilistic analysis procedures could not be further emphasized, such that it could be utilized in evaluating the structural fire resistance under known uncertainties to develop performance-based design guidelines for the structures under fire [30]. Hence, the present work addresses this requirement of establishing probabilistic analysis procedure to evaluate vulnerability of RC portal frame under fire.

In this study, a framework for fragility assessment of reinforced concrete (RC) portal frame subjected to elevated temperature is presented, considering uncertainties in fire loading, and subsequently parametric studies are conducted by varying material properties affecting the structural capacity. The probabilistic study of the thermo-mechanical response of the RC portal frame is conducted through development of density functions and fragility curves based on the defined limit states of failure. Three-dimensional (3-D) nonlinear FE model is developed for the RC portal frame, and the thermo-mechanical response evaluated from it are compared with the experimental results available in the literature. The specific objectives of the present investigation are: (i) to study the mechanics of the thermo-mechanical responses obtained for the RC structure subjected to fire, (ii) to study the variation of the responses considering uncertainties in the material parameters and fire loading, and (iii) to develop fragility (vulnerability) curves for estimating failure probability of the RC structure under fire, useful in performance-based fire engineering of structures.

2. Structural description and numerical modeling

An RC portal frame is assumed as a part of an office building for which the plan and elevation are shown in Fig. 1. The portal frame consists of a beam spanning 2700 mm (clear span) with a cross-section of 300 mm × 230 mm, and two columns with a cross-section of 300 mm × 300 mm having height of 3000 mm. The beam is reinforced with six steel rebar of 16 mm φ (where, φ represents the diameter/ size of the steel rebar) enclosed by shear stirrups of 6 mm φ spaced at 100 mm center-to-center (c/c) distance throughout the beam as shown in Fig. 2(a). The columns are reinforced with eight rebar of 25 mm φ enclosed by lateral ties of 10 mm φ spaced at 150 mm c/c distance in the central portion, and 75 mm c/c distance near the supports up to a length of 500 mm, as shown in Fig. 2(b). The reinforcements in the beam and columns are provided maintaining a standard concrete nominal cover of 30 mm and 40 mm, respectively for 2-h fire resistance as specified in the Indian standard, IS 456:2000 [33]. The effective span of the beam considered for the present study is 3000 mm, which is based on the commonly adopted span to depth ratio of the RC structure.

The mechanical loading from the dead load (DL) and live load (LL) is calculated on the basis of “Business and Office Buildings” as per the Indian standard, IS 875 – Part 2: 1987 [32]. The DL is calculated considering the self-weight of the members located above the portal frame and the LL is estimated by considering 2.5 kN/m² for ‘rooms for general use with separate storage’. The mechanical load of 221.25 kN/ m² is applied at the upper surface of the beam in the form of distributed load, which includes the self-weight of the RC frame. The seismic design of the RC portal frame has been carried out under earthquake ground motion pertinent to a selected site located at New Delhi in India assuming that the RC frame is capable of sustaining inelastic deformation and providing energy dissipation under the lateral seismic load. However, it is assumed that the RC portal frame has not been specifically designed to resist any accidental loads such as fire loads. Scenario-based fire loading is considered for the investigation, in which the fire load is assumed to act at the soffit of the beam and internal sides of the columns, i.e. single-side exposure for the beam and columns.

2.1. Numerical modeling and analysis of the RC portal frame

In the current study, a finite element (FE) model of the RC portal frame is developed. To predict its behavior when subjected to fire loading, the thermo-mechanical analysis is conducted in commercially available software ABAQUS® [1]. To add further, multiple simulations are carried out through the ABAQUS® scripting interface programmed in the object-oriented Python language for probabilistic studies for the considered RC frame. The scripting interface is used to access the functionalities of the ABAQUS®, and the scripts coded in the interface
holds the advantage of performing a number of simulations without intervention of the user. The FE model of the RC frame is developed using the 3-D extrusion option available in the software. It is important to note that the 3-D model is deliberately chosen for the FE modeling of the RC portal frame in order to capture the volumetric changes occurring due to the thermal actions [15].

The nonlinearity of concrete is modeled using the continuum- and plasticity-based concrete damaged plasticity (CDP) model, which takes into account the degradation of the elastic stiffness induced by the plastic strains in both compression and tension. The CDP model is a smeared crack approach in which individual micro-cracks need not be tracked independently [36,51]. The effect of dowel action between concrete and rebar is modeled using suitable tension stiffening parameters available in the CDP model. The steel reinforcement bars are modeled based on classical metal plasticity and have axial tensile forces and nodal temperatures as degrees of freedom. The reinforcing steel is considered to behave as elastoplastic hardening material in both the compression and tension, with von-Mises plasticity yield criterion having associated plastic flow and isotropic hardening. In case of the thermal analysis, shear failure is not a governing criterion; hence, stirrups or lateral ties are not modeled in the RC beam and columns of the RC portal frame.

