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Fire fragility of reinforced concrete panels under transverse out-of-plane and compressive in-plane loads

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ABSTRACT

This article focuses on investigating vulnerability of fire-exposed reinforced concrete (RC) panels under transverse out-of-plane and compressive in-plane loads following a probabilistic methodology. The three-dimensional (3-D) RC panels are modeled by considering material and geometric nonlinearity and subsequent transient thermo-mechanical analysis is conducted for single-side exposure to fire loading. Uncertainties in the system are considered in structural capacity (material strength), heat transfer-, and fire model to study the stochastic thermo-mechanical response of the RC panels. The stochastic responses are studied in terms of central deflection, distribution of temperature, and variation in stresses and strains developed at the exposed and unexposed faces of the RC panels. It is concluded that the system uncertainty has significant influence on the duration of fire resistance of the RC panels under the considered thermo-mechanical loadings. The RC panels under the transverse out-of-plane loading exposed to fire are relatively more vulnerable as compared to the panels under the compressive in-plane loading. Moreover, the thicknesses of the RC panels have significant influence in determining the fire rating. Finally, it is recommended to use RC panels of at least 125 mm thickness irrespective of functionality if the desired code-specified fire rating is to be achieved.

1. Introduction

Regardless of technological progress in the concrete industry, severe fire threats posed to reinforced concrete (RC) structures has continued to shatter economic developments, causing devastating effects across the globe [1]. Although concrete structures have shown excellent performance under fire loading, one of the reasons for the excessive damage of structure is horizontal and vertical spread of fire through structural walls and slabs, which are unable to restrict fire outspread and maintain required structural stability. Hence, RC wall and slab panels must explicitly satisfy three criteria to achieve the required fire resistance, which include stability, insulation, and integrity [2]. Stability criterion refers to the ability to carry and possess adequate load-bearing capacity of the structural members under fire exposure for desired duration; whereas, insulation criterion refers to the ability to limit the rise in temperature beyond ignition point at unexposed side of the panels. Finally, the integrity criterion denotes the ability to provide compartmentation without developing cracks. The above-mentioned fire resistance criteria, according to the current design guidelines, are generally achieved from a prescribed reinforcement cover for a given structural member size, aggregate type [3], and the subsequent exposure category such as, mild, moderate, severe, very severe, and extreme [4], very dry (X0), dry (XC1), wet and rarely dry (XC3), etc. ([56], 5]). However, the existing prescriptive design approaches do not necessarily consider the effect of several critical parameters affecting the thermal behavior, which eventually lead to practically over-conservative and uneconomic structural design [6]. In reality, actual nature of fire loading is difficult to predict due to complexities in each stage of fire development. For instance, temperature rise in a compartment during an actual fire event depends on the amount, distribution, and composition of combustible materials in the compartment along with dimensions and ventilation of the compartment [7]. As a result, significant amount of uncertainty remains in the system, which demands appropriate understanding to predict the fire performance of the structures. In this regard, probabilistic techniques have the potential to explicitly capture the uncertainties involved in such extreme scenario providing safe and economic designs for fire-resistant structures [8,9]. Hence, for achieving this goal, the required framework should utilize the probabilistic approach to evaluate the parameters affecting structural fire resistance while implementing advanced performance-based design concepts in

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fire-resistant structures [10].

RC slab and wall panels are being widely employed in building systems, especially in residential and commercial buildings due to multiple functions, such as, load resistance ability and partitioning the space in both vertical and horizontal directions [11–13]. From this functional viewpoint, behavior of such RC panels exposed to fire has been a subject of diversified research since the last couple of decades. RC slab and wall panels, i.e., the panels under transverse out-of-plane and compressive in-plane mechanical loads, respectively are typically designed assuming a single-side fire exposure (compartment fire) and most of the research until the date, although limited, is based on this assumption. Initially, numerical techniques were proposed to predict structural response of RC panels under the transverse out-of-plane loading (slab), when exposed to fire considering the complexities associated in the structural behavior [14–16]. The results showed that RC slab panels had excellent fire resistance when the panels develop tensile membrane action while deforming in double curvature. Gradually, with time, use of innovative materials, such as fiber-reinforced polymers (FRPs), in structural applications, has increased owing to high strength, resistance to electrochemical corrosion-, and costs associated with the construction. In this regard, studies have been conducted to assess fire endurance of the RC slab panels reinforced with the FRP bars under transverse out-of-plane loading [17–20]. The studies indicate that the heat transfer behavior of the RC slabs, reinforced with the FRPs, was similar to that of the steel-reinforced RC slabs. Moreover, thickness of concrete cover and the type of reinforcement played key roles in influencing the fire resistance of the FRP-reinforced concrete slabs. Similarly, investigations on fire behavior of precast concrete structural panels gained prominence for further utilizing the high-strength property of the FRP bars, although such structural panels have demonstrated satisfactory performance in fire [21,22]. It was shown that fire-induced spalling governs the fire resistance of the structural members, which can be prevented by using high dosage of micro-fibers. Although, RC slab panels are designed to withstand transverse out-of-plane loads, research has also been conducted to investigate fire behavior of the RC slab panels under combined biaxial in-plane and out-of-plane loads [23].

The fire endurance of compressive in-plane loaded RC wall panels was similarly studied to understand their behavior when exposed to fire. Initial researches focused on modeling techniques to investigate the behavior of the RC wall panels subjected to compressive in-plane mechanical and fire loads [24–27]. These studies demonstrated significant coupling effects of heat and mass transfer mechanisms involved in concrete. The influence of different structural parameters on fire rating was also observed in terms of load-bearing capacity, structural integrity, and thermal insulation. Moreover, results also showed that flexural cracking on the fire-exposed surface minimized the likelihood of spalling in concrete. Thereafter, few experimental investigations were carried out to examine out-of-plane behavior and stability of the RC panels under compressive in-plane loading (wall). Go et al. [28] conducted experimental studies on the RC wall panels with lightweight aggregates exposed to fire. They showed that the wall panels with lightweight aggregates have excellent fire-resistant properties as compared to the panels with normal weight aggregates. Lee et al. [29] studied the thermo-mechanical response of the RC walls exposed to fire on all sides. They concluded that the RC walls with shorter curing periods and high-strength concrete were more vulnerable to the loss of fire resistance. Ngo et al. [30] and Mueller et al. [31] studied the thermal behavior of the RC walls and concluded that the RC structural walls were able to withstand a long duration of fire exposure.

Recently, studies were also conducted on the modeling strategy to study the effect of moisture content on thermal behavior of the RC wall panels subjected to fire loadings [32]. The modeling technique proposed incorporated a moisture-clog zone, which was found effective in predicting the behavior of the concrete walls under fire. Studies based on the numerical and experimental investigations provided some impetus to investigate the fire endurance of the RC wall panels, which was deemed necessary to achieve structural fire safety. Despite advanced modeling techniques demonstrated by various researchers, the prescriptive design concepts used in obtaining the thermo-mechanical responses have been ineffective for reliable structural design of the RC members subjected to fire, as behavior of these RC members against fire involves significant complexities directly or indirectly depending on the potential fire scenarios. This serious issue demands implementation of stochastic design approaches, which would consider the effect of uncertainties to obtain desired performance of the RC panels subjected to fire.

While few researches are available contributing to the probabilistic aspect of performance of other structural RC panels, such as, beams and columns under the fire loading [33,34], research is found exceptionally scarce in the application of stochastic methods to assess the vulnerability of the RC panels under transverse out-of-plane and compressive in-plane loads exposed to fire scenarios. Recent studies are observed in probabilistic domain to incorporate performance-based design (PBD) philosophies in structural fire engineering. Lately Van Coile and Bisby [35] used deflection-based criteria to demonstrate the performance of the RC slabs under thermo-mechanical conditions. In this study, they suggested that assumption of lognormal distribution was inappropriate for structural fire design. Moreover, Heidari et al. [36] conducted parametric studies using simple probabilistic methodology to determine the resistance of a simply-supported RC slab using Eurocode parametric fire curve. The study was helpful to identify the uncertainties in terms of critical parameters of the design fire curves for practicing engineers and designers. Regardless of the extent of uncertainties present in the system that induce high sensitivity in the response of the structure [37], the fire resistance criteria, defined in terms of fire rating is based on a deterministic assessment assuming life-safety as the sole design parameter [38]. The deterministic methodology, deduced from the empirical models based on the standard fire tests, has negligible emphasis on the uncertainty in governing system parameters, which largely affects the performance of members under fire [39–41]. Hence, the shift towards probabilistic domain is vital to recognize the impact of uncertainties in evaluating the performance of the structural systems exposed to fire for developing a more reliable performance-based design methodology [38, 42].

Herein, finite element (FE) models of three-dimensional (3-D) reinforced concrete (RC) panels are developed in ABAQUS®-Python environment. The stochastic simulations are carried out by scripting in Python programming interface linked with ABAQUS® [54] solver. The responses obtained from the FE study are assessed with experimental results available from the literature. Probabilistic assessment is conducted by constructing fragility curves considering variabilities in material strength, heat transfer model, and demand (fire model) of the system. In this context, objectives of the current investigation are: (i) to study the variation in responses for the RC panels under the considered stochastic environment, (ii) to study the effect of uncertainty in the input parameters for the RC panels, and (iii) to develop fire fragility curves for the RC panels independently under the effect of compressive in-plane and transverse out-of-plane mechanical loads.

2. Finite element (FE) modeling and analysis of RC panels

2.1. Structural detailing

Three dimensional (3-D) RC panels of different thicknesses exposed to fire are investigated under transverse out-of-plane and compressive in-plane mechanical loads, respectively to obtain the thermo-mechanical responses of the RC panels. The RC panels (both slab and wall) used for the study have same planar dimensions of 6.1 m × 4.3 m with varying thicknesses and are assumed to be located at the third floor (uppermost storey) of height 4.3 m in a 3-storey building (Fig. 1) located in New Delhi, India. The RC panel under the transverse out-of-plane load represents a typical floor slab subjected to flexural load transverse to the plane-of-action. Similarly, the RC panel under the compressive in-plane load illustrates a typical load-bearing wall subjected to axial load at certain eccentricity, which is a typical representation of accidental eccentricity during application of service load on the RC wall. Since, planar dimensions of the slab and wall are assumed same, the structural details in terms of size and reinforcement details, and the finite element (FE) modeling technique adopted are carefully described using 'RC panels'. The RC structural panels of different thicknesses are embedded with a single layer of steel reinforcement in concrete having clear cover of 25 mm on the tension side. Along the longitudinal direction, the RC panels, irrespective of loading conditions, are reinforced with 12 mm ϕ steel rebar spaced at 428 mm center-to-center (c/c) on the tension side of the panels, where ϕ represents diameter of the rebar in mm. Similarly, along the transverse direction, the RC panels are reinforced with 12 mm ϕ rebar spaced at 300 mm c/c on both sides, as shown in Fig. 2. Even though the adopted reinforcement detailing is resulting in smaller amount of reinforcement along shorter span of the RC slab, it is still adopted to compare responses with that of the RC wall panels. However, it was made sure that the slab panel is safe against the action of out-of-plane loads. The mechanical loads applied on the RC panels are calculated from the dead load (DL) and live load (LL), as per Indian standard [43] based on the functionality viewpoint. The mechanical load acting on the RC panel under transverse out-of-plane load comprises of its individual dead and live loads, whereas, the mechanical load acting on the panel under the compressive in-plane load is treated as axial force, which is calculated from the dead loads of beam and slab, and imposed load on slab. The mechanical loads on the panels of different thickness and the accidental eccentricity assumed for the RC wall panel under the compressive in-plane load are provided in Table 1. Both the RC panels are designed for seismic actions pertinent to the site located in New Delhi, India, which is based on the assumption that the structural panels are capable of sustaining inelastic deformation and providing energy dissipation under the lateral seismic load. However, it is assumed that the RC panels are not specifically designed to resist any accidental fire loads, nor any fire resisting material is applied on the panels; except that the code-prescribed concrete cover is provided.

