A probabilistic framework for assessment of reinforced concrete wall panel under cascaded post-blast fire scenario

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ABSTRACT

In a terrorist attack, structural elements such as, reinforced concrete (RC) wall panels have a relatively higher degree of exposure to blast loading which has the potential to trigger devastating fire hazard. As a result, there is significant limitation to the load carrying capacity of the structure, thereby leading to early collapse of structure. Therefore, to address this critical issue, a new probabilistic framework is developed and used to investigate reinforced concrete (RC) wall panels of different thicknesses exposed to cascading post-blast fire (PBF) scenario. The RC wall panels at elevated temperature after a random blast scenario are analyzed by nonlinear finite element (FE) approach considering material and geometric nonlinearities. The developed FE models for the RC wall panels are first exposed to uncertain blast scenario, and subsequently to random fire exposure. Uncertainties are considered in the capacity of structural members in terms of material strengths and heat transfer properties, and demand in terms of mechanical, blast, and fire loading. Response compared are in terms of deflection of the wall panels, variation of stresses at different interfaces, and damage pattern. It is concluded that the response under cascaded hazard involves significant damage to the RC wall panel, wherein the PBF requires ~27%–~35% less time to initiate failure, which is quite substantial considering the fire safety of structure. Further, the system uncertainty has significant influence on the duration of fire resistance of the RC wall panels under the PBF scenario. Finally, irrespective of the damaging concerns, it is recommended to incorporate the effects of the cascading hazard scenario to protect the structures from extreme accidental/mannmade threats.

1. Introduction

Recent terrorist activities in the form of blast and its cascading effects in the form of fire have caused significant catastrophic failure of structures through intricate complex phenomena [16]. Reinforced concrete (RC) structures have shown excellent resistance to such accidental blast or fire exposure as compared to steel structures, and in this regard, the RC material is widely used in design of blast- or fire-resistant structures. Despite noteworthy performance under extreme loading, one of the reasons for excessive global damage of the structures is loss of structural wall panel. The panel members, forming an essential part of the building envelop, are expected to withstand any unnecessary external threats such as, blast loading, and are also expected to restrict the horizontal fire outspread and maintain required stability. The RC wall panels, thus, form an integral part of a building system; therefore, it is deemed important to study their performance under extreme blast and the cascaded fire loads.

Dynamic loadings such as blast and explosion induce dynamic effects in concrete, which in turn alters the cracking pattern in concrete more rapidly as compared to its static state [4]. Thereafter, high temperature attained from the PBF loading induces further micro-structural changes in the already degraded RC structural member [41]. In such critical scenario, transfer of heat is relatively faster to the core of RC member through the induced cracks and spalling, thereby reducing the strength drastically at comparatively lower temperatures. Hence, it becomes essential to investigate the thermo-mechanical behavior of the RC structural member after considering the dynamic high strain-rate effects with the purpose of improving the integrity, stability, and survivability of concrete structures under the cascaded effects of post-blast fire.

Although notable developments have been made in design and
patterns with temperatures rising to 450°C. They concluded that the fire load altered blast load response and crack loadings [5,13,15,24,36,40,52], there has not been much research under the cascading effects of blast and fire. Kakogiannis et al. investigated blast-bearing capacity of RC hollow core slabs under PBF [32]. They concluded that the fire load altered blast load response and crack patterns with temperatures rising to 450 °C. Yi et al. compared the performance of prestressed concrete (PSC) and RC members under the cascading effects of impact and fire loadings [58]. On an interesting note, the RC members had relatively lower energy absorption and impact resistance capacity than the PSC members. Recently, Jin et al. investigated fire resistance of steel fiber RC beams after subjecting to blast-induced low-velocity impact loading [31]. They concluded that on relatively lower impact energy, the beams show good fire resistance despite weakened by pre-impact. Therefore, it can be observed that the research involved until date is limited that accounts for interaction of the hazards and its possible cascading effects. Moreover, no notable research is observed relating to assessment of RC structural wall panels under extreme PBF scenario.