The thermo-mechanical analysis is conducted in two steps: (i) Step-1 involves mechanical analysis of the portal frame, and (ii) Step-2 relates to thermal computation, considering the effect of the propagated mechanical load to obtain the thermo-mechanical responses for the fire exposed structure. The mesh of the concrete geometry generated in both the steps of the FE analysis contains coupled temperature-displacement continuum element, C3D8RT with reduced integration. Each node of an element has four degree of freedoms (DOFs), translational DOFs (1, 2, 3) in three mutually perpendicular directions, x, y, z, and a temperature DOF (11). The x and y directions are in plane, whereas the z direction is along the length of beam. The steel rebar is meshed using truss element, T3D2T which have an axial DOF and an additional temperature DOF. The interaction between the steel rebar and surrounding concrete is achieved using the TIE constraint option, which allows both mechanical interaction and the temperature distribution between concrete and steel. The meshed configuration of the RC portal frame with steel rebar is shown in Fig. 3. The geometric nonlinearity in the model is considered using NLGEOM option available in ABAQUS®. The portal frame is
assumed to be fixed at the supports of the column. 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 the calculation of dead and live loads acting on the portal frame. Thus, the effect of lateral restraint in thermomechanical response of the beam is neglected in order to keep the computational efforts tractable. The connections of the beam and column adopted in the analysis are in accordance with the design guidelines prescribed by the Indian standard (IS 456:2000) [33], which allows carrying out the subsequent modeling to assess the behavior of the RC frame under the effect of fire. Scenario-based fire loading is considered with single-side exposure for beam and columns, i.e. the fire load is assumed to act at the soffit of the beam and internal sides of the columns, as shown in Fig. 2. As the failure in flexural mode is predominant in the beam, which is primarily owing to the fire exposure at the soffit of the beam, single-side fire exposure for the beam is considered for the study. A convective heat transfer coefficient in the form of surface film condition is applied at the exposed and unexposed surface of the frame, whereas radiation is provided in terms of emissivity coefficient. The attributes of the FE model are presented in Table 1, which shows the modeling parameters adopted for coupled-temperature displacement analysis of the RC structure under fire. The thermal analysis in Step-2 includes heat transfer into the beam through conduction, while fire at the surface is applied by thermal convection and radiation process. The entire portal frame is subjected to an ambient temperature of 20°C in the form of predefined initial condition. During the thermal analysis, the fire load is applied to the nodes of the exposed face of the structure through convection and radiation in the form of natural fire curve. The thermal gradient is thus developed between the nodes of the exposed face and the unexposed nodes in the structure, leading to the heat flow through conduction depending on the thermal conductivity and specific heat at the particular temperature. The thermo-mechanical analysis is carried out for a fire duration of 180 min.

Fig. 2b. Details of the reinforcement in the RC column.

Fig. 3. Meshed configuration of the RC portal frame with steel rebar.

<table>
<thead>
<tr>
<th>Material</th>
<th>Element Type</th>
<th>Elasticity</th>
<th>Plasticity Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>C3D8RT</td>
<td>Elasticity (E)</td>
<td>Concrete damagedplasticity (CDP)</td>
</tr>
<tr>
<td>Reinforcing steel</td>
<td>T3D2T</td>
<td>Poisson’s ratio (ν)</td>
<td>Classical plasticity</td>
</tr>
<tr>
<td>Constraint</td>
<td>TIE constraint</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Geometric nonlinearity</td>
<td>NLGEOM</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Restraint</td>
<td>Fixed at column bases</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fire exposure</td>
<td>Beam soffit and inner column sides</td>
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<td></td>
</tr>
</tbody>
</table>
2.2. Numerical solution scheme

The numerical solution scheme adopted in the study is coupled displacement-temperature approach, where the effect of mechanical and thermal loading are evaluated simultaneously. The numerical techniques used for the thermo-mechanical analyses are finite difference method (FDM) discretization and finite element method (FEM) spatial approximation. The solution schemes used to assess the RC frame under the thermo-mechanical loading are discussed briefly hereunder.

The thermo-mechanical analysis of the RC frame under the fire loading is performed in ABAQUS®/ Standard as it is more efficient for solving smooth nonlinear thermo-mechanical problems, where the responses are estimated incrementally. The RC portal frame is subjected to two types of boundary conditions for the thermo-mechanical analysis: mechanical and thermal. The equation for the thermal conduction in an isotropic material in terms of Cartesian coordinates is of the form,

\[ Q = \rho c \frac{\partial T}{\partial t} - \nabla \cdot (\lambda \nabla T) \]  

(1)

where, \( Q \) is the overall heat transferred from outside; \( \rho \) is the density of material (kg/m\(^3\)); \( c \) and \( \lambda \) are the specific heat capacity (J kg\(^{-1}\)C\(^{-1}\)) and thermal conductivity (W m\(^{-1}\)C\(^{-1}\)), respectively; \( \frac{\partial T}{\partial t} \) is the partial derivative of temperature with respect to time; and \( \nabla \) denotes \( \frac{\partial}{\partial x} \). From the equation, the transient conduction problem follows parabolic relation with time dependence and elliptic behavior with spatial co-ordinates. Furthermore, Eq. (1) can be more explicitly expressed for heat transfer at the surface and within the beam as [73],

\[ Q = \beta \frac{\partial T}{\partial n} \bigg|_\Gamma = h_c (T_f - T_c) + \sigma \varepsilon (T_f^4 - T_c^4) \]  

(2)

\[ \frac{\partial T}{\partial t} = \frac{1}{c_p} \left[ \frac{\partial}{\partial x} \left( \lambda \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left( \lambda \frac{\partial T}{\partial y} \right) \right] \]  

(3)

where, \( \beta \) is the heat transfer coefficient; \( \Gamma \) is the surface of heat transfer; \( h_c \) is the convective heat transfer coefficient; \( T_f \) and \( T_c \) are the ambient and surface temperatures, respectively; \( \sigma \) is the Stefan-Boltzmann constant; and \( \varepsilon \) is the emissivity coefficient.

The thermal boundary condition imposed on the FE model is a Dirichlet boundary condition for which the temperature variation on the body of the structure with time is given as,

\[ T(X, t) = T_w(X, t) \]  

(4)

where, \( T_w \) is a function providing temperature at a specified time; and, \( X \) is a spatial component. The mechanical properties of concrete and reinforcing bars depend upon the temperature. Using the thermal conduction equation, a distribution of temperature is obtained for the model and the mechanical analysis is carried out based on the degraded mechanical properties of the material at elevated temperature.

The implicit nonlinear FE solution technique uses the backward finite difference algorithm to integrate the governing equations in time domain. The thermal field is solved in each time step \( \Delta t \), and subsequently the mechanical problem is solved using the obtained thermal strains. The nonlinear system of equations is approximated as linear within each time step and are solved using the Newton-Raphson
iteration scheme for static equilibrium. ABAQUS®/Standard uses automatic increment by default and the rate of convergence in each time step is monitored to determine the appropriate time and load increments. The tolerances on residual errors need to be satisfied in every iteration before proceeding to the next time step. For the finite element analysis, the time-step of analysis depends on the element size, calculated as $\Delta t > (pc/6k) \Delta t^2$ where, $p$ is mass density, $c$ is specific heat, $k$ is conductivity, and $\Delta t$ is element size. Based on this formulation, the minimum time step required is estimated for the investigation of the RC portal frame subjected to thermo-mechanical loading.