2.2. Finite element (FE) modeling of RC panels

Finite element (FE) modeling of the RC panels under compressive in-plane and transverse out-of-plane loads, respectively is carried out by developing a ABAQUS®-Python script to predict the behavior of the RC panels exposed to fire duly taking into account the effect of material and geometric nonlinearity. The nonlinearity in concrete is defined using concrete damaged plasticity (CDP) model, as the CDP model takes into account the degradation of the elastic stiffness in both tension and compression. The CDP model assumes two key failure mechanisms in concrete: tensile cracking and compressive crushing. In the current modeling strategy, uniaxial tensile and compressive behavior is considered in the damaged plasticity model. The tensile and compressive behavior of concrete is characterized by respective stresses and cracking/inelastic strains to represent complete inelastic behavior of concrete. The parameters of the CDP model, i.e., dilation angle (ψ), eccentricity (e), ratio of the equi-biaxial compressive to the uniaxial compressive strength of concrete (f_{b0}/f_{co}), ratio of the second invariant of the deviatoric stress tensor (K_c), and viscosity (μ_e) are presented in

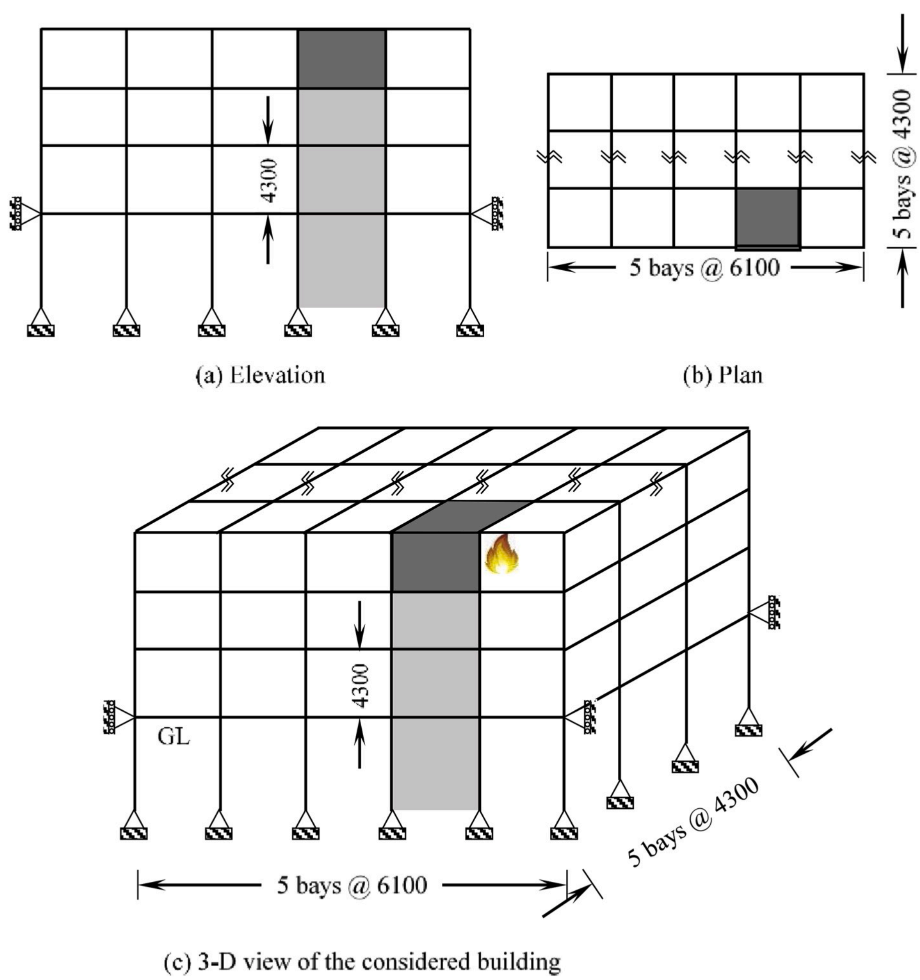

Fig. 1. Elevation, plan, and 3-D view of the 3-storey RC building.

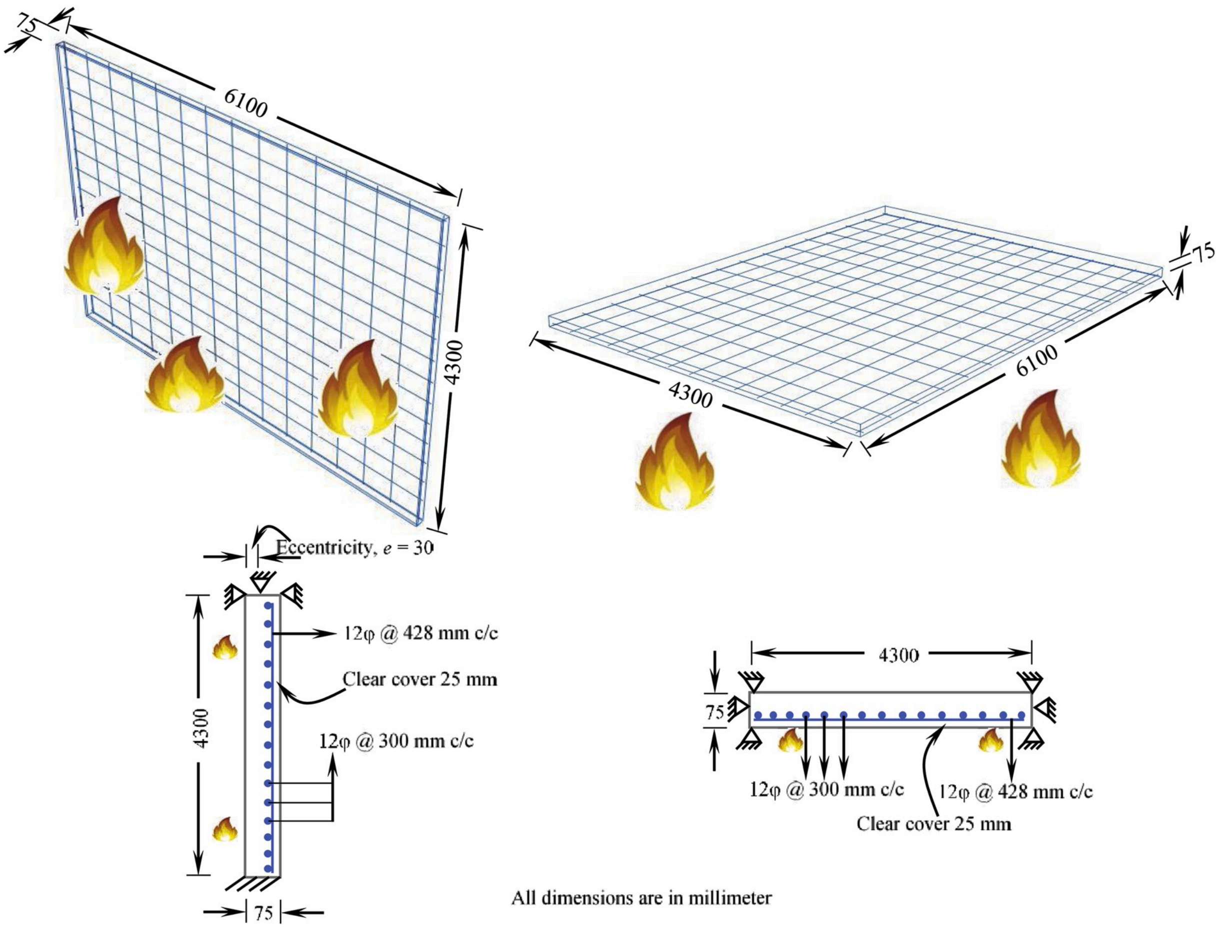

Fig. 2. 3-D view showing fire exposure and sectional cross-section view showing reinforcement details of the RC panels.

Table 1
Accidental eccentricity and mechanical load assumed for the analysis.

RC Panel	Thickness (mm)	Accidental Eccentricity (mm)	Mechanical Load (kN/m^2)	
			Dead Load	Live Load
Transverse out-of-plane loaded member	75	–	1.875	2
	100		2.5	
	125		3.125	
Compressive in-plane loaded member	75	30 mm towards the direction of fire exposure	107.5	320
	100			
	125			

Table 2. The steel reinforcement used to transmit axial forces is modeled by one-dimensional (1-D) element using the classical metal plasticity involving von-Mises yield criterion with associated plastic flow and isotropic hardening. The plasticity in the model is simulated by providing yield stress for corresponding uniaxial plastic strain. The meshing of concrete geometry is carried out by 8-node trilinear continuum C3D8RT element with reduced integration and hourglass control. Each node of the 3-D continuum element has four degrees of freedom (DOFs) such as, translational DOFs (1, 2, 3) in three mutually perpendicular axes, x, y, z, and a temperature DOF (11); where x and y directions are in-plane, whereas z direction is normal to the surface of the RC panels. The mesh of steel rebar geometry is lumped by 2-node linear truss element T3D2T to introduce temperature DOF for the thermo-mechanical analysis. The concrete and steel are assembled with TIE constraint feature mainly to constrain the nodes of the concrete and steel rebar for allowing the proper distribution of temperature from concrete to steel. Boundary conditions of the RC panels are considered based on a realistic scenario depending on the position of the RC panels under the transverse and compressive loading. The RC slab panel under the transverse out-of-plane loading is modeled considering simply-supported condition on all sides; whereas, the RC wall panel under the compressive in-plane loading is modeled considering fixed boundary condition (BC) at the bottom and pinned on other three sides allowing the necessary rotations. The BCs assumed for the RC panels are shown in Table 3.

The geometric nonlinearity is modeled using NLGEOM option coded in the ABAQUS®-Python script because large displacement is expected from the thermo-mechanical behavior, which is directly associated with geometric nonlinearity. Thermal loading on the structural panels is applied as convection and radiation boundary conditions at the desired surface of exposure, as shown in Fig. 2. A convective heat transfer

Table 2
Parameters used for the concrete damaged plasticity (CDP) model.

Dilation Angle (ψ)	Eccentricity (e)	f_{b0}/f_{co}	K_c	Viscosity (μ_e)
36.31	0.1	1.16	0.667	0

Table 3
Boundary conditions of the RC slab and wall panels.

Panel Member	Boundary Conditions
Transverse out-of-plane loaded member	Simply-supported on all sides
Compressive in-plane loaded member	Fixed at bottom and pinned (rotation allowed) on other three sides

coefficient in the form of surface film condition is applied at the exposed face of the RC panels. It is assumed that heat transfer through the unexposed face is minimal, and therefore is ignored in the present study. The radiation boundary condition, given in terms of emissivity coefficient, is applied only at the exposed face of interest in the RC slabs and walls.