While few significant contributions exist for safety of structures under independent blast and fire scenarios in probabilistic domain [11,12,23,42,53,59], research is lacking in the same to demonstrate the complexities affecting the structural reliability under post-blast fire scenario. This is attributed to the fact that the current design standards, i.e., resistance of structures against either blast or fire consider prescriptive design approaches, also do not necessarily utilize the effect of several critical parameters affecting the dynamic blast, or thermal behavior, or interaction effects of the both [19,47,57]. The conventional deterministic approaches, in such circumstances, have eventually resulted in practically over-conservative and uneconomic structural design. In this context, improvising the current design guidelines against extreme post-blast fire (PBF) scenario should be one of the crucial objectives to minimize the community vulnerability against such cascaded hazard, as the existing standards and guidelines do not consider the integrated approach of multiple hazards for design life of a structure. Such advanced multi-hazard approach would help in upgrading the existing assessment methodology in probabilistic scale to form a benchmark for investigating the survivability of structural systems under multiple hazard scenarios [8,49].

Herein, a probabilistic methodology is proposed to investigate the vulnerability of RC wall panels under post-blast fire (PBF) scenario. The three-dimensional (3-D) RC wall panels are modeled and analyzed in finite element (FE) platform considering material and geometric nonlinearities. Uncertainties are considered in capacity of structural members in terms of mechanical and thermal properties, and demand in terms of mechanical, blast, and fire loading. Response compared are in terms of rotation and deflection of the wall panels, variation of stresses at different interfaces, and damage pattern under both blast and PBF scenario and compared against normal fire (NF) scenario. Therefore, based on the above-mentioned research gaps, the main objectives enlisted are: (i) to study and compare the behavior of the wall panels under NF and PBF scenarios, and compare the additional damaging effects caused by the PBF loading, (ii) to compare the response of the wall panels under the NF and PBF scenarios, and highlight the difference in fire resistance rating caused by the PBF, (iii) to study the probability of failure for the wall panels by constructing fragility curves, and (iv) to study the effect of uncertainty in input parameters influencing the output response parameters.

2. Fragility framework for cascading post-blast fire scenario

In recent times, the concept of performance-based (PB) approach has emerged in blast or fire engineering, which uses probabilistic relationship between frequency of failure of a structural member or system as a function of some measure of scenario-based extreme loading condition (blast/fire) [33,51]. Using this approach, the probability of failure for the independent blast/fire and cascaded PBF is estimated. In this case, the probability of failure ($p_f$) under only blast loading is accordingly expressed as,

$$p_f = \int \int P(F|S = \delta^b) dP(S^b)$$

(1)

where, $S^b = \text{reflected overpressures characterized by charge weight (w) and standoff distance (s)}; P(S^b) = \text{annual probability of occurrence of blast in terms of the reflected overpressures};$ and $P(F|S = \delta^b) = \text{probability of failure (F) conditional on the intensity of blast loadings characterized by reflected overpressures,} S^b$. Here, $F$ represents a failure event, defined as the exceedance of a threshold limit state.

However, the utilization of the PB approach in PBF is not a straightforward procedure due to the challenges involved in defining the elementary components of the framework as well as the understanding the complexities in cascading effects induced by the blast and fire, such as integration of conditional failure probabilities over the distribution of uncertain resistance and loading parameters [2]. Despite the complicated approach, fragility functions are developed based on Bayes’ theorem of probability in which the cascading fire event occurs after blast.

Hence, the probability of failure under the PBF, conditional on the occurrence of blast is given by,

$$p_{bf} = \int \int P(F|S^b = \delta^b, S^f = \delta^f) dP(S^f) dP(S^b)$$

(2)

where, $S^f = \text{fire hazard characterized by fire load density, q}; P(S^f = \delta^f) = \text{conditional probability of occurrence of fire after occurrence of blast with characteristics,} S^b; P(F|S^b = \delta^b, S^f = \delta^f) = \text{probability of failure (F) due to} S^f \text{conditional on} S^b$.

In such scenario, the components of the fragility functions are adjusted based on the degradation of structural parameters, especially the strength and stiffness obtained from the pre-damaged structural member exposed to extreme blast loading. The failure probability curves under fire conditioned on the damage occurred after blast are finally constructed based on two-parameter lognormal distribution, as prescribed by Shinozuka et al. [54].