3. Material model for thermal analysis

$$\sigma_{ref} = \sigma_t \begin{bmatrix} 1.012 - 0.0005T \leq 1.0 \\ 0.985 + 0.0002T - 2.235 \times 10^{-2}T^2 + 8 \times 10^{-3}T^3 \\ 0.44 - 0.0004T \\ 0 \end{bmatrix} \begin{bmatrix} 20^\circ C \leq T \leq 100^\circ C \\ 100^\circ C < T \leq 800^\circ C \\ 800^\circ C < T \leq 1000^\circ C \\ T > 1000^\circ C \end{bmatrix}$$

In order to predict the performance of the RC portal frame under the effect of fire, the material properties at elevated temperatures must be explicitly known in order to determine the behavior of the structural members accurately [67]. The non-homogenous RC material requires a suitable nonlinear material model defined through appropriate parameters to capture both elastic and plastic behavior in compression and tension under the thermo-mechanical effects caused during fire.

3.1. Thermal properties

The properties influencing rise in temperature and its distribution in the concrete section and reinforcing bars are mass/ density loss ($\rho$), thermal conductivity ($\lambda$), coefficient of thermal expansion ($\alpha$), and specific heat capacity of the steel reinforcing bars at elevated temperature are obtained from the comparative studies reported by Gardner et al. [24].

3.2. Mechanical properties

The mechanical properties determining the performance of the RC frame under elevated temperatures are compressive strength, modulus of elasticity, and stress-strain response of constituent materials [42]. The compressive and tensile strengths of concrete under elevated temperature are obtained from the following expressions [4,13],

$$\sigma_{ct} = \sigma_t \begin{bmatrix} 1.05 - 0.0025T \leq 1.0 \\ 0.5 \end{bmatrix} \begin{bmatrix} 20^\circ C \leq T \leq 100^\circ C \\ 100^\circ C < T \leq 200^\circ C \end{bmatrix}$$

$$\sigma_{tt} = \sigma_t \begin{bmatrix} 1.2 - 0.0011T \leq 0 \\ 0.8 \end{bmatrix} \begin{bmatrix} 200^\circ C < T \leq 800^\circ C \end{bmatrix}$$

where, $\sigma_c$, $\sigma_t$, $\sigma_{ct}$ and $\sigma_{tt}$ are the compressive and tensile strengths of concrete cylinder at ambient and elevated temperature, $T$, respectively. The elastic modulus at elevated temperature, $E_{ct}$ is affected similarly by the factors that influence the compressive strength, which is determined by the following expression,

$$E_{ct} = E_t \begin{bmatrix} 1.015 - 0.00154T + 2 \times 10^{-2}T^2 + 3 \times 10^{-4}T^3 \end{bmatrix} \begin{bmatrix} 20^\circ C \leq T \leq 100^\circ C \\ 100^\circ C < T \leq 1000^\circ C \\ T > 1000^\circ C \end{bmatrix}$$

Thermal conductivity, coefficient of thermal expansion, and

Table 2

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Unit</th>
<th>M20</th>
<th>M30</th>
<th>M40</th>
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</thead>
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<tr>
<td>Compressive strength of concrete ($f_{ck}$)</td>
<td>MPa</td>
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<td>30</td>
<td>40</td>
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<tr>
<td>Elastic modulus of concrete ($E_t$)</td>
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<td>22360.68</td>
<td>27386.12</td>
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<td>Density ($\rho$)</td>
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<tr>
<td>Conductivity ($\lambda$)</td>
<td>W/(m°C)</td>
<td>1.52</td>
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<td></td>
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<tr>
<td>Specific heat ($c$)</td>
<td>J/(kg°C)</td>
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<tr>
<td>Coefficient of expansion ($\alpha$)</td>
<td>°C</td>
<td>9.0 \times 10^-6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
ε_{max} = 2\sigma_c/E_{ci} + 0.21 \times 10^{-4} (T - 20) - 0.9 \times 10^{-3} (T - 20)^2 \tag{11}

\beta_{aT} = \beta_{aT}^{(fitted)} = [1.02 - 1.17(E_p/E_{ci})^{-0.74}]; \quad \varepsilon_{f} \leq \varepsilon_{max}

\beta_{aT} = \beta_{aT}^{(fitted)} + (a + 8b); \quad \varepsilon_{f} > \varepsilon_{max}

a = 2.7 \times (12.4 - 1.66 \times 10^{-3} \varepsilon_{cT})^{-0.46}; \quad b = 0.83 \exp(-911/\sigma_T) \tag{12}

where, \beta_{aT} is a material parameter depending on the shape of the stress-strain curve; \varepsilon_T is the initial tangent modulus at ambient temperature; \varepsilon_{ci} is the secant modulus at peak stress; \varepsilon_T is the strain at temperature, \varepsilon_{max} is the strain at peak stress at temperature, \varepsilon_{f} and \varepsilon_{cT} are fitting parameters of the stress-strain curve.

The strength of concrete in tension is much lower than its compressive strength; hence, the tensile strength calculations are neglected in code-recommendations both at ambient and elevated temperatures. However, from the point of view of fire resistance, the tensile strength is important due to the development of tensile stresses and progression of micro-cracking, which leads to more severe cases of fire-induced spalling [38,42]. The constitutive relation of concrete under tensile stress at elevated temperature is given by,

\sigma_{ct} = \begin{cases} 
\varepsilon_{cT} E_{ci} & ; \quad \varepsilon_T \leq \varepsilon_{cT} \\
\varepsilon_{cT}^0 E_{ci} & ; \quad \varepsilon_T > \varepsilon_{cT} 
\end{cases}

\tag{13}

where, \varepsilon_{cT} is the tensile strain at the peak stress.

The degradation in the mechanical properties of the reinforcing steel, such as, yield strength and modulus of elasticity is considered vital, affecting the thermal performance of the structural RC frame. For the present investigation, the coefficients of degradation for elastic modulus and yield strength are determined from the Eurocode, EC2-1-2 [23]. The stress-strain curve is assumed bilinear and subsequently, the values of stresses and strains are determined at elevated temperature.