Coupled temperature-displacement procedure is used to conduct the thermo-mechanical analysis, which involves nonlinear calculation to simultaneously solve nodal displacements and temperatures. In this analysis procedure, effect of temperature on the displacement is calculated by thermal expansion and radiation coefficient. The entire RC panel under each type of mechanical loading scenario is subjected to an ambient temperature of 20°C in the form of predefined initial condition. The absolute zero temperature was set by providing −273.15°C because ABAQUS® [54] solver uses temperature in degree Celsius (°C). The thermo-mechanical analysis is carried out for a duration of 120 min and the response at each node is stored for post-processing. The thermo-mechanical analysis of the RC panels after exposure to mechanical and fire loading is carried out considering the degradation in the mechanical properties of the concrete and steel rebar. It is assumed that heat transfer into the wall is through thermal convection and radiation mechanisms, and within the wall by thermal conduction process. Finally, the mesh size is determined by convergence trials to ensure no influence in the obtained numerical results on further decreasing the element size.

2.3. Numerical solution approach for thermo-mechanical analysis

The numerical solution approach and solution scheme is conducted in a single step, viz., coupled displacement-temperature analysis in which the action of temperature on displacements and vice-versa is taken into account. ABAQUS®/Standard solver uses backward finite difference method (FDM) through Newton-Raphson iteration for solving both displacement and temperature at every increment at each node. The solution scheme used to assess the RC panels under the thermo-mechanical loading is discussed hereunder.

The thermo-mechanical analysis of the RC panels under temperature loading is performed using ABAQUS®-Python script in standard solver, it being more efficient for solving such smooth incremental nonlinear problems. In the coupled temperature-displacement analysis for the RC panels, the heat is transferred to the surface of the RC wall by means of convection and radiation mechanisms; whereas, the heat is transferred within the elements of the RC wall by heat conduction mechanism. The transient heat conduction equation along with the boundary condition and the convection condition can be expressed as,

$$Q = \rho c \frac{\partial T}{\partial t} - \nabla \cdot (\lambda \nabla T) \tag{1}$$

$$(-\lambda \nabla T) \cdot n = h_c (T_f - T_c) + \sigma \varepsilon (T_f{}^4 - T_c{}^4) \tag{2}$$

where, Q is the overall heat transferred from outside, ρ is the density of material (kg/m^3), c and λ are the specific heat capacity, J/(kg·K) and thermal conductivity, W/(m·K), respectively, n is the unit vector outer normal to the boundary, h_c is the convective heat transfer coefficient, T_f and T_c are the ambient temperature and temperature at reference surface in Kelvin (K), respectively, σ is the Stefan-Boltzmann constant in W/(m^2·K^4), and ε is the emissivity coefficient.

The implicit nonlinear FE solution technique uses backward finite difference algorithm to integrate governing equations in the time domain. The thermal field is solved at each time step Δt, and subsequently the mechanical problem is solved using the obtained thermal strains. The tolerances on residual errors need to be satisfied in every iteration before proceeding to the next time step. 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. The technique requires several global equilibrium iterations, which can be written as,

$$\left[K'^{\,t'+\Delta t'}_{\,i-1}\right]\left\{\Delta u'^{\,t'+\Delta t'}_{\,i-1}\right\} = \left\{R^{t'+\Delta t'}_{i-1}\right\} \tag{3}$$

where, $\{\Delta u'^{\,t'+\Delta t'}_{\,i-1}\}$ is incremental change to the vector solution, expressed in terms of temperature and displacements in thermo-mechanical problems, $[K'^{\,t'+\Delta t'}_{\,i-1}]$ is tangent stiffness matrix also known as the material Jacobian $[J]$, and $\{R^{t'+\Delta t'}_{i-1}\}$ is the residual error vector obtained from the external and internal force vectors. Further, on solving Equation (3) for $\{\Delta u'^{\,t'+\Delta t'}_{\,i-1}\}$, convergence at the next time step, t+Δt is checked and subsequently updated by Equation (4) written as,

$$\left\{u'^{\,t'+\Delta t'}_{\,i}\right\} = \left\{u'^{\,t'+\Delta t'}_{\,i-1}\right\} + \left\{\Delta u'^{\,t'+\Delta t'}_{\,i-1}\right\}. \tag{4}$$

ABAQUS®/Standard solver uses automatic increment by default and the rate of convergence in each time step is monitored to determine the appropriate time and load increments.

3. Material model for thermo-mechanical analysis

It is important to understand the behavior of RC material through the available constitutive models in order to assess the accuracy of responses. Concrete exhibits extremely complex behavior under mechanical loading at elevated temperatures; hence, to obtain accurate computational solution of the RC panels under fire, enhanced and efficiently validated thermal and constitutive material models are required to capture the nonlinearity of the RC materials for advanced computational analysis.

3.1. Thermal properties for thermo-mechanical analysis

The behavior of an RC concrete member exposed to fire is partly dependent on thermal properties of the material. The properties influencing rise in temperature and its distribution in the concrete section are mass loss (ρ), thermal conductivity (λ_c), coefficient of thermal expansion (α_c), and specific heat capacity (c_c). The mass loss, in terms of density and thermal conductivity for normal strength concrete with siliceous aggregates are obtained from the review conducted by Kodur [44]. The mass loss is minimal for concrete with siliceous aggregate up to 1000°C, and this model is in close conjunction with the widely accepted model of the EC2 1-2 [57]. Similarly, upper bound values of the thermal conductivity are adopted for all types of aggregates provided in widely accepted EC2 1-2 [57]. The coefficients of thermal expansion and specific heat capacity at temperature, T (°C) are obtained from the studies conducted by Ruan et al. [45]. More importantly, this model overcomes the abrupt increase in the specific heat at temperature between 700°C and 800°C provided in the EC2 1-2 [57]. Fig. 3 shows the thermal and mechanical properties of reinforced concrete at elevated temperatures. The thermal conductivity, coefficient of thermal expansion, and specific heat capacity of the steel reinforcing bars at elevated temperature are obtained from the comparative studies reported by Gardner et al. [46]; as shown in Fig. 4, which are also based on the widely accepted models provided in the EC2 1-2 [57]. The deterministic thermal properties, discussed herein, serve a basis for the subsequent probabilistic studies required further for developing the fragility curves of the RC panels.

3.2. Mechanical properties for thermo-mechanical analysis

The mechanical properties influencing the behavior of a typical RC structure subjected to fire are compressive and tensile strength, modulus of elasticity, and constitutive law of the ingradient materials [44]. The constitutive relationships of concrete (under compression and tension) exposed to fire are obtained from the study conducted by Aslani and

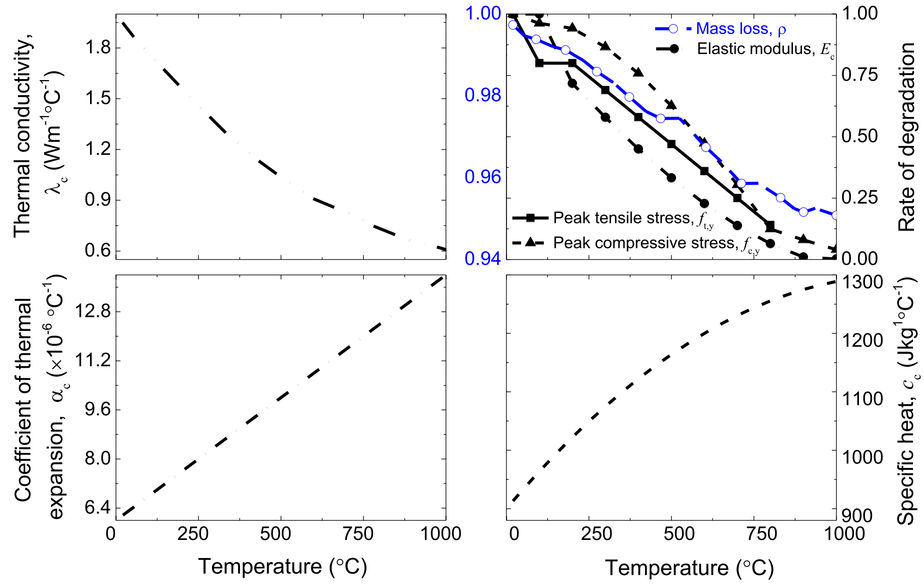

Fig. 3. Thermal and mechanical properties of concrete at elevated temperature (EC2, 2004 [56]; [45]).

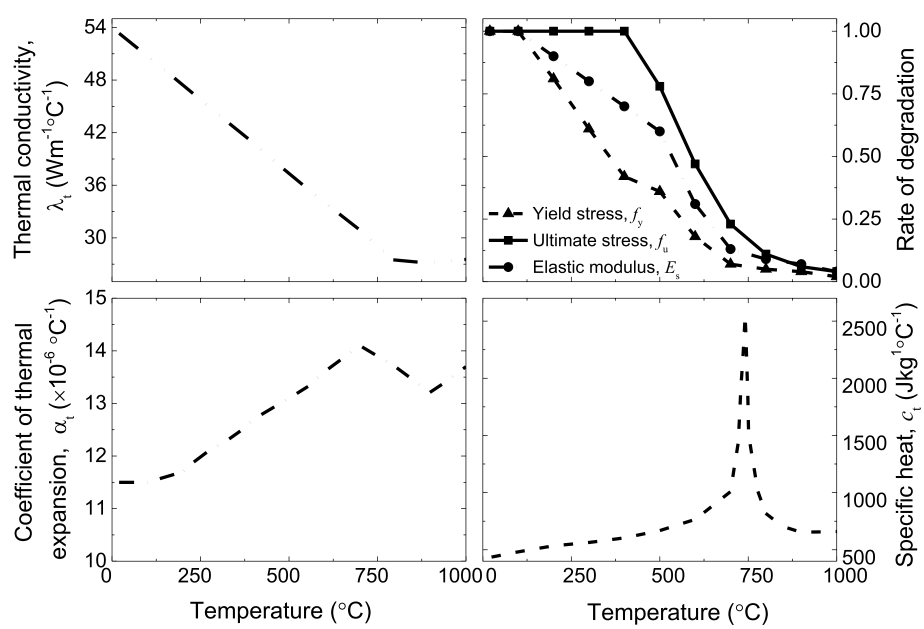

Fig. 4. Thermal and mechanical properties of steel rebar at elevated temperature (EC2, 2004 [56]; [45]).

Bastami [47]; as most of the available constitutive relationships of concrete at elevated temperature are not reliable for determining the stress-strain relationships effectively. Moreover, the developed models are able to predict accurate results, which are in good agreement with the experimental test results. These relationships effectively provide the required database to predict the fire-performance of the RC structures exposed to fire.

The strength of concrete in tension is much lower than its compressive strength; hence, the tensile strength calculations are usually neglected at ambient and elevated temperatures. However, from the point of view of fire resistance, the tensile strength is important due to the development tensile stresses, progression of micro-cracking, further leading to spalling [44]. The constitutive relation of concrete under tension at elevated temperature is similarly obtained from the study conducted by Aslani and Bastami [47].

The degradation in the mechanical properties of the reinforcing steel, such as, yield strength and modulus of elasticity is considered to be vital, considerably affecting the thermal performance of the structural RC panels. In the present scenario, the coefficients of degradation for elastic modulus and yield strength are determined from the EC2 1-2 [57]. The stress-strain relation is also obtained from the EC2 1-2 [57] and subsequently, the values of stresses and strains are determined at elevated temperature. The mechanical and thermal properties of concrete and steel at ambient temperature are shown in Tables 4 and 5. Moreover, the variability in the mechanical properties is considered based on the deterministic thermal and mechanical properties to study the effect of uncertainties in the properties affecting the vulnerability of the RC panels exposed to fire.