The PB framework is mathematically characterized by four primary variables, viz., intensity measure (IM), engineering demand parameter (EDP), damage measure (DM), and decision variable (DV) obtained from four steps of analysis (though the latest step is not considered in the current study). The first step of the PB framework involves introduction of proper IM parameters for the primary and cascaded events. In the current study, fire load density and duration are assumed as the IM parameters to compute the failure probability under the PBF scenario. The second step of the PB framework involves assessing the response of structure, which is defined by suitable engineering demand parameters (EDPs) for given blast pressures and/or thermal loads. Here, the structural capacity is defined in terms of a limiting deflection and rate of deflection for the RC wall panels [48]. The third and final step in obtaining the fragility curve includes a damage analysis, relating the obtained EDPs. In general, the DM parameters are chosen as superficial, moderate, heavy, hazardous, and blowout damage for blast scenario [47]; whereas, the damage can be quantified based on no damage, spalling, repairable, and irreparable, collapse, etc. for the damage under fire hazard. In this study, the fire-induced damage is studied for collapse...
mechanism, which is determined from the threshold value of limiting deformation and rate of deformation of the structural wall panels [48].

It is to be noted that the performance-based framework for PBF is developed assuming mutual exclusivity between the events, i.e., there is no correlation between the intensity measure of blast (charge weight, standoff distance, etc.) and fire (fire load density, duration, etc.) throughout the design life of the structure. Although the probability of ignition and fuel leakage significantly depends on the intensity and magnitude of blast overpressure, the growth of fire in the present study has no dependency on the probability of occurrence of blast, which is rather dependent on the fuel load and ventilation available in the compartment. Hence, the hazards are countered independently assuming that the damage induced under the blast exposure is fully propagated further to assess the performance of the wall panels under the fire.

3. Uncertainty in cascaded post-blast fire analysis

As PBF is an example of causation effect of the cascading action between blast and fire hazard, the level of complexity compels to consider the uncertainty of the parameters governing the structural design under such multiple hazards. Herein, an attempt is made to compute the structural performance for meeting the safety objectives considering uncertainty in system parameters.

3.1. Uncertainty in capacity

3.1.1. Uncertainty in material strength

The material strength, represented by characteristic strength of concrete/steel, is chosen as the parameter influencing the performance of the RC structural members under PBF. This is because due to inherent randomness in the material microstructure, there is a possibility of not replicating the exact stress-strain curves for the same material strength, which causes significant difference in the actual and nominal capacity of the structure. Different studies have considered variability of about 10%–15% in the concrete compressive strength assuming normal or lognormal distribution as best fit to obtain more realistic response [7,46]. On the other hand, basic variability of yield strength of reinforced steel bars varies from 1% to 10%. Hence, in the present study, the uncertainty is adopted in mean characteristic strength with coefficient of variation (CoV) of 10% having lognormal distribution as appropriate fit.

3.1.2. Uncertainty in thermal properties

Similarly, uncertainties in temperature dependent (thermal) properties have significant contribution to variability in the results of concrete when exposed to standard/parametric fire [3]. Research conducted by Koduri and Sultan [37] suggested that the properties influencing the 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 ($c$), which are the main parameters governing the thermal behavior in the heat transfer model.

The normal mass density of concrete, which is between 2150 kg/m$^3$ to 2450 kg/m$^3$, decreases with increase in temperature due to significant loss of moisture [20,37]. Past studies have also demonstrated that there is a significant variation in the pattern of mass loss for concretes with siliceous and carbonate aggregates, while the models available in EC2 1-2 [20] does not consider different models for various types of aggregates. Statistically, normal weight concrete is represented using normal distribution [53], and few studies have adopted 2400 kg/m$^3$ as mass density of concrete at ambient temperature [17]. Hence, for the present study, the model for mass loss prescribed by Lie and Koduri (1996) is used, which is in close conjunction with the widely accepted model of [20]; wherein the weight of concrete is adopted as 2400 kg/m$^3$ with 10% coefficient of variation (CoV) having normal distribution. Further, the thermal conductivity of normal weight concrete at ambient temperature ranges from 1.4 W/m·K to 3.6 W/m·K, which are obtained from several test data and empirical relations (Lie and Koduri, 1996; [20,35]). Such range of data demonstrates significant variability involved in the system. For the present study, upper bound values of thermal conductivity provided in widely accepted [20] are adopted for the deterministic and probabilistic analyses having 10% CoV following normal distribution.