4. Fire loading

The coupled thermo-mechanical analysis is conducted by providing thermal boundary conditions in the form of convection and radiation to the surface of interest in the RC frame. In order to simulate the thermal behavior of the structure, a realistic fire curve is obtained using the recommendations prescribed in Eurocode, EC1-1-2 [22]. The realistic fire curve comprises of two parts: (i) heating phase and (ii) cooling phase. The time-temperature curve for the heating phase is given as,

\[ T = 20 + 1325(1 - 0.324e^{-0.02t} - 0.204e^{-1.75t} - 0.472e^{-8.91t}) \tag{14} \]

where, \( T \) is the temperature obtained during the heating phase, and \( t^* = \Gamma t \) is determined in seconds. The parameter \( \Gamma = (O/b)^2/(0.04/1160)^2 \) depends on opening factor, \( O \) and thermal inertia, \( b \) is given by \( b = \sqrt{pc} \). The duration of the heating phase is given by, \( t_{max} = \max (0.2 \times 10^{-3} \times q/O, t_{lim}) \), where, \( q \) is the fire load density considered in the design depending on the occupancy of the building. According to the code recommendations, \( t_{lim} \) is slow, medium, and fast fire growth rate are 25 min, 15 min, and 10 min, respectively. Likewise, the temperature profile for cooling phase is given as,

\[ T = T_{max} - 625(t - t_{max}^*) \quad \text{for} \quad t_{max}^* \leq 0.5 \]
\[ T = T_{max} - 250(3 - t_{max}^*)(t - t_{max}^*) \quad \text{for} \quad 0.5 \leq t_{max}^* \leq 2 \tag{15} \]

where, \( T_{max}^* = T_{max} + x \times 1 \) for \( t_{max}^* > t_{lim}^* \) and \( x = t_{lim}/t_{max}^* \).

Although the maximum fire temperature is dependent on fire load density and thermal inertia, in the present study, the maximum fire temperature is assumed as a random variable [40], based on which duration of heating phase of fire is calculated from the equation of the parametric fire curve provided in EC1-1-2 [22]. The cooling phase then subsequently follows the peak temperature up to the duration of the fire for which the fire curve was chosen. The developed fire curves are then used for the probabilistic studies of the RC portal frame subjected to thermo-mechanical loading to investigate the performance of the structure.

5. Fragility assessment

Design code provisions have traditionally been relied on deterministic approaches due to their ease in implementation and lack of experimental data. The probability-based approach on the other hand allows catering to the effects of uncertainty to estimate the structural vulnerability for the performance-based design of structure. The main objective of the probabilistic assessment is to provide an insight to the stochasticity for assessing the performance of the structure, which may not be represented effectively otherwise using the deterministic approach. To assess the performance of structures a probability-based approach is presented here, by constructing fragility (F) curves under the fire hazard. The fragility function is thereby derived from the commonly accepted form of Pacific Earthquake Engineering Research (PEER) based performance-based earthquake engineering (PBEE), proposed by Cornell and Krawinkler [17] which is given as,

\[ F(DV) = \int \int p(DV|DM)p(DM|EDP)p(EDP|IM)p(IM)d(IM)d(DP)d(DM) \tag{16} \]

where, \( DV \) is the decision variable (example: loss ratio in terms of repair cost to structural cost); \( DM \) is the damage measure (example: life safety is typically considered in fire engineering); \( EDP \) is the engineering demand parameter (example: repairable or irreparable damage); and \( IM \) is the intensity measure (example: fire duration and fire load density).

The fragility function \( F(DV) \) for damage state, \( DM \), as discussed above, is denoted by \( p(DM|EDP) \), which indicates the structural vulnerability for a given limiting value of the \( EDP \). The structural performance is defined in terms of capacity (strength) of the structure, and the demand (load) on the structure defined in terms of intensity measure (IM). In simpler terms, the development of the fragility curves requires a probabilistic study of the capacity of the RC structure relative to the predefined limit states of damage as well as the demand on the structure exposed to fire.

In this context, Fig. 4 is built to illustrate a flowchart of the stochastic analysis framework developed and used to investigate the performance of the RC frame under the effect of uncertain fire loading. Here, \( \sigma^* \) and \( \mu^* \) respectively are the standard deviation and mean of the uncertain parameters: thermal inertia (\( b \)), fire load density (\( q \)), maximum fire temperature (\( T^* \)), and maximum duration (\( d^* \)).

The present investigation hereby deals with the behavior of the RC portal frame under thermo-mechanical loading, which involves uncertainties only in demand, i.e., fire loading in the structure, despite the presence of other several variables, such as, geometry (concrete cover), structural resistance or capacity (material strength and elastic modulus), heat transfer model (coefficient of expansion, specific heat, and thermal conductivity), and mechanical load (dead and live load). According to the previous research conducted, the most significant parameters influencing the vulnerability of RC element are the geometry and mean fire temperature, which is proportional to the fire load available in the compartment [19,40]. However, researches also indicated that the parameters influencing structural resistance and heat transfer have the least influence on reliability of RC members, although the mechanical load is considered to be moderately influential parameter [19]. From this viewpoint, the current study investigates the performance of the RC portal frame due to uncertainty associated with the
demand (thermal loading). The effect of structural geometry, thereby, is neglected and the subsequent vulnerability analyses are carried out considering the effect of uncertain fire loading exposed to the structure.

5.1. Capacity estimation

The first step in obtaining the fragility curve includes determining structural capacity to investigate the realistic performance of structures in fire. The structural adequacy is obtained through the interaction between the mechanical and thermal forces induced in the structural members. The structural resistance can be defined in terms of the strength of the section, such as, permissible stresses in the members or plastic moment capacity of the members; however, empirical or semi-empirical relations are extremely difficult to develop, as there is non-proportional variation of the temperature over the section. Alternatively, failure in terms of structural capacity can be defined as a limiting deformation or rate of deformation, suited for different structural members. Some of the limit states of failure or engineering demand parameter (EDPs) considered by researchers include, maximum deformation, maximum section temperature, time of failure and maximum bending moment [40]. For the present study, the limit state of failure is defined in terms of limiting deformation in the RC beam, \( L / 66 \) as suggested by Moss et al. [55], where, \( L \) represents the length of the beam.