4. Fire load scenario

Fire resistance rating of RC structures is based on standard fire curves prescribed by the ISO 834, which uses idealized time-temperature curves. In this regard, the standard fire curves are relatively simpler,

Table 4
Mechanical and thermal properties of concrete at ambient temperature.

	Parameters	Value
Mechanical properties	Compressive strength of concrete (f_c)	30 MPa
	Elastic modulus of concrete (E_c)	27386.12 MPa
Thermal properties	Density (ρ)	2400 kg/m^3
	Conductivity (λ)	1.95 W/(m·K)
	Specific heat (c)	913.22 J/(kg·K)
	Coefficient of expansion (α)	6.16 × 10^{-6}/K

Table 5
Mechanical and thermal properties of reinforcing steel at ambient temperature.

	Parameters	Value
Mechanical properties	Yield strength of steel (f_y)	415 MPa
	Ultimate strength of steel (f_u)	621 MPa
Thermal properties	Elastic modulus of steel (E_y)	2 × 10^5 MPa
	Density (ρ)	7850 kg/m^3
	Conductivity (λ)	53.34 W/(m·K)
	Specific heat (c)	436.09 J/(kg·K)
	Coefficient of expansion (α)	1.15 × 10^{-5}/K

although the accuracy involved in the fire severity has drawn much criticism. On the other hand, natural fire curves recommended in the EC1 [55] are advanced and realistic; however, no standard guidelines have been set to use these advanced time-temperature curves for determining the fire resistance of structure. Nevertheless, researchers recently have been using the EC1 parametric fire curves to determine the performance and resistance of structural members [36]. This is based on existence of uniform temperature in small to medium compartments, unlike in large compartments. The same assumption is difficult to incorporate in large compartments due to existence of considerable differences between the uniform and non-uniform fires [48]. Hence, in the present study, parametric fire curves prescribed in the EC1 [55] is used to determine the fire resistance of the RC panels under fire.

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 panels. For the current study, a realistic natural fire curve is used from the recommendations prescribed in the EC1 [55] to predict the thermo-mechanical behavior of the RC panels. The natural fire curve comprises of two parts: heating phase and cooling phase. The time-temperature curve for the heating phase is given as,

$$T = 20 + 1325\left(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}\right) \tag{5}$$

where, T is temperature obtained during the heating phase in degree Celsius (°C) and $t^* = t\Gamma$ is determined in seconds. The parameter, $\Gamma = (O/b)^2/(0.04/1160)^2$ depends on opening factor, O and thermal inertia, b-factor given by $b = \sqrt{(\rho c \lambda)}$. Here, ρ is density of concrete, c is specific heat capacity, and λ is thermal conductivity mainly used to characterize the thermal properties of concrete material. Here, the thermal inertia, b-factor is assumed for only concrete because the RC panels are not designed for any fire protection systems, nor the panels are provided with any fire resisting coatings. The duration of the heating phase is given by, $t_{max} = \max\{(0.2 \times 10^{-3} \times q_{t,d}/O), t_{lim}\}$; where, $q_{t,d}$ is the fire load density (fire load per total area of the compartment) considered in the design fire curve depending on the occupancy of the building obtained from the characteristic fire load density, $q_{f,d}$ (fire load per floor area of the compartment). The EC1 [55] provides 80% fractile values of the characteristic fire load density, $q_{f,k}$ for different occupancies; however, no proper guideline is available for assumption of the design fire load density. There is also an implication about the fact that the design value should be equal to or greater than the given 80% fractile [49]. Hence, the design fire load density, $q_{f,d}$ is chosen as 80% fractile value of the characteristic fire load density, $q_{f,k}$. Subsequently, the design fire load density, $q_{t,d} = q_{f,d} \times A_f/A_t$ is calculated based on the ratio of the surface area of the floor, A_f to the total surface area of the enclosure (floor), A_t. Moreover, according to the code recommendations, t_{lim} for slow, medium, and fast fire growth rate are 25 min, 20 min, and 15 min, respectively. Likewise, the temperature for cooling phase in degree Celsius (°C) is given as,

$$\begin{aligned} T &= T_{max} - 625\left(t^* - t^*_{max}x\right) \quad \text{for } t^*_{max} \le 0.5 \\ T &= T_{max} - 250\left(3 - t^*_{max}\right)\left(t^* - t^*_{max}x\right) \quad \text{for } 0.5 \le t^*_{max} \le 2 \end{aligned} \tag{6}$$

where, $t^*_{max} = t_{max}\Gamma$, $x = 1$ for $t_{max} > t_{lim}$ and $x = t_{lim}\Gamma/t^*_{max}$.

As the compartment size is < 500 m^2 of the floor area and a maximum compartment height is 5 m, assuming that openings are absent in the roof, the natural fire curve is used for evaluating fire resistance of the RC panels. The required parameters for the heat transfer analysis are provided in Table 6. Finally, considering the variabilities in thermal inertia and fire load density, a set of natural fire curves is obtained, which is plotted in Fig. 5.

5. Fragility function

Fragility estimation has been a common practice to investigate the

Table 6
Deterministic and stochastic parameters used for the thermo-mechanical analysis.

	Parameters		Unit	Distribution	Mean Values	COV
Mechanical properties (concrete and steel)	Strength of concrete/steel (f_c/f_y)		MPa	Lognormal [40]	Table 1	0.1
	Elastic modulus of concrete/steel (E_c/E_y)		MPa			
Geometric properties	Size of panels		m × m × m	Deterministic	Fig. 1, and Fig. 2	–
	Reinforcement details		mm^2			
Mechanical loading	Slab	Dead load	kN/m^2	Deterministic	Table 1	–
		Live load		Extreme Type - I [50]		0.25
	Wall	Dead load	kN/m^2	Deterministic		–
		Live load		Extreme Type - I [50]		0.25
Thermal properties (concrete and steel)	Density (ρ)		kg/m^3	Normal	Table 3, and Table 4	0.1
	Conductivity (λ)		W/(m·K)	Normal		
	Specific heat (c)		J/(kg·K)	Normal		
	Coefficient of expansion (α)		/K	Normal		
Fire loading	Thermal inertia (b)		J/(m^{2}s$^{1/2}$K)	Normal	1800	0.1
	Characteristic fire load density ($q_{f,k}$)		MJ/m^2	Extreme Type - I ([55])	780	0.2
	Duration of fire loading (d)		hour	–	0.1–3	
	Opening factor (O)		m$^{1/2}$	Deterministic	0.075	–
	Fire growth (t_{lim})		min		Slow (25)	–
Heat transfer	Stefan-Boltzmann Constant		W/(m^2·K^4)	Deterministic	5.67 × 10^{-8}	–
	Convection (exposed)		W/(m^2·K)		35	–
	Convection (unexposed)				4	–
	Radiation/emissivity (exposed)		–		0.8	–

performance and determine probability of failure (vulnerability) of structure against imposed forces. The concept of fragility function is defined here as a probabilistic relationship between frequency of failure of a structural member or system (here, RC panels) as a function of some measure of scenario-based extreme loading condition (here, fire loading). More precisely, the fragility function is a mathematical function that describes the probability when the undesirable fire event triggers the response of the structural component (demand) to reach or exceed a threshold limit state (capacity) observed as a function of some intensity measure. In a mathematical form, the fire fragility function can be written as,

$$p_f = P(D \geq C|IM) \tag{7}$$

where, p_f is the probability that the response, i.e., the demand D exceeds the limiting capacity C subjected to fire loading scenario with intensity measure, IM. For the present study, the probability of failure can be expressed as,

$$\begin{aligned} p_f &= P\left(\delta_{top,dem} \geq \delta_{top,cap} \middle| q, d\right) \\ p_f &= P\left(\delta_{cip,dem} \geq \delta_{cip,cap} \middle| q, d\right) \end{aligned} \tag{8}$$

where, δ_{top} indicates the out-of-plane displacement of the RC slab panel under the transverse out-of-plane loading, δ_{cip} represents the out-of-plane displacement of the RC wall panel under the compressive in-plane loading, and q and d represent the IM parameters adopted here, which are fire load density (q) and duration of fire loading (d), respectively. Subsequently, a traditional two-parameter lognormal distribution function is assumed to construct the fragility curves, which is given as,

$$p_f = \Phi\left[\frac{\ln(x/\mu)}{\sigma}\right] \tag{9}$$

where, x represents the fire load density or duration of the fire loading. The two parameters μ and σ are obtained by maximizing the likelihood function, given by Shinozuka et al. [51].

In a typical fire scenario, the structural response, otherwise known as engineering demand parameter (EDP), for the RC panels under the transverse out-of-plane and compressive in-plane loads is generally chosen as deflection of the member, maximum temperature attained in the member, ratio of axial load to critical load, plastic moment capacity, etc. On the other hand, examples of potential IMs include fire load density, fire duration, peak fire temperature, fuel load, cumulative radiant heat, heat release rate, normalized heat release rate, etc. [52]. In this context, one of the best-chosen IM parameters to investigate the structural behavior is fire load density in a compartment, because the fire load density has substantial influence on the response in the adopted numerical processes [53]. Moreover, the evolution of temperature for the parametric fire curve also depends on the assumed fire load density. On the other hand, the duration of fire loading significantly helps in determining the fire resistance of the structure [42]. The structural failure is well-understood when the demand reaches or exceeds the threshold limit state capacity, which is provided by some empirical or semi-empirical relations. In this context, the next sub-section explains the procedure to quantify the threshold limit state of EDP for the RC panels under compressive in-plane and transverse out-of-plane mechanical loads exposed to fire.