The thermal expansion of concrete increases to about 1.3% at 700 °C and then generally remains constant through 1000 °C. High thermal expansion resulting from aggregates and cement paste in concrete induces the substantial increase from 20 °C to 700 °C. Being an important property that induces deformation, abundant research works have been conducted to demonstrate range of values for thermal expansion of normal strength concrete [20,50]. Therefore, the coefficients of thermal expansion at temperature, $T$ (°C) are obtained from the empirical relations provided by Ruan et al. [50]; having 10% CoV following normal distribution. The specific heat of concrete at ambient temperature varies from 840 J/kg·K and 1800 J/kg·K for different aggregate types [35]. The specific heat of concrete increases very gradually up to 400 °C, followed by a significant spike up to about 700 °C [20]. Hence, for the present study the model provided in the literature by Ruan et al. [50] is used, as this model overcomes the abrupt increase in the specific heat at temperature between 400 °C and 800 °C provided in the EC2 1-2 [20]. Therefore, based on the degree of uncertainty, proper values of CoVs are assumed for the probabilistic analysis of the RC wall panels under PBF. Finally, it is to be noted that in the subsequent probabilistic analysis, uncertainty is assumed for the values at ambient temperature, and accordingly the values of thermal properties are obtained for elevated temperatures. The thermal conductivity, coefficient of thermal expansion, and specific heat capacity of the steel reinforcing bars at elevated temperature are obtained from widely accepted models provided in the EC2 1-2 [20]. The degree of uncertainty and nature of distribution is adopted the same as of concrete. Finally, the deterministic thermal properties of concrete and steel at different temperatures used here can also be obtained from the study conducted by Roy and Matsagar [48].

In order to predict the behavior of fire-exposed RC wall panel after being subjected to blast loading, material properties at elevated temperatures after exposed to blast scenario (pre-damaged condition) must be explicitly known to accurately determine the behavior of the structural members. However, current literature lacks information on the material properties at elevated temperature after subjecting to a certain damage level; hence, independent material models available for strain-rate and elevated temperature under quasi-static condition are used for the current analysis.

3.2. Uncertainty in loading

In complex cases of PBF, the fire resistance may vary significantly depending on the degree of damage inflicted in the member. Such intricate scenarios require a great deal of understanding in loading process, which contributes to the structural reliability in an effective manner.

3.2.1. Uncertainty in mechanical loading

Dead load represents the typical gravity load due to self-weight of structural and non-structural components and are not generally expected to vary throughout the design life of structure. However, few studies have recommended a CoV of 8%–10% assuming the dead load follow normal distribution [22,45]. Live loads, comprising of combination of sustained and transient live loads, are relatively more uncertain as compared to the dead loads, and the statistical parameters of the live load depend on the floor area under consideration. Studies show that the combined maximum live load is modeled using Extreme Type-I distribution with CoV of 6%–28% [21,45]. Therefore, the present study considers the variation in dead load and combined live loads as 10% and 25% respectively modeled with normal and Extreme Type-I
distributions. The deterministic mechanical loads acting on the RC wall panels having some accidental eccentricity are provided in Table 1.

### 3.2.2. Uncertainty in blast loading

The blast loading is generally affected by peak pressure, specific impulse, positive phase duration, and decay coefficient, which in turn depends on mass of explosive, standoff distance, and atmospheric conditions, which surprisingly have shown a relatively higher degree of uncertainty [10]. Studies have shown that the tolerance or CoV for simple or non-commercial explosives, such as ammonium nitrate fuel oil (ANFO) may exhibit relatively higher variabilities, up to 50% assuming lognormal distribution. The variability of the system also depends on the range, i.e., standoff distance, which indicates placement of the explosives and the type of guidance system used. The CoV for stand-off ranges differently for different instances, such as, CoV = 0 for explosive ordnance storage, suicide bomber, etc., CoV = 10%–25% for a terrorist vehicle-borne improvised explosive device (VBIED), and CoV up to 30% for collateral damage estimation (CDE) from military weapons [42]. In this context, the current investigation assumes variabilities the blast loading through a mean blast load of 200 kg equivalent trinitrotoluene (TNT) at a stand-off distance of 15 m with CoV of 10% modeled using lognormal distribution. For simulating the reflected blast wave, the charge weight is multiplied by 1.8 because it is assumed that the reflected pressure increases many-folds upon encountering a rigid surface before incident on the target structure [33]. Finally, the blast pressure histories are computed using the empirical relationships provided by Kinney and Graham [34]; and the peak reflected pressure are computed using the coefficient of reflection ($C_r$) charts provided in the Unified Facilities Criteria (UFC) [57].