5.2. Demand estimation

The second step in obtaining the fragility curve includes determining demand on the structure exposed to fire. It is represented by the intensity measure (IM) of the hazard to investigate the vulnerability of the fire-exposed structure. The determination of suitable IMs is not straightforward since the IMs mainly depend on the effect of the considered hazard on the structure. The best adapted IM in fire engineering is the fire load density in a compartment \( (q) \), described by the quantity of fire load in a specific floor area [25]. Other potential IMs include fire duration \( (d) \), peak fire temperature \( (T) \), cumulative radiant heat, heat release rate and normalized heat release rate [55]. Subsequently, the IM has direct interrelating effect on the engineering demand parameter \( (EDP) \) of the structure, which forms a critical parameter to have strong correlation with the damage. Finally, the EDP is required to be correlated with a damage measure \( (DM) \), such as no damage, spalling, repairable, and irreparable, collapse, which is indicative of the direct cost involved. Gernay et al. [25] suggested that it might be impractical and irrelevant to consider all possible sources of variabilities. Therefore, for the present investigation, the demand in terms of IM is represented by thermal inertia \( (b) \), fire load density \( (q) \), maximum fire temperature \( (T) \), and maximum duration \( (d) \) to investigate the vulnerability of the structure exposed to fire.

6. Numerical study

The numerical study comprises of the assessment of a 3-D nonlinear RC portal frame subjected to fire. In this study, the thermal and mechanical properties of the concrete and steel materials at ambient temperature are taken from the standard literature as discussed earlier, and are shown in Tables 2 and 3. The materials, concrete and steel, are modeled using the CDP and metal plasticity models, respectively. The parameters of the CDP model to incorporate the nonlinearity in concrete are presented in Table 4. The mesh of concrete geometry generated for the beam and columns is 100 mm × 100 mm × 100 mm. The optimum mesh is derived after performing a series of trials of mesh convergence tests, as shown in Fig. 5, which indicates no influence in the obtained numerical results on further decreasing the element size. The thermomechanical responses of the RC portal frame are studied in terms of central deflection of the beam and columns, and distribution of temperature in them. The failure of the portal frame is investigated on the probabilistic scale through the development of the fragility curves. The structural and loading parameters defined above are treated as deterministic, which forms a basis of the probabilistic study with assumptions that the random parameters (variabilities) follow certain distributions based on the physical nature of the structural system and the development of fire with respect to time. However, the accuracy of modeling strategy depends on proper rationalization beforehand. Hence, the modeling strategy is justified from due validation of the FE models for the individual RC beam and column, for which experimental results are available in the literature. It may be noted that the input parameters, such as, material and geometrical properties and fire demand curve, used in this study differs from that used in the study to validate the modeling approach. The subsequent sections provide the validation study and the probabilistic assessment to quantify the failure of the structure under the effect of fire loading.

6.1. Validation of finite element (FE) modeling

The responses of beam and column under the effect of thermal and mechanical loadings are individually compared with the results available in the literature to validate the numerical approach considered for modeling. This is mainly due to limited availability of experimental and numerical investigations on the RC portal frame under the effect of thermo-mechanical loading. The FE modeling and thermo-mechanical solution approach adopted herein is validated by comparing the results of the 3-D RC beam and column individually with those reported by Rafi et al. [60,61] and Kodur et al. [45], respectively.

A nonlinear 3-D simply-supported RC beam is modeled in ABAQUS®

![Fig. 5. FE mesh convergence study.](image-url)
with a clear span of 2 m and effective span of 1.85 m in accordance with the experimental and numerical study conducted by Rafi et al. [60,61].

The cross-section of the beam is 120 mm × 200 mm having two rebars of 8 mm φ at top and two rebar of 10 mm φ at bottom as longitudinal reinforcement and stirrups of 6 mm φ with a c/c spacing of 100 mm as transverse reinforcement. The mechanical loading is applied on the beam in the form of two concentrated loads of 15.5 kN at one-third length of the beam from each end. The standard fire curve as per the ISO 834 [34] is used to simulate the fire loading applied at the beam soffit and the adjacent sides. The thermal and mechanical properties of concrete and steel are adapted from the referred literatures. The thermo-mechanical analysis is carried out using the C3D8RT element for

![Fig. 6. Validation of numerical results in the present study with previous experimental investigations.](image)

![Fig. 7. Mechanism of thermo-mechanical behavior in the RC portal frame.](image)
concrete and the T3D2T for rebar. The deflection response at the center of the beam is compared, as shown in Fig. 6(a). On calculating the predicted and measured response, the values are obtained respectively as 65 mm and 65.86 mm, which show an accuracy of more than 95% (precisely 98.69%) in the results obtained by the 3-D FE modeling for thermo-mechanical analysis of the RC member.

Apart from the RC beam, a 3-D FE model of an RC column is also developed in accordance with the experimental study conducted by Kodur et al. [45]. The height of the column is 3810 mm and the cross-section is 305 mm × 305 mm, in which the boundary condition is assumed fixed at both the ends. The column is reinforced with two rebars of 25 mm ϕ on each side and lateral ties of 10 mm ϕ with c/c spacing of 75 mm at top and bottom up to 600 mm from either end and at 145 mm spacing elsewhere. The column is subjected to an axial load of 930 kN at its top. Normal strength concrete (NSC) had been considered in the study, and the thermal and mechanical properties of the concrete and steel rebars are obtained as specified by Kodur et al. [45]. The thermal loading is directly taken from the experimental study, which is represented as average furnace temperature and is applied on all four sides of the column. The thermo-mechanical analysis is carried out using the C3D8RT element for concrete and the T3D2T for rebar. The obtained FE results are in good agreement with the experimental study conducted by Kodur et al. [45], which demonstrates the validity of the modeling technique adopted here.