5.1. Limit states of failure

As discussed earlier, the fire resistance is evaluated when the RC panel reaches insulation, integrity, or stability related limit states of failure. Considering the limit states, current standards fail to address such definitive criteria to consider deflections when the failure is being impending. The reason being, understandably large number of experiments is required to be performed to produce statistically correct data set; however, this procedure involves significant efforts. For example, threshold limit state of the EDP, defined in terms of limiting horizontal out-of-plane deflection in the RC panel, under compressive in-plane load is $h/100$ [2]; where, h represents the height of the member. The disadvantage in stating the limit state is that, no effect of thickness/size of the element is considered, which may be a governing factor determining the response of the structure. Since, the data is limited in this context, the threshold limit state for the RC panel under compressive in-plane load is

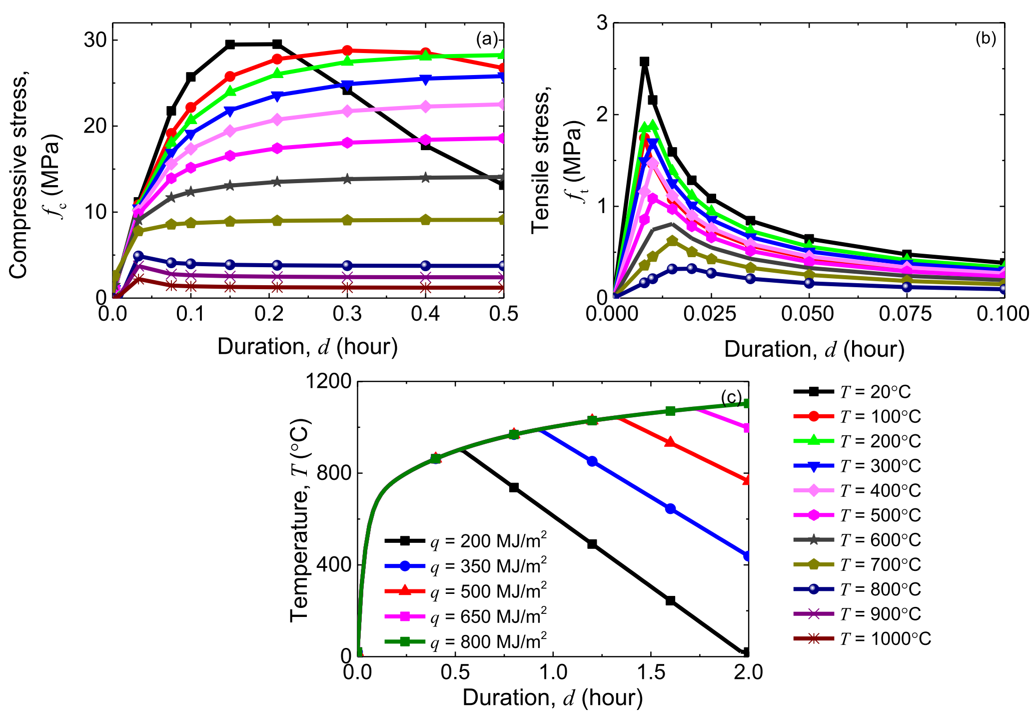

Fig. 5. Stress-strain curves of concrete under (a) compression and (b) tension at different temperatures and (c) time-temperature curves of parametric fire curve for different values of fire load density.

taken as $h/100$. Moreover, for flexural elements, the threshold limit is $L^2/400d$, with a condition that $d \geq L/30$; where, d and L denote the depth and clear span of the panels, respectively. This limit is more specific for flexural element such as RC beam, and considering practical d/L ratios of the RC slabs, the abovementioned threshold limit cannot be used. Hence, considering the limitations in existing limits, a threshold limit state, $L/20$ is chosen based on the span of the RC slab panel. Finally, an algorithm is presented here to evaluate the fragility of the RC panels considering the threshold limit states of failure.

5.2. Procedure followed in fragility estimation

To conduct a number of simulations for computing the failure probability of the RC panels, ABAQUS®-Python scripting interface is used in which the scripts are written in Python programming language. The advantage of using the scripting interface is that, the components of the modeled RC panels can be modified for multiple simulations without the interference of the user to finally extract the output through a user-defined script. After the responses are obtained under the random natural fire curves, probabilities of failure under different IMs are computed by using the formulations mentioned above. In this regard, a nine-step Monte Carlo (MC) algorithm is outlined for construction of fragility curves based on the above-mentioned traditional formulation. Following this algorithm, a flowchart is shown in Fig. 6 to delineate the proposed probabilistic framework for obtaining probability of failure for the RC panels under fire. The steps involved in generation of the fragility curves are discussed hereunder.

Step 1: Define the structural capacity, i.e., deflection capacity (δ_{cap}), of the RC slab panel under transverse out-of-plane load corresponding to the threshold limit state.
Step 2: Generate n random independent and identically distributed (iid) samples for each uncertain parameter (here, 13), ξ_i, $i = 1, 2, 3, \ldots, 13$. The random samples are generated based on a certain distribution considering a fixed mean and standard deviation as the stochastic parameters.
Step 3: Compute the time-temperature curves of the natural fire loading corresponding to the random samples generated for the uncertain IM (q and d).
Step 4: Analyze the RC slab panel under transverse out-of-plane load for each simulated time-temperature curves and subsequently obtain the structural response in terms of vertical out-of-plane deflection of the structural panel (δ_{dem}).
Step 5: Compare the vertical out-of-plane deflection (δ_{dem}) with the corresponding deflection capacity (δ_{cap}) for the single set of IM parameter. The RC slab panel under the transverse out-of-plane load is considered to fail when the vertical deflection exceeds the threshold limit state ($L/20$).

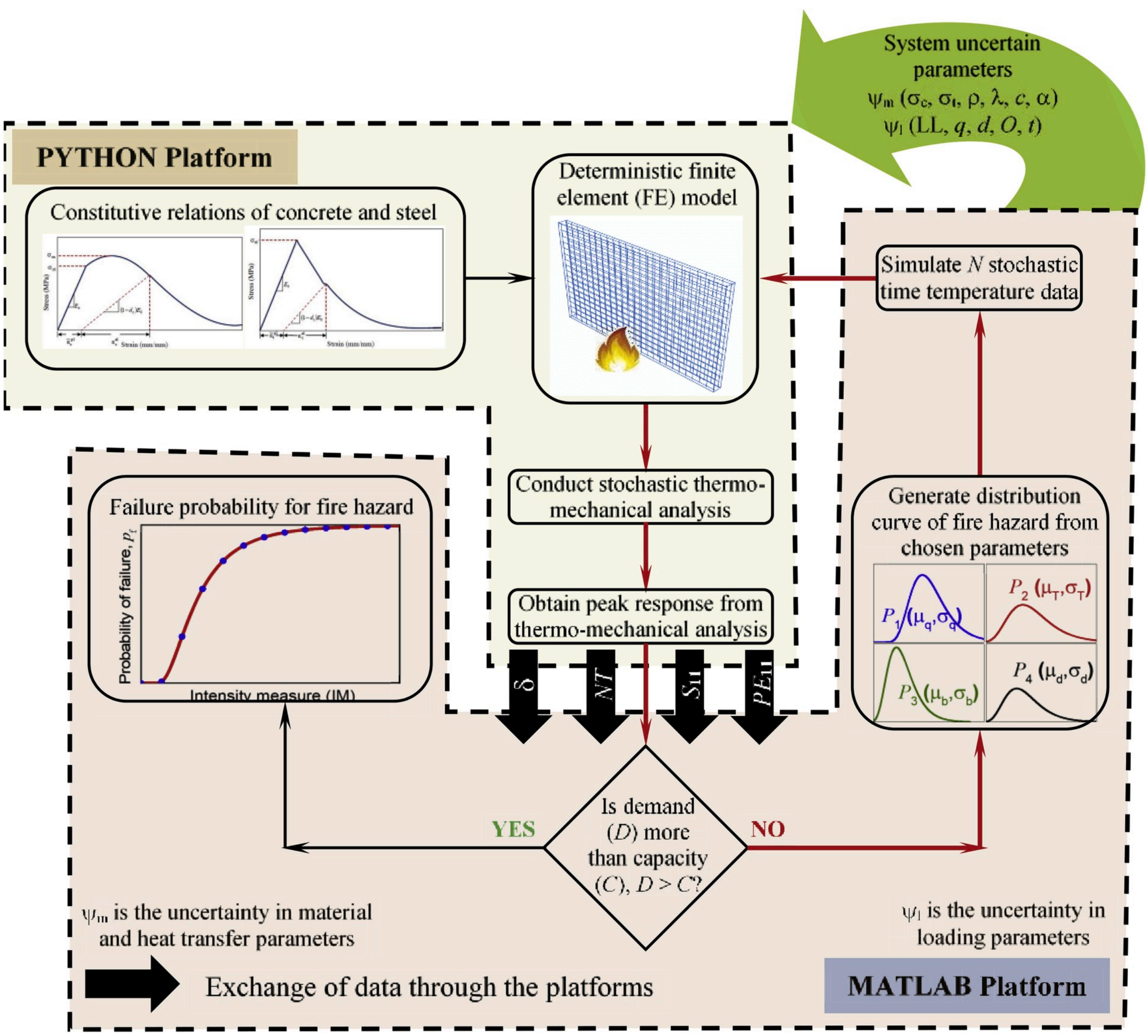

Fig. 6. Framework for fragility estimation of the RC panels under fire.

Step 6: Compute the probability of failure (p_f) using Equation Eq 9 with a condition that the demand (δ_{dem}) reaches or exceeds the capacity (δ_{cap}) for a particular set of IM.
Step 7: Repeat Steps 2 to 6 if the number of simulations in which the demand does not exceed the capacity is N_f; where, N_f is assumed to be 20 for less-skewed fragility curve.
Step 8: Plot fragility curves with IMs (q and d) on the abscissa x-axis and probability of failure on the ordinate y-axis.
Step 9: Repeat Steps 2 to 8 for computing the failure probability of the RC wall panel under compressive in-plane load independently.

6. Numerical study

Herein, the three-dimensional (3-D) RC panels are studied under the effect of thermo-mechanical loading considering nonlinearity in the material and geometric properties. The RC panels used for the study have planar dimension of 6.1 m × 4.3 m with varying thicknesses of 75 mm, 100 mm, and 125 mm. The RC panels are not designed to resist any accidental fire loads, nor any fire resisting material is applied on the panels; only the code-prescribed concrete cover is provided based on site exposure category. The 3-D RC panels are modeled by considering material and geometric nonlinearity and the subsequent transient thermo-mechanical analysis is conducted for single-side exposure of fire loading. The analysis is conducted by incorporating the temperature variation in thermal and mechanical properties of concrete and steel rebar to obtain realistic thermo-mechanical behavior of the panels. The RC panels are subjected to scenario-based fire loadings exposed to single-side, i.e., the fire load is assumed to act at the inner side of the wall and towards the compartment. Similarly, the fire load is assumed to act on the tension side of the slab. The mesh of concrete geometry generated for the RC slab and wall panels with different thicknesses is 60 mm × 60 mm × 60 mm. The optimum mesh is derived after performing a series of trials of mesh convergence tests, which indicates no influence in the obtained numerical results on further decreasing the element size. The thermo-mechanical responses of the RC panels are studied in terms of deflection, distribution of temperature, stress, and plastic strain at the center of the RC panels. Here, $\delta_{top,s}$ denotes transverse out-of-plane deflection, $\delta_{cip,w}$ denotes compressive in-plane deflection, NT denotes nodal temperature, S_{11} denotes normal stress, and PE_{11} denotes plastic strain. The uncertainties in responses depend on the development of fire in different phases, which include the amount of fire load, maximum temperature in fire, duration, etc., heat transfer process, thermal and mechanical properties of the materials, etc., which are considered in the present study. Monte Carlo (MC) simulation is used to generate 1000 random samples for the stochastic analysis to obtain statistically significant output. The probabilistic study is conducted assuming proper distributions based on the deterministic parameters, which are provided in Table 6. Moreover, probability density function (PDF) curves are obtained to observe the nature of the responses obtained from the thermo-mechanical analysis. Finally, fragility curves in terms of cumulative distribution function (CDF) plots are constructed to investigate the vulnerability of the RC panels under the fire loading. Furthermore, the proposed modeling strategy needs to be accurately validated; hence, the FE modeling strategy is justified by duly validating with experimental results available in the literature. The subsequent section provides the validation study to justify the strategy adopted for modeling to further investigate the effect of uncertainty under the fire loading.

6.1. Validation of the FE model

The FE modeling strategy used in the present study is validated by comparing the numerical results of a 3-D RC wall with the experimental results from full-scale fire tests, as reported by Ngo et al. [30].