### 3.2.3. Uncertainty in fire loading

The cascaded fire or PBF hazard represents an uncertain technical hazard, which is modeled considering variabilities in fire ignition as well as in flashover and post-flashover phase. The fire ignition and the subsequent intensity during flashover phase depends on amount, nature, distribution, and characteristics of the materials in compartment. Moreover, the temperature attained in the compartment heavily depends on a range of variables, such as, ignition source, fire load density, compartment ventilation, and thermal absorptivity, which have great deal of uncertainty in their values. A number of temperature-time relations for fire exists in literature to simulate the behavior of fire in structure, which comprises of standard fire curves and realistic parametric fire curves [18,28]. In order to simulate the actual behavior of fire on the structure here, parametric fire curve prescribed by Eurocode, EC1 [18], is used. The temperature growth in parametric fire curve depends on fire load density ($q$), thermal absorptivity/inertia ($b$), and opening factor ($O$).

The fire load density measures the amount of energy released in a fire event based on the amount of combustible materials per unit area in the compartment. According to the guidelines prescribed by the Society of Fire Protection Engineers (SFPE), the fire load density in a compartment depends on properties and arrangement of fire loads, which has inherent randomness. According to Thomas [56], the average fire load density for dwelling systems was 900 MJ/m$^2$ with coefficient of variation in the range of 50%–80% following lognormal distribution. The EC1 [18] recommends the fire load density for residential house and dwelling system to be 780 MJ/m$^2$ with standard deviation of 234 MJ/m$^2$ considering Gumbel Type-I distribution, which is widely accepted in the probabilistic fire safety design. According to Buchanan and Abu [9], the New Zealand Building Code recommended design fire load density of 400 MJ/m$^2$ for residential buildings. It was also recommended to assume the design fire load at the 90th percentile, which was recommended to be 1.65 to 2 times the average load density. Hence, for the present study, the mean design fire load density was assumed to be 780 MJ/m$^2$ with standard deviation of 234 MJ/m$^2$ considering Gumbel Type-I distribution, as prescribed by the EC1 [18].

The thermal absorptivity/inertia ($b$) of the material is a measure of the amount of heat absorbed by the structural concrete, which depends on density ($\rho$), thermal conductivity ($\lambda$), and specific heat capacity ($c$) of concrete. As already discussed, the thermal properties of the material have significant uncertainties, which in turn affects the measure of heat absorption by concrete. Studies show that statistically normal weight concrete is represented using normal distribution considering mean of 1830 J/(m$^2$S$^2$K) and CoV of 9.4% [25,9]. Hence, in this study, the stochastic analysis is conducted assuming mean of thermal absorptivity/inertia as 1800 J/(m$^2$S$^2$K) with CoV of 10% considering normal distribution.

The opening factor ($O$) represents the ventilation settings in a fire compartment, which depends mainly on area and height of the openings governed by windows and doors in a compartment. The opening factor contributes mostly to the duration and severity of fire, for which it is reasonable to treat the opening factor to be uncertain. The value of the opening factor lies in range of 0.02 m$^{1/2}$ to 0.20 m$^{1/2}$ represented by truncated lognormal distribution. However, researchers also assumed nominal values for opening factor as mean values with a COV of 5% having normal distribution. Hence, for the present study, mean value is assumed as 0.075 m$^{1/2}$ with a COV of 10% having normal distribution.

### 4. Mechanical properties of concrete and steel under blast and fire

Blast load induces high strain-rate in RC material and in this regard, it is important to define strain-rate properties for introducing dynamic strength effect in concrete and steel materials. The blast loading causes a significantly higher strain-rate of about 10$^2$ s$^{-1}$ to 10$^4$ s$^{-1}$ as compared to static/quasi-static loading on the structure [44]; Lu et al., 2017). Under the dynamic conditions, the stress-strain response of concrete are affected by three physical mechanisms, viz., thermo-activated mechanism, macroscopic viscosity mechanism (the Stefan effect), and inertial mechanism, which causes increased dynamic strength as compared to lower static or quasi-static strain-rate effect. On the other hand, the dynamic impact also has significant effect on reinforcing steel as compared to its static state due to the evolution of dynamic dislocations affecting the microscopic scale. It is possible to estimate post-blast behavior of column using simplified approach as reported by Momeni et al. [39]. Nevertheless, such design approaches for ease in modelling need to be substantiated by detailed investigations a priori.