6.2. Thermo-mechanical behavior of RC portal frame

Fig. 7 indicates the overall behavior of the RC portal frame with fixed column bases at various stages of the thermo-mechanical analysis as observed from the 3-D FE analysis. The portal frame, when exposed to fire along with the mechanical load, undergoes a sequence of mechanical loads resulting in vertical mid-span deflection (δMP) in the beam (transverse in-plane deflection) and an inward rotation at the beam-column joints as shown in Fig. 7(b). On application of initial fire, the mid-span deflection of the beam reduces due to the expansion of the columns, causing the columns to increase in their height. This structural redistribution pulls the beams up to cause an upwards deflection due to the moment restraint of the beam-column connections, as shown in Fig. 7(c). This process of upwards deflection is known as thermal bowing caused due to high thermal inertia of concrete. Subsequently, as the fire propagates through the concrete, causing thermal expansion and degradation of the material strength and stiffness, the applied quasi-static mechanical loading overrides the available restraining force to allow the mid-span deflection to increase once again, shown in Fig. 7(d).

Finally, as the fire loading increases, the temperature of the columns begins to increase causing the columns to rotate outwards. The temperature in the beam also increases thereby further reducing the stiffness with degradation of material strength and increasing the mid-span deflection of the beam.

7. Results and discussion on deterministic and stochastic studies

A numerical study of the RC portal frame is conducted here under the fire-loading scenario considering material and geometric nonlinearity of the structure through a probabilistic approach. A parametric study on the structural capacity is conducted to obtain the responses for the RC portal frame with varying characteristic strength of concrete and steel. The compressive strength for concrete used in the RC portal frame at ambient temperature is taken as 20, 30, and 40 MPa (Table 2); whereas, the yield strength of the steel reinforcing bars are taken as 415 and 500 MPa (Table 3).

In order to evaluate the vulnerability of the RC frame under fire, the possible uncertainties related with the performance of the system are required to be considered explicitly in the assessment. The demand on the system depends on random parameters, such as fire development in different phases (fire load density, maximum temperature attained in fire, and exposure duration), heat transfer process, and thermal and mechanical properties of the materials. In the present study, the aleatory or the system-based randomness are assumed as the fire load density, thermal inertia, maximum temperature in fire, and fire duration. Since, the current investigation does not involve the structural as well as heat transfer properties, the parameters influencing the fire loading are all taken into account to study the vulnerability of the RC portal frame under fire. Parametric studies are conducted for the capacity of the system, i.e., by varying the mechanical properties of the materials. Table 5 also shows a detailed description of the parameters used for the fire model, the heat transfer model, and the structural model to indicate

| Table 5 | Deterministic and stochastic parameters used for the thermo-mechanical analysis. |
|---|---|---|---|---|
| Parameters | Unit | Distribution | Mean Values | COV |
| Material properties (Concrete and Steel) | Strength of concrete/steel (fck/fc) | MPa | Deterministic | Table 1 |
| | Elastic modulus of concrete/steel (Ec/Ey) | | | |
| Geometric properties | Size of beam | m | Deterministic | Fig. 2a |
| | Size of column | | | |
| Mechanical loading | Beam surface load kN/m | Deterministic | Fig. 2a |
| | Column axial load | | | |
| Thermal properties (Concrete and Steel) | Density (ρ) | kg/m³ | Deterministic | Tables 1 and 2 |
| | Conductivity (λ) | W/(m·°C) | | |
| | Specific heat (c) | J/(kg·°C) | | |
| | Coefficient of expansion (α) | °C | | |
| Fire loading | Thermal inertia (θb) | J m⁻²·°C⁻¹·s⁻¹/² | Normal [31,41,53] | 2000 | 0.2 |
| | Fire load density (ϕ) | MJ/m² | | 420 | 0.2 |
| | Maximum temperature (T) | °C | | 800 | 0.2 |
| | Duration of fire loading (d) | hour | Deterministic | 3 | 0.2 |
| | Opening factor (O) | m¹/² | | | |
| | Fire growth (lim(θb)) | min | | | |
| Heat transfer | Stefan-Boltzmann Constant | W/(m²·°C) | Deterministic | 5.67 × 10⁻⁸ | – |
| | Convection (exposed) | W/(m²·°C) | | | |
| | Convection (unexposed) | | | | |
| | Radiation/emissivity | | | 0.8 | – |
the parameters having sources of uncertainty. Normal distribution is assumed to generate 1000 random samples for incorporating the stochasticity in the fire-loading scenario using MC simulation. Based on the literature, the number of simulations was assumed 1000 for obtaining statistically significant results [28,65]. The subsequent limit state for limiting deformation in the beam is obtained as 45 mm from the suggested empirical model given by [55] as \( L/66 \), where \( L = 3000 \) mm. Thereafter, probability density function (PDF) and fragility curves are obtained to quantify probability of failure (\( p_f \)) for the RC portal frame under fire.

In the present study, one-at-a-time (OAT) approach for sensitivity analysis is used to identify the parameter influencing the vulnerability of the RC structure and determine the uncertainty in the system. The method is a local approach that gives elementary effects and variance for the input parameters helping to eliminate the less influential parameters. The OAT method is the simplest approach to conduct the sensitivity analysis, where the factorial design is easy to conceptualize. As the current model does not deal with much complex effect in determining the responses of the structure, the OAT method is more suited to conduct the sensitivity analysis of this RC portal frame exposed to fire.

Fig. 8 shows the deflection profiles in the beam and column for various grades of concrete with steel reinforcement of Fe415. It is observed from Fig. 8 that the RC frame analyzed with the lowest concrete grade, M20, undergoes the highest deformations in beam and column, as the flexural rigidity and load carrying capacity of this frame is evidently less as compared the frames made of M30 and M40 grades of concrete. The central deflection as well as the deformation throughout the length of the beam and column is particularly studied to investigate the nature of the responses under the fire exposure. The central deflection curves for the beam and column with respect to time show that the frame with M20 grade concrete experienced the highest deflection. For the column, initial eccentricity generated by the axial loading as well as the beam-column joint assembly plays a crucial part in the lateral deflection experienced. The effects of thermal bowing and mechanical loading are noticeable from the response obtained for the RC frame under the simultaneous mechanical and temperature loading. After 3-h of fire exposure, the beam and column with M20 grade of concrete is observed to have the highest deflection. On increasing the grade of concrete, the deflections in beam respectively reduce by \( \sim 33\% \) and \( 42\% \), whereas the deflections in beam respectively reduce by \( \sim 38\% \) and \( 50\% \) for M30 and M40 grades. Similarly, the central deflections also reduce by \( \sim 34\% \) and \( 42\% \) for the beam and \( \sim 39\% \) and \( 43\% \) for the

![Fig. 8. Deflection profiles in the RC beam and column for different grades of concrete and Fe415.](image)

![Fig. 9. Time-temperature response at different beam and column sections.](image)
Fig. 10. Probability density function plots considering parametric variation in capacity of the structure following lognormal distribution.