A nonlinear 3-D RC load-bearing wall is modeled by developing a code in ABAQUS®-Python environment having dimensions 1000 mm × 150 mm × 2400 mm. In the wall RC panel, 8 vertical bars are provided as longitudinal reinforcement of 16 mm ϕ; whereas, horizontal bars of 14 mm ϕ are provided as lateral reinforcement with center-to-center (c/c) spacing of 300 mm with concrete clear cover of 25 mm. The characteristic compressive strength of the concrete used in the model is 31.8 MPa. The load-bearing wall is subjected to an in-plane (axial) load of 485 kN at an eccentricity of 10 mm towards the fire loading. The eccentricity represents typical accidental eccentricity caused due to construction imperfections, geometry, etc. The load-bearing wall is analyzed for simply-supported conditions at the top and bottom surface. A 2-h duration standard fire curve as per ISO 834 is used to simulate the behavior of fire in the wall with single-side exposure. The thermo-mechanical analysis is carried out using C3D8RT element for concrete and T3D2T for rebar. The deflection response at the center as well as different interfaces of the wall is compared as shown in Fig. 7, which shows an accuracy of 97% in the obtained result. The obtained FE results are in good agreement with the experimental study conducted by Ngo et al. [30]; which demonstrates the accuracy of the modeling technique adopted.

7. Results and discussions

7.1. Deterministic study of the thermo-mechanical response

Fig. 8 shows displacement and temperature responses at the center of the RC panels under transverse out-of-plane and compressive in-plane loadings exposed to fire. Here, vertical ($\delta_{top,s}$) and horizontal ($\delta_{cip,w}$) out-of-plane deflections, as well as temperature history for the exposed and unexposed face of the RC panels are plotted to study the behavior under different loading conditions exposed to fire. The deflection profile for the RC slab panel under transverse out-of-plane ($\delta_{top,s}$) is different in comparison to the RC wall panel with compressive in-plane loading ($\delta_{cip,w}$) due to the application of the mechanical load, effect of boundary conditions, and thermal gradient generated due to the temperature difference. The deflected shape of the RC slab panel under transverse loading is governed by the distribution of temperature in the slab and the subsequent thermal strains developed. On the other hand, high thermal inertia of concrete induces high thermal gradients resulting in relatively higher expansion near the exposed face of the RC wall panel, which causes a displacement of the wall away from exposed face. This effect, known as thermal bowing, causes an extensive horizontal out-of-plane displacement at the unexposed side. The maximum vertical out-of-plane deflections ($\delta_{top,s}$) observed for the RC panel under the transverse mechanical loading are 325.19 mm, 268.38 mm, and 179.82 mm, respectively for 75 mm, 100 mm, and 125 mm thick panels. The maximum horizontal out-of-plane deflections ($\delta_{cip,w}$) observed for the RC wall panel under compressive in-plane loading are 127.22 mm, 71.18 mm, and 37.52 mm, respectively for the three thicknesses. However, post-peak deflection is not observed in this case because the fire curve considered for the deterministic study have negligible cooling phase as observed in Fig. 5(c). Moreover, there is a significant difference in nodal temperature (NT) observed at the exposed and unexposed faces of the RC panels, which is also due to temperature gradient and high thermal inertia of concrete.

The fire rating is also obtained from the behavior of the RC panels using the failure limit criteria set for obtaining the vulnerability of the structural panels. In this case, the RC slabs with 75 mm and 100 mm thicknesses have fire ratings of 0.438-h (~26 min) and 0.858-h (~51 min), respectively. Hence, for increase in thickness by 25 mm, the fire resistance has increased by almost 100%, and with further increase in thickness of the panel by 25 mm, the fire resistance time crosses 2-h indicating how the thickness of the RC panel significantly influences its behavior when exposed to fire. On the other hand, the threshold limiting value for failure of the RC wall panel under compressive in-plane loading is based on the deflection and the observed rate of change of deflection. The fire rating of the RC wall with 75 mm thickness is observed to be 1.221-h (~73 min). The fire ratings for the walls with

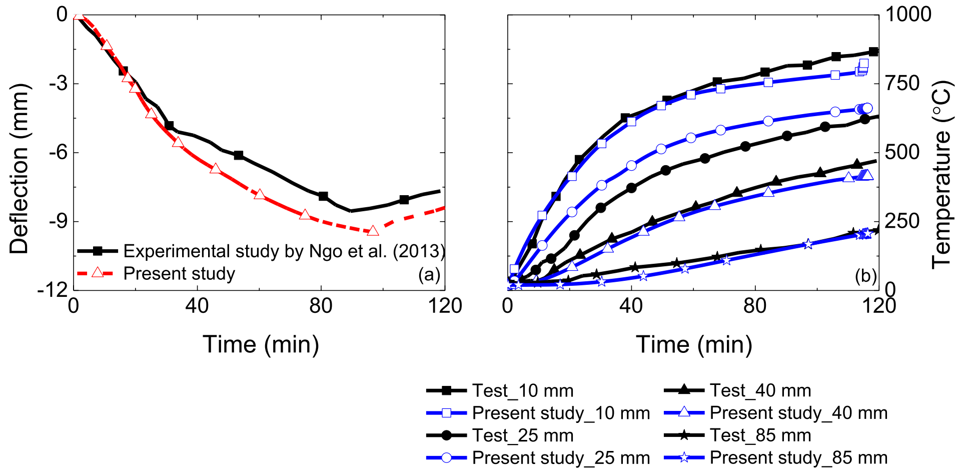

Fig. 7. Validation of FE model with that of the experimental results.

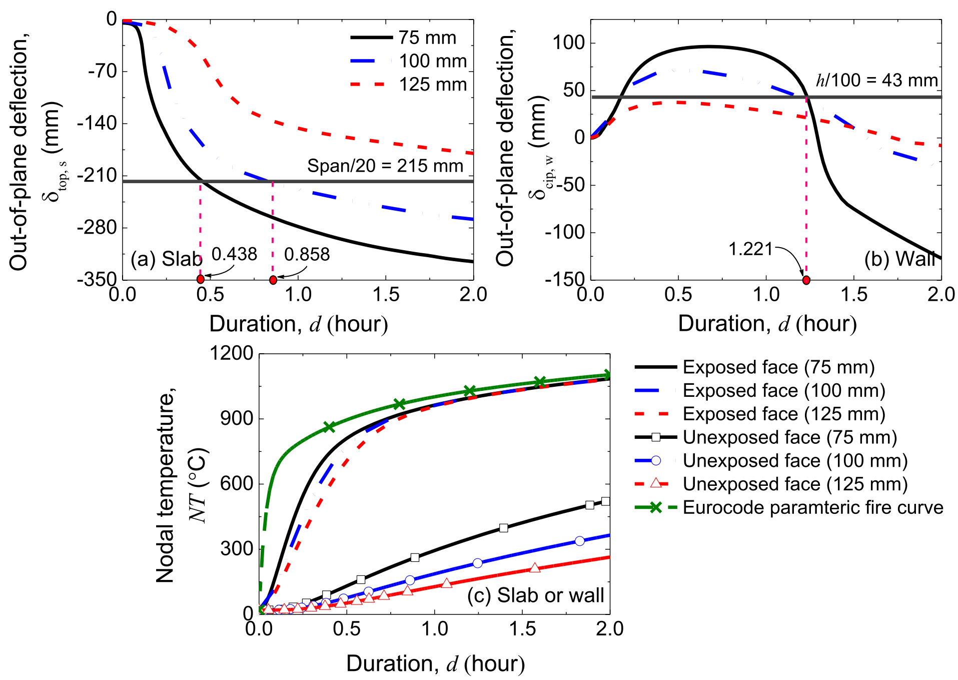

Fig. 8. Out-of-plane deflection of the RC panels under (a) transverse out-of-plane loading, (b) compressive in-plane loading, and (c) temperature responses at the exposed and unexposed sides of the RC panels under transverse out-of-plane loading.

100 mm and 125 mm are more than 2-h, primarily based on the rate of change of deflection, although the limiting value of displacement is observed to be satisfying and is more close to the fire rating for the 75 mm wall. Therefore, the thickness of the RC panels has significant influence in determining the fire resistance duration. Consequently, it can be concluded that the fire resistance duration of the RC panels should be in accordance with the thickness of the panel, and not merely based on the span or height of the panel.

Fig. 9 illustrates the induced normal stress history (S_{11}) in concrete for different RC panels with varying thicknesses. Here, both compressive and tensile stresses are developed in the RC panels depending upon the behavior of the structure under the thermo-mechanical loading. The exposed side of the RC slab under transverse loading is observed to have tensile stresses, while compressive stresses are developed at the exposed face of the RC wall under compressive loading. Again, high thermal inertia of concrete induces high thermal gradients and differential thermal expansion, which results in higher expansion near the exposed face, causing a displacement of the RC slab in direction opposite to the mechanical loading. However, due to relatively significant mechanical loading, the thermal bowing is overcome causing out-of-plane vertical deflection of the RC slab. Therefore, tensile stresses are generated in the exposed face of slab as this face undergoes tension due to the effect of mechanical loading as well as due to insignificant thermal bowing.

On the other hand, relatively more compressive stresses are developed on the exposed faces of the RC wall under compression as an effect caused due to simultaneous mechanical and thermal load application. This causes the thermal bowing to be in the same direction, which allows the exposed face to be in compression zone. In this case, maximum compressive stresses generated due to the thermo-mechanical loading are 15.93 MPa, 17.83 MPa, and 18.39 MPa, respecively for 75 mm, 100 mm, and 125 mm thick RC walls. Further, at the exposed face of the wall, there is a significant increase in the rate of increase in stress, which is mainly due to the development of thermal gradients. The changes in the stress level within the wall section cause a shift in the neutral axis, which is perhaps towards the unexposed face. The stresses soon start to reverse as the temperature increases within the RC wall and at the unexposed

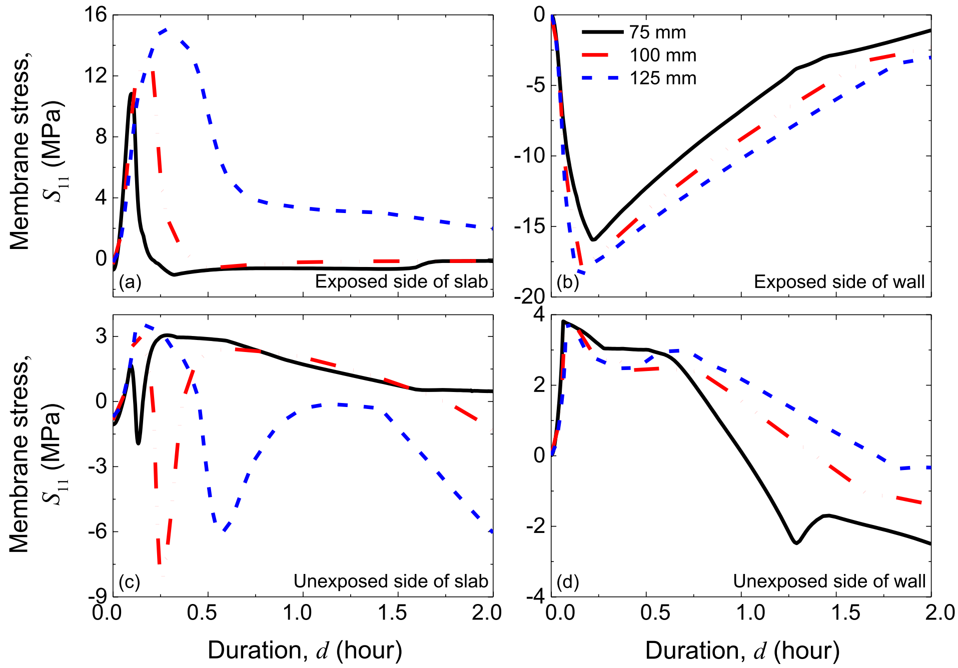

Fig. 9. Stress (S_{11}) of the RC panels under different mechanical loadings exposed to fire.

faces, the RC wall panel undergo compressive and tensile stresses, respectively. Overall, the peak stresses do not vary significantly; however, the nature of stresses changes due to the effect of mechanical loading.