With increase in strain-rate, there is an increase in yield stress, ultimate tensile stress, and ultimate tensile strain; however, the material modulus of elasticity does not experience any change. Therefore, the dynamic behavior of RC materials under the strain-rate effects caused by blast loadings is captured by considering efficient dynamic increase factor (DIF) models. In this regard, models proposed by Al-Salloum et al. and Zhou and Hao [4,60] are respectively used to calculate the DIFs of concrete under compression and tension. Details of the constitutive equations for concrete in compression and tension under high strain-rate can be obtained from Roy and Matsagar [49]. The effect of blast loading on concrete and steel is simulated using the DIF values, provided in Table 2.

The PBF damage is simulated using the damage parameters for the CDP model, which are provided in Table 3. Subsequently the dynamic

### Table 1

<table>
<thead>
<tr>
<th>Thickness of Panel (mm)</th>
<th>Accidental Eccentricity (mm)</th>
<th>Mechanical Load (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Dead</td>
</tr>
<tr>
<td>75</td>
<td>30 mm towards the direction of fire exposure</td>
<td>107.5</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>125</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The PBF damage is simulated using the damage parameters for the CDP model, which are provided in Table 3. Subsequently the dynamic
compressive and tensile strengths, and modulus of elasticity of concrete are obtained by multiplying the respective DIFs with the assumed uncertain values in static/quasi-static conditions. The constitutive relation of the reinforcing steel under quasi-static state is adopted from the literature published by Silva and Lu [55]. Thereafter, the dynamic strengths of steel rebar are calculated by using the DIFs obtained from Asprone et al. [6]. Finally, Fig. 1 is plotted to illustrate the stress-strain curves of concrete and steel for different strain-rates. The relations used to compute the stress-strain curves for concrete and steel have already been reported [49].

In a similar manner, the behavior of an RC structural member exposed to fire is partly dependent on mechanical properties of concrete, which vary as function of temperatures. The mechanical properties influencing the performance of the RC structure under elevated temperatures are obtained from the study conducted by Roy and Matsagar [48]. The mechanical and thermal properties of concrete and steel at ambient temperature are also provided in Table 4, which also acts as deterministic values for the subsequent probabilistic analysis. Finally, Fig. 2 is plotted to show the stress-strain curves of concrete under compression and tension at different temperatures.

### 5. Structural description and numerical modeling

Herein, a scenario-based cascaded PBF assessment is conducted on a three-dimensional (3-D) RC wall panel having exposed planar area of 6.1 m × 4.3 m. The numerical study is conducted for three different wall thicknesses, viz., 75 mm, 100 mm, and 125 mm to investigate the fire resistance of the wall panels after being subjected to blast explosion. The structural details are adopted from the study conducted by Roy and Matsagar [48]. The RC wall panels are subjected to blast and fire loadings exposed to single-side, which are opposite to each other (Fig. 3). The blast load is assumed to act on the exterior surface of the wall panel, whereas the fire load is assumed to act on the interior surface of the wall panel, as shown in Fig. 4. Finally, it is assumed that the RC wall panels are not specifically designed to resist any accidental blast or fire loads, nor any resistive materials are applied on the panels to protect the

### Table 2

Dynamic increase factors (DIFs) used for concrete and steel.

<table>
<thead>
<tr>
<th>Strain Rates</th>
<th>Concrete compression</th>
<th>Concrete tension</th>
<th>Steel tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1.02</td>
<td>3.56</td>
<td>1.18</td>
</tr>
<tr>
<td>100</td>
<td>1.43</td>
<td>7.00</td>
<td>1.21</td>
</tr>
<tr>
<td>1000</td>
<td>2.74</td>
<td>11.90</td>
<td>1.24</td>
</tr>
</tbody>
</table>

### Table 3

Damage parameters used in CDP model under compression for high strain rate and thermal loadings.