Fig. 11. Quantile-quantile plots of the beam responses to test for lognormal distribution against normal distribution for uncertain fire load density.
columns for M30 and M40 grades, respectively.

Fig. 9 shows the variation in temperature distribution with time at different interfaces of the beam and column. The peak temperature near the exposed face of the beam and column is around $800 \, ^\circ \mathrm{C}$ and $700 \, ^\circ \mathrm{C}$; whereas, the peak temperature at the unexposed faces are less than $50 \, ^\circ \mathrm{C}$. Such large thermal gradient under fire exposure between the exposed and unexposed faces of the RC beam and column is attributed to the high thermal inertia of the concrete. It is noticed that steep thermal gradients are developed within the concrete section due to relatively high thermal inertia of concrete, which keeps the temperature within the beam and column cross-section below $600 \, ^\circ \mathrm{C}$ to $650 \, ^\circ \mathrm{C}$. As a result, high thermal expansion occurred near the exposed face of the RC beam as compared to the unexposed face, causing its upward thermal bowing. However, due to the quasi-static mechanical loading applied transversely on the beam and gradual increase in the temperature, there is a degradation in the material properties, which overcomes the thermal bowing to cause downward vertical deflection in beam.

Fig. 10 shows the probability density function (PDF) plots for the failure responses obtained when the demand exceeds the capacity of the structure, refer flowchart of the stochastic framework in Fig. 4. Only one random parameter is varied at a time, whereas the other parameters are considered deterministic to: $b = 2000 \, \text{J m}^{-2} \cdot \text{C}^{-1} \cdot \text{s}^{-1/2}$; $q = 420 \, \text{MJ/m}^2$; $T = 800 \, ^\circ \mathrm{C}$; and $d = 3 \, \text{h}$ from case to case. Although the input variables have been assumed to have normal distribution, the plots of the PDF for the failure responses follow lognormal distribution, which is shown from the quantile–quantile plots constructed and shown in Fig. 11. Moreover, the plots show the range of the responses obtained for the parametric variation in the structural properties considering the uncertainty in the fire loading. The failure probability density $f(\cdot)$ against the maximum transverse in-plane deflection response of the RC beam ($\delta_{\text{bt}}$) shows wide spread in mean values for the variation in the fire load density ($q$) as compared to the uncertainty in the thermal inertia, $b$, as shown in Fig. 12. The distribution and spread of the responses show that the randomness in the fire load density has a strong influence on the development of the fire and consequently structural response. This parameter is observed to be extremely crucial in identification and development of design fire for the performance-based design of structures under fire loading.

The transverse in-plane deflection in the beam is observed to be much larger for variation in the thermal inertia ($b$), although higher thermal inertia caused high thermal bowing. The increased vulnerability in terms of temperature-dependent deflection response is mainly attributed to the faster material degradation under the combined thermal and mechanical loads. The PDFs also show that the mean of the responses for the lowest material strength are relatively higher for the considered uncertainties. Hence, the uncertainty in the fire loading and thermal properties of the material has significant effect on the thermo-mechanical responses, which requires due attention in developing guidelines for the performance-based design of structures under fire.
the M40 grade of concrete signifies its reduced fire vulnerability. The steepness of the fragility curves for the structure with concrete strengths (in IS 456:2000 [33], nominal concrete cover thickness is prescribed). Grades of concrete. However, according to the prescriptive methodology, the fire resistance is obtained from the size of the member rather than the strength of the member, which is more conservative (in IS 456:2000 [33], nominal concrete cover thickness is prescribed). Hence, in the performance-based fire design, it is recommended to take into account the thermal mass (depending on specific heat, thermal conductivity, and density of material) of the structural members, which has significant influence on the failure behavior of the concrete members.

Fig. 14 shows the probability of failure ($p_f$) for the RC frame considering uncertainty in the fire load density ($q$), whereas $b = 2000 \text{ J m}^{-2} \text{C}^{-1} \text{s}^{-1/2}$, $T = 800 \, ^\circ\text{C}$, and $d = 3$-h are kept deterministic at their mean values. The uncertain fire load density has significant influence in determining the failure probability of the RC frame with varied structural capacity, as observed from the plots. The failure probability is the highest corresponding to the lowest strength of the material properties used in the RC frame; whereas, the lower failure probability exists for the higher material strengths. Moreover, quite interestingly, at higher fire load density (550 MJ/m$^2$), the failure probability of the RC frame with concrete and steel of M20 and Fe415 grades is lower than the frame with M30 and Fe500 grades. Considering the deterministic scenario (for office building, $q = 420$ MJ/m$^2$), the probability of failure has significantly decreased with increased material strengths, and negligible failure probability is observed for the highest material strengths of M40 and Fe500 considered in this study. According to the code provisions, the determination of fire resistance or fire rating is based on a standard fire curve, which has no directly established influence on the fire load density that affects the temperature growth. This practice overestimates the design parameters of the structures required for fire safety and necessitates reconsideration to achieve safe as well as economic design guidelines. Hence, the importance of considering uncertainty in the fire load density to determine the failure probability and subsequently leading to estimate the fire resistance of the RC members is deemed important. Such probabilistic assessments will help in incorporating necessary amendments in the development of the performance-based design guidelines for structures in fire.