7.2. Probabilistic study on the thermo-mechanical response

Herein, the influence of uncertainties is studied to investigate the vulnerability of the RC panels under coupled mechanical (transverse out-of-plane and compressive in-plane) and fire loadings. The effect of uncertainty in the obtained responses ($\delta_{top,s}$, $\delta_{cip,w}$, NT, S_{11}, and PE_{11}) for the RC panels is investigated from the box plots, probability density function (PDF) plots, and fragility curves. Fig. 10 shows the statistics of 1000 responses ($\delta_{top,s}$ and $\delta_{cip,w}$) for the RC slab and wall panels of 75 mm thickness. The figures also indicate the mean response (50th percentile) along with 84th and 95th percentile responses to show the significance of the data set. It is observed that considering a practical range of uncertainty, the fire resistance of the slab panel with the transverse out-of-plane loading lies from 0.254-h (~15 min) to 0.885-h (~53 min) with the mean resistance being 0.449-h (~27 min). The deterministic fire resistance obtained is 0.438-h (~26 min). Similarly, the fire rating of the RC wall under the compressive in-plane loading indicates that the fire rating lies between 0.836-h (~50 min) to 1.905-h (~114 min) with the mean and deterministic fire resistance being 1.151-h (~69 min) and 1.221-h (~73 min). The differences in the fire resistance durations are marginal in this respect (2.75% and 6.17%); however, the extent of the duration of fire resistance indicates the degree of uncertainty involved in the process. Moreover, the behavior of the RC panels in the cooling phase can also be observed from the post-peak deflections. In the cooling phase, the deflections are observed towards the direction of fire, which is significant. According to the current design guidelines, the fire rating is obtained from a specific thickness, aggregate ratio, and concrete cover, which shows the existing prescriptive methodology is relatively conservative and lesser rational, as it does not include the real-time variability involved in the system. Therefore, it can be concluded that the system uncertainty has a significant influence on the duration of fire resistance of the RC panels under the thermo-mechanical loading.

Fig. 11 shows the box plots constructed to show the degree of uncertainty in the responses ($\delta_{top,s}$, $\delta_{cip,w}$, NT, S_{11}, and PE_{11}) of the RC panels under the thermo-mechanical loadings. The displacement responses ($\delta_{top,s}$) obtained for the RC slab panel under transverse out-of-plane loading show that mean response of the 75 mm thick panel is the largest, whereas the mean response of the 125 mm thick panel is the smallest, which is expected. Moreover, for the 125 mm thick RC panel, the responses from 5% to 95% are marginally skewed on the lower side. In contrast, the displacement response ($\delta_{cip,w}$) for the RC panel under the compressive in-plane loading indicates the same trend as the RC panel

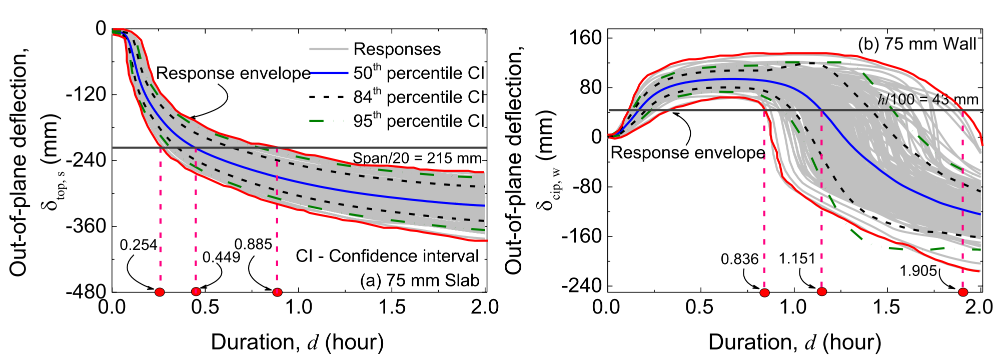

Fig. 10. (a) Vertical and (b) horizontal out-of-plane deflection for the 75 mm size RC panels under transverse out-of-plane and compressive in-plane loadings.

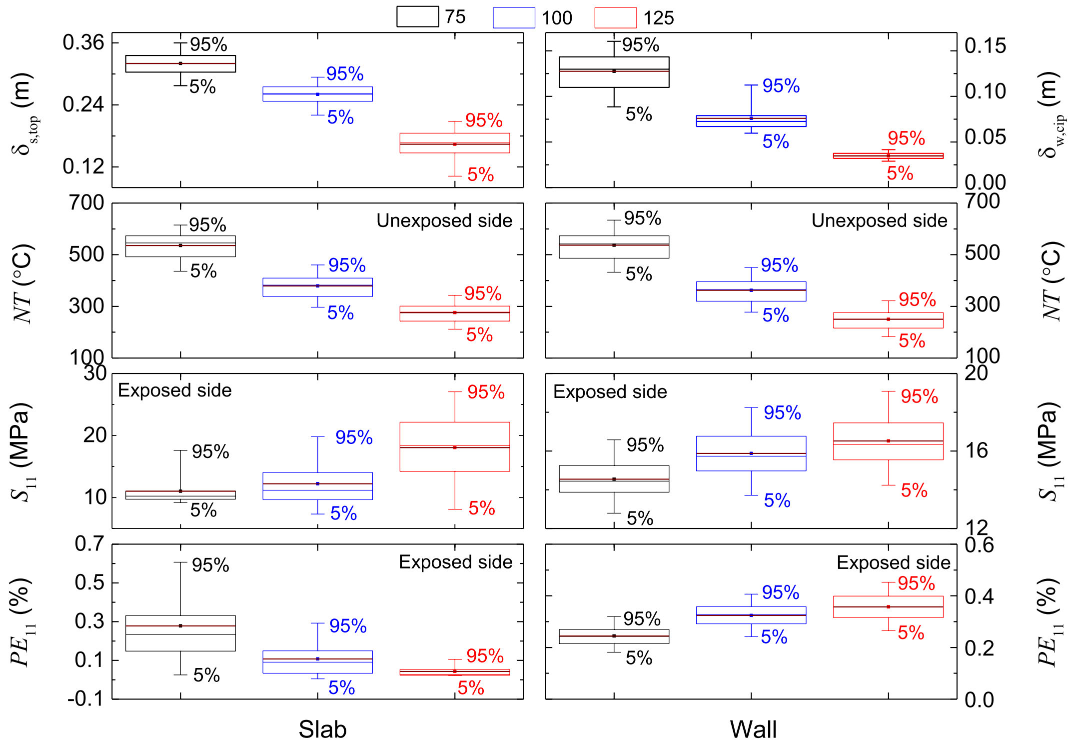

Fig. 11. Box plots representing the degree of uncertainty in the adopted responses of the RC panels.

under the transverse out-of-plane loading with respect to the mean response. However, the response ($\delta_{cip,w}$) obtained for 75 mm and 100 mm thick panels under the compressive in-plane loading are relatively more uncertain with substantial degree of skewness. This is because the mechanical loading has greater influence in determining the deflection behavior of the RC wall panels. The degree of uncertainty observed for the nodal temperature (*NT*) at the center of the RC panels is almost analogous because the temperature response is independent of the type of mechanical loading and is entirely dependent on the heat transfer properties of the materials. Furthermore, the level of uncertainty in the stresses (S_{11}) and plastic strains (PE_{11}) are comparably higher for the RC panels under the transverse out-of-plane loading. Therefore, the structural capacity, heat transfer properties, and fire loading induce significant uncertainties in the response of the RC panels under the thermo-mechanical loading scenarios.

Fig. 12 shows the variation in displacements ($\delta_{top,s}$ and $\delta_{cip,w}$) and

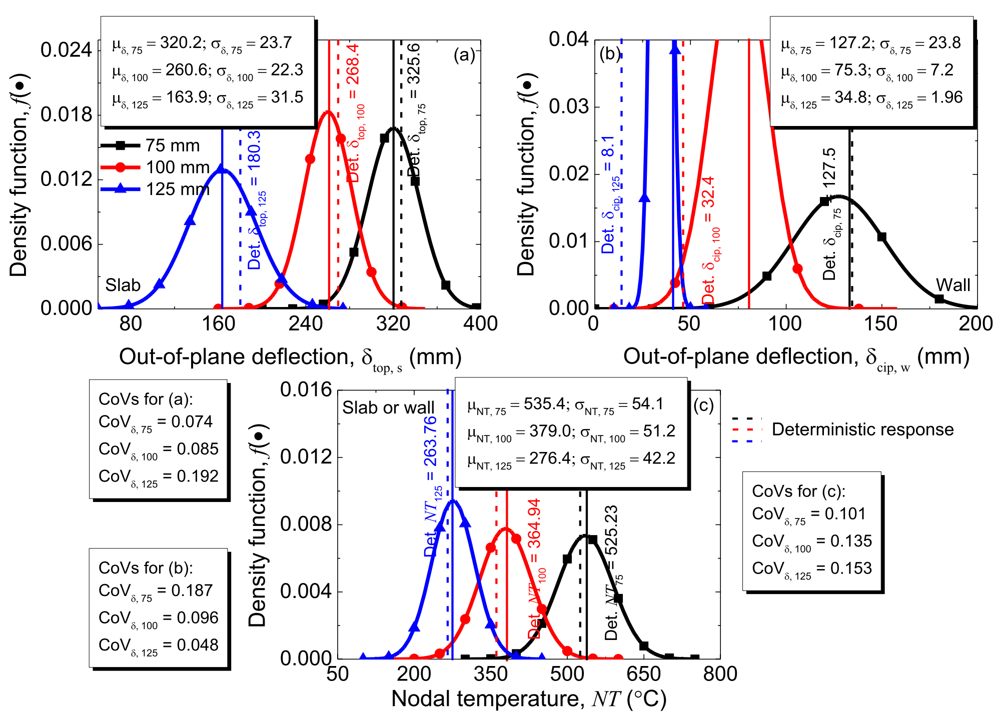

Fig. 12. PDF of the responses for the RC panels under the thermo-mechanical loadings.

nodal temperature (*NT*) of the RC panels for different thicknesses in terms of probability density function (PDF) plots. The PDFs, $f(\bullet)$ of the out-of-plane deflection ($\delta_{top,s}$) due to the transverse out-of-plane loading indicate that the mean values of the responses (μ_*) increase with decrease in the panel thickness. The mean values of the responses (μ_*) are further compared with the peak deterministic responses to assess the effect of uncertainty in the responses. The mean response ($\mu_{\delta,75}$) obtained for the RC panel of thickness 75 mm is 320.20 mm whereas the peak deterministic response is obtained as 325.60 mm. The percentage (%) difference in the responses here is obtained as 1.66%. Similarly, the percentage differences in the responses for the 100 mm and 125 mm thick RC panels are obtained as 2.91% and 9.10%. On the contrary, the differences in the responses ($\delta_{cip,w}$) obtained for the wall panel with compressive in-plane loadings show extreme variation in the mean and deterministic values. The percentage differences in the responses for the 100 mm and 125 mm thick panels are 56.97% and 76.72%, respectively. The mean (μ_*) and deterministic values of the nodal temperature (*NT*) obtained at the center of the panels on the unexposed side indicate that the percentage differences are insignificant, as the temperature distribution is independent of the nature of mechanical loading and is dependent on the heat transfer properties, which are mostly identical. The difference in the responses is attributed mainly to the effect of mechanical loadings, as the variation in the heat transfer properties is the same.

As observed, the effect of mechanical loads is more pronounced with increasing temperatures, causing significant deflection in the RC panels. This shows that the effect of mechanical loadings in combination with the material degradation caused by elevated temperature has significant role in determining the thermo-mechanical response of the RC panels. Furthermore, the coefficient of variation (CoV) is studied to understand the effect of uncertainties on the fire rating of the RC panels. The CoVs obtained for the slab panels under transverse out-of-plane loadings for the 75 mm, 100 mm, and 125 mm thicknesses are 0.074, 0.0855, and 0.1922, respectively. Similarly, the CoVs for the wall panels under compressive in-plane loading show similar trend (Fig. 12). Although, a majority of the input variables have CoV of 0.1 or 0.2, a wide range of the CoVs for the responses clearly indicate significant influence of the uncertain parameters in determining the fire rating of the RC panels.

Figs. 13 and 14 show the probability of failure (p_f) curves for the RC panels under transverse out-of-plane and compressive in-plane loadings considering threshold limit state of failure in terms of fire load density (q) and duration (d) of fire exposure. The fragility curves are constructed according to Equation (9), in which the lognormal parameters are obtained from the optimization technique. Here, μ_{s*} and μ_{w*} respectively represent the mean of lognormally fitted fragility curves for the RC slab

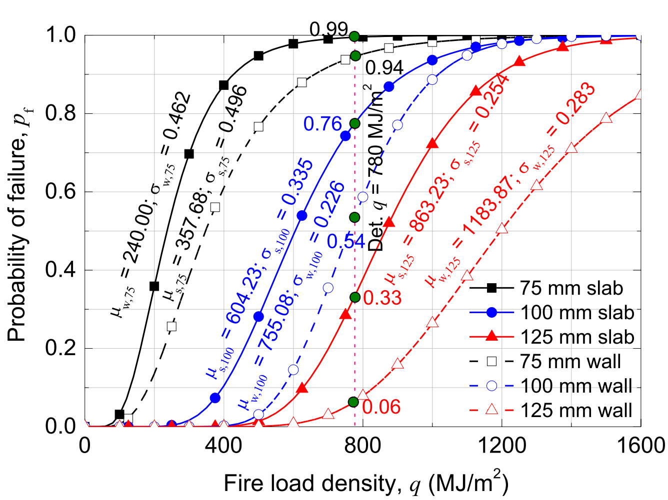

Fig. 13. Fragility curves of the RC panels under the thermo-mechanical loadings considering fuel load as intensity measure parameter (IM).

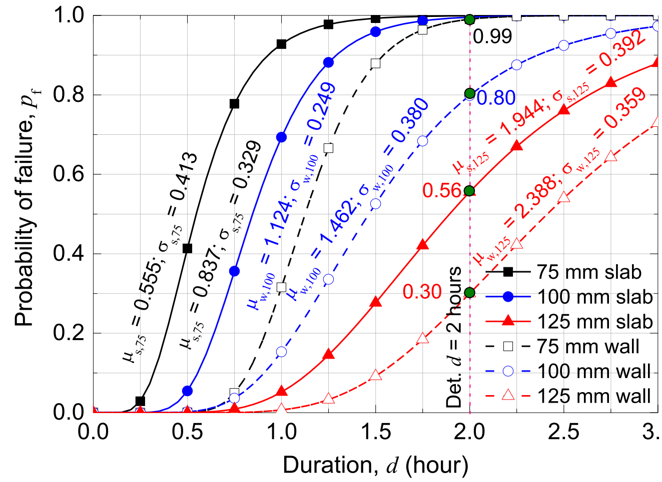

Fig. 14. Fragility curves of the RC panels under the thermo-mechanical loadings considering duration as intensity measure (IM) parameter.

under transverse out-of-plane loading and the RC wall under compressive in-plane loading; whereas, σ_{s*} and σ_{w*} respectively represent the standard deviation of the lognormally fitted fragility curves for the slab and wall panels.

Fig. 13 shows the fire fragility curves for the RC panels of different thicknesses for fire load density. The fragility curves indicate that the vulnerability increases with decrease in thickness of the panels, and the RC panels under transverse out-of-plane loading exposed to fire are relatively more vulnerable as compared to the RC panels under compressive in-plane loading. Therefore, buildings such as, hospitals, offices, schools, etc. ($q \sim$ 200–300 MJ/m^2) constructed with RC panels of 75 mm thickness may undergo significant distress causing failure, as probability of failure under the thermo-mechanical scenario may be higher. Furthermore, the effect of uncertainty in the system is also observed from the range of standard deviation (σ_*) obtained for constructing the fragility curves. The standard deviations (σ_*) of the curves are obtained in the range from 0.226 to 0.496, which indicate the influence of uncertainty governing the structural behavior of the RC panels. Considering the deterministic fire load density obtained for dwelling house systems (q = 780 MJ/m^2), the RC panels of thicknesses 75 mm and 100 mm have considerably higher probability of failure (more than 50%) as compared to the panel with 125 mm thickness. This shows that, to ensure fire protection for the dwelling houses, it is recommended to use the RC panel of 125 mm, irrespective of its functionality.

Although the design of RC building consisting of the panels is carried out using the Indian standard, the philosophy behind this general recommendation is based on the fact that similar thicknesses of concrete cover are used by almost all design codes in different parts of the world for similar type of exposure conditions. Hence, any building not designed for fire resistance will portray similar performance under the exposure of natural fire curve, for which this recommendation might be helpful to achieve the required fire performance. Moreover, the proposed framework also shows that given the realistic parameter inputs for such type of RC structure, the minimum required thickness of the panel should be 125 mm, because there is significant failure for the RC panels with lesser thicknesses. Upon increasing the thickness of the RC panels, failure would be observed on further increase in the fire loading as well as the duration, and for the given present conditions, the failure probability might not be observed even. As for residential buildings, a 2-h rating member is deemed high enough for fire protection and evacuation, the minimum thickness is thus recommended to be 125 mm. Furthermore, according to the code provisions and design guidelines, the determination of fire resistance or fire rating is based on a standard fire curve, which has no direct influence on the fire load density that affects temperature growth. Such practice overestimates the design parameters of the structures required for fire safety, and necessitates reconsideration to achieve safe as well as economic designs. Hence, the importance of considering uncertainty in the fuel load to determine the failure probability and subsequently leading to estimate the fire resistance of the RC panels is deemed important. Such probabilistic assessments will enormously help in incorporating necessary amendments in the development of performance-based design guidelines for structures in fire.

Fig. 14 shows the failure probability (p_f) against duration to obtain the fire resistance of the RC panels. Similar to the fragility curves obtained for the limit state of fire load density, the failure probability curves for duration of fire loading indicate that the vulnerability increases with decrease in thickness of the panels. Moreover, the RC wall panels under the compressive in-plane loading exposed to fire are significantly less vulnerable. It is also observed that the failure of the RC slab with 75 mm thickness under the transverse out-of-plane loading starts at almost 0.25-h (15 min) as compared to almost 0.7-h (42 min) for the 75 mm thick wall panel under compressive in-plane loading. Moreover, the earliest time to start the failure for the panels with 125 mm thicknesses under transverse out-of-plane and compressive in-plane is 0.75-h (45 min) and 1-h (60 min), respectively. Therefore, it can be concluded that the RC wall panels under the compressive in-plane loading are less vulnerable upon exposure to fire. The extent of uncertainty is also observed from the standard deviation (σ_*), which ranges from 0.249 to 0.413, and also observed from the steepness of the curves. Finally, the fire rating is also obtained from the fragility curves, which is determined from 50% probability of failure. The fire resistance for the RC slab panels under the transverse out-of-plane loading is obtained from 0.56-h (~34 min) to 1.95-h (117 min); whereas, for the RC wall panels under the compressive in-plane loading the resistance is obtained from 1.12-h (~67 min) to 2.39-h (143 min). From the deterministic 2-h fire duration, the vulnerability for the panels with 125 mm is relatively lower with the failure probabilities (p_f) obtained as 56% and 30% respectively for the RC panels under the transverse out-of-plane and compressive in-plane loading.

To summarize, the present stochastic framework is effective in quantifying the failure probability (p_f) that accounts for different sources of uncertainty affecting the fire vulnerability of RC panels under different mechanical loadings. The developed fragility curves are beneficial to predict the structural damage to further benefit the building construction 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 structures in fire. Finally, the developed stochastic framework is also suitable to construct analytical fire fragility functions for other different typologies of the RC framed structures.

8. Conclusions

Herein, a probabilistic approach is employed to compute the probability of failure for fire-exposed RC panels under the transverse out-of-plane and compressive in-plane mechanical loadings. The three-dimensional (3-D) RC panels are modeled by considering material and geometric nonlinearity, and the subsequent transient thermo-mechanical analysis is conducted for single-side exposure of fire loading. The uncertainties in the system are considered in material strength i.e., structural capacity, heat transfer model, and parametric fire curve to study the stochastic thermo-mechanical response for the RC panels. Based on this investigation, the major conclusions drawn are as follows:

1. The mechanical loading in combination with the material degradation caused by the elevated temperature has significant influence in determining the thermo-mechanical response of the RC panels under transverse out-of-plane and compressive in-plane loadings as compared to purely thermal loading. The vulnerability increases with decrease in the thickness of the panels and the RC slab panels under the transverse out-of-plane loading exposed to fire are relatively more vulnerable as compared to the RC wall panels under the compressive in-plane loadings.
2. The thicknesses of the RC panels have substantial effect in obtaining the fire rating of the structural panels. Therefore, the fire resistance duration of the RC panels is proposed in accordance to the thicknesses of the panel, and not merely based on the span or height of the panels. The fire resistance for the RC slab panels under the transverse out-of-plane loading is obtained as 0.56-h to 1.95-h; whereas, for the RC wall panels under the compressive in-plane loading, the fire resistance is obtained as 1.12-h to 2.39-h. Moreover, to ensure fire protection for the dwelling houses, it is recommended to use the RC panel of 125 mm thickness, irrespective of its functionality.
3. According to current design guidelines, the fire rating is obtained from a specific thickness, aggregate ratio, and concrete cover, which shows the existing prescriptive methodology is relatively conservative as it does not include the real time variabilities involved in the system. However, the effect of uncertainty in the system is substantial, which is also observed from the range of standard deviation obtained for determining the vulnerability of the RC panels. Therefore, it can be concluded that the system uncertainty has significant influence on the fire resistance duration of the RC panels under the thermo-mechanical loading.
4. According to the code provisions and contemporary design guidelines, the determination of fire resistance or fire rating is based on a standard fire curve, which has no direct influence on the fuel load that affects 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. Hence, the importance of considering uncertainty in the fuel load to determine the failure probability and subsequently leading to estimate the fire resistance of the RC panels is deemed important.

Appendix A. Supplementary data

Supplementary data to this article can be found online at https://doi.org/10.1016/j.firesaf.2020.102976.

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