Fig. 15 shows the probability of failure ($p_f$) for the RC frame with stochasticity considered in the maximum fire temperature ($T$) attained during the process, whereas $b = 2000 \text{ J m}^{-2} \text{C}^{-1} \text{s}^{-1/2}$, $q = 420$ MJ/m$^2$; $d = 3$-h are kept deterministic at their mean values. The probability of failure decreases considerably on increasing the material strength in the RC frame with $T$ as an IM. This is because, under a constant fire severity, the capacity of the structure is more corresponding to the higher material strengths, which restrains the excessive deformations in the structure under the fire loading. As a result, the performance of the structure is seen to be directly related to the severity of the fire, for which the existing code provisions does not directly address such interrelation of fire rating with fire severity. Nevertheless, scenarios having higher fire temperature may induce extensive failure in the structures constructed with lower material strengths. Moreover, the uncertainty in the fire temperature has significant influence on the failure probability as observed from the spacing between the curves corresponding to the variation in material properties. At the deterministic scenario of $T = 800 \, ^\circ\text{C}$, the failure probability is almost negligible in case of all the grades of materials used for the RC frame here. This is possibly due to assumption of the high opening factor, for which the natural fire curve has fast cooling, providing reduced fire severity. However, the situation might change for some lower opening factor triggering higher probability of failure at lower fire temperature as well, which shall induce higher fire severity in the structure. Hence, it can be concluded that the probabilistic tools utilize the effect of uncertainty to incorporate the fire severity in defining the structural fire resistance for better and safe performance of structures under random fire loading.

Fig. 16 shows the failure probability ($p_f$) of the RC portal frame for the randomness considered in the duration ($d$) of fire exposure, whereas $b = 2000 \text{ J m}^{-2} \text{C}^{-1} \text{s}^{-1/2}$, $q = 420$ MJ/m$^2$; $T = 800 \, ^\circ\text{C}$ are kept deterministic at their mean values. It is observed that the failure probability is insignificantly influenced by the strength in steel, however the failure is mainly governed by the concrete strength when uncertain fire exposure duration is considered. Nevertheless, higher material strengths have evidently exhibited lower as well as delayed failure probability under varied fire exposure duration. Considering the deterministic case, a fire loading of 3-h duration has negligible effect on the failure probability of the structure with the highest material strengths M40 and Fe500; whereas, a substantial failure probability is observed for the structure with the lowest material strengths M20 and Fe415. Thus, the duration of fire loading is crucial in the design of the RC frame in view of their material strengths. This observation is important to estimate the fire rating of the structural members considering the effect of material properties used in the construction practices. In this context, the present stochastic framework has been effective in quantifying the failure probability ($p_f$) that accounts for different sources of uncertainty affecting the fire vulnerability of an RC portal frame. The developed fragility curves are beneficial to predict the structural damage to further benefit the building community for assessing the resultant thermo-mechanical responses under the probabilistic fire disaster, which should pave a way for the futuristic performance-based design of structure. Finally, the developed stochastic framework is also suitable to...
8. Conclusions

A probabilistic analysis framework is developed to identify the parameters strongly influencing the fire resistance of the reinforced concrete (RC) portal frame and to quantify its vulnerability under exposure to fire. For this purpose, a three-dimensional (3-D) nonlinear finite element (FE) model is developed for the RC frame to conduct transient thermo-mechanical analysis numerically to investigate the influence of uncertainties due to the temperature-dependent material parameters and fire loading. The proposed FE modeling strategy is effective in simulating the structural behavior and obtaining the thermo-mechanical response of the RC portal frame subjected to fire loading. On calculating the predicted and measured response of the beam, the values of transverse displacement are obtained as 65 mm and 65.86 mm, which show an accuracy of more than 95% (precisely 98.69%) in the results obtained by the 3-D FE modeling for thermo-mechanical analysis of the RC member. Furthermore, the obtained FE results for the column are also in good agreement with the experimental study reported in literature, which demonstrates the validity of the modeling technique adopted here for the beam and column.

The central deflection curves for the beam and column with varying strength of the members show that the frame with the lowest grade concrete experienced the highest deflection. The reduction in fire response for the member with M30 grade of concrete is observed to be 31% as compared to M20 grade, whereas the reduction in the same response upon using M40 grade concrete is observed to be around 60%. This reduction in the response shows that the fire resistance is dramatically changed and is dependent on the strength of the members. Moreover, considering the threshold limit of failure in beam deflection, the fire resistance of the beam with M20 grade of concrete is obtained as 2.25-\(h\), which is the lowest in comparison to the other higher grades. This is attributed to the thermal and mechanical material properties of the structural members, which have significant influence on the failure of the RC frame. However, according to the prescriptive methodology, the fire resistance is evaluated from the size of the member rather than the strength of the member. Hence, the fire-resistant design of the RC structures has been recommended to be based on the strength of the member.

The probability density \(f()\) of the maximum transverse in-plane deflection of the RC beam shows wider spread in mean values for the variation in the fire load density as compared to the uncertainty in the thermal inertia. The difference of the highest and lowest mean values observed show that the variation due to the fire load density is around 28% as compared to 20% for the thermal inertia. The distribution and spread of the responses show that the randomness in the fire load density has a strong influence on the development of the fire and consequently structural response. As the fire load density is the most random quantity having a strong influence on the development of fire, this parameter has emerged extremely crucial in identification and development of design fire for the performance-based design of structures under fire.

The uncertain fire load density has significant influence in determining the failure probability of the RC frame with varied structural capacity. The structural vulnerability starts at a higher fire load density, i.e., 450 MJ/m\(^2\) for the structure with M40 grade of concrete as compared to the 200 MJ/m\(^2\) for the M20 grade of concrete. Increase in the grade of concrete significantly reduces the probability of failure of the RC structure in fire as compared to the increase in the grade of the reinforcing steel. Lastly, the maximum fire temperature attained due to the combustion of the materials induces the structural failure from almost 1000\(^\circ\)C. Therefore, based on the probabilistic studies carried out for uncertainty due to the maximum temperature in fire loading, the maximum fire temperature allowed for the structure should not exceed 1000\(^\circ\)C. Finally, the framework and the obtained results should provide a benchmark for fire safety design in India, wherein the current guidelines are based on prescriptive and conservative deterministic approach.

**Declaration of Competing Interest**

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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