<table>
<thead>
<tr>
<th>Loading</th>
<th>Compression Damage ((d_c))</th>
<th>Plastic Strain ((\varepsilon_{pl}))</th>
<th>Temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blast</td>
<td>0.21618138</td>
<td>0.00257</td>
<td>–</td>
</tr>
<tr>
<td>Fire</td>
<td>0.009779</td>
<td>0.931</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>0.013162</td>
<td>0.967</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>0.666372061</td>
<td>0.00584</td>
<td>500</td>
</tr>
</tbody>
</table>

Fig. 1. Stress-strain curves of concrete and steel for different strain rates.
5.1. Finite element (FE) modeling of RC wall panel

Herein, a finite element (FE) approach is adopted to investigate each RC wall panel under cascaded PBF, mechanics of which has been mentioned with complete details by Ref. [49]. The probabilistic assessment is carried out in commercially available software ABAQUS®, which is used to solve the FE problem scripted in Python programming interface. The main advantage of using the Python program is that the time-consuming multiple simulations are carried out without the intervention of user. The FE model of each RC wall is developed by the use of predefined field becomes relatively limited when there is a change in geometry in both the phases.

The mesh of concrete geometry generated in both the phases, Phase-1 and Phase-2, of the FE analysis is created by coupled temperature-displacement continuum C3D8RT element with reduced integration. As the C3D8RT element has DOFs 1, 2, 3, and 11, DOF 11 is suppressed for Phase-1 dynamic stress analysis, whereas in the subsequent Phase-2 coupled temperature-displacement analysis, DOF 11 is allowed to obtain the response related to the temperature DOF. Truss element T3D2T is used to mesh the geometry of steel rebar in both the phases, which includes a temperature DOF required for the cascaded thermo-mechanical analysis in Phase-2. Each RC wall is fixed at the bottom and clamped on other three sides. Thus, the boundary conditions in each RC wall panel are given as: (i) bottom face: DOFs - $u_x (1) = u_y (2) = u_z (3) = \phi_y (4) = \phi_n (5) = \phi_n (6) = 0$, and (ii) remaining sides: DOFs-$u_x (1) = u_y (2) = u_z (3) = 0$. Here, DOFs (1, 2, 3, and 11) are standard notations used in ABAQUS® solver [1].

The blast loading on each RC wall panel is applied as pressure loading at the external surface of exposure because it is assumed that the blast event is external. On the other hand, the thermal loading is applied at the internal face as it is assumed that the blast event may trigger compartment fire due to combustibles inside the room (Fig. 4). The entire RC wall panel under thermo-mechanical scenario is subjected to an ambient temperature of 20 °C. Finally, each thermo-mechanical analysis is carried out up to a duration of 120 min and the response at each node is stored for post-processing.

### Table 4

Mechanical and thermal properties of concrete and steel at ambient temperature.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Properties</th>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Mechanical properties</td>
<td>Compressive strength of concrete ($f_c$)</td>
<td>30 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Elastic modulus of concrete ($E_c$)</td>
<td>27386.12 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coefficient of expansion ($\alpha$)</td>
<td>$6.16 \times 10^{-6}$/°C</td>
</tr>
<tr>
<td>Thermal</td>
<td>Density ($\rho_c$)</td>
<td>2300 kg/m³</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Conductivity ($\lambda_c$)</td>
<td>1.95 W/(m·K)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specific heat ($c_c$)</td>
<td>913.22 J/(kg·K)</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>Mechanical properties</td>
<td>Yield strength of steel ($f_y$)</td>
<td>415 MPa</td>
</tr>
<tr>
<td></td>
<td>Ultimate strength of steel ($f_u$)</td>
<td>621 MPa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Coefficient of expansion ($\alpha$)</td>
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<td></td>
</tr>
<tr>
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<td>Elastic modulus of steel ($E_s$)</td>
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</tr>
<tr>
<td></td>
<td>Density ($\rho_s$)</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Conductivity ($\lambda_s$)</td>
<td>53.34 W/(m·K)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specific heat ($c_s$)</td>
<td>436.09 J/(kg·K)</td>
<td></td>
</tr>
</tbody>
</table>

Nonlinearity in concrete is considered using continuum, plasticity-based, damage model for concrete, or otherwise known as concrete damaged plasticity (CDP) model that utilizes isotropic damaged elasticity along with isotropic tensile and compressive plasticity to represent the nonlinear behavior (cracks and post-cracking behavior) of concrete. The advantage of the CDP model is that the model can be used to define strain-rate and temperature dependent properties. The parameters of the CDP model are presented in Table 5. The degradation of the elastic stiffness is characterized by two damage variables, $d_c$ and $d_s$, which are assumed to be functions of plastic strains and/or temperature. On the other hand, the reinforcing bars are modeled as discrete embedded truss element tied to the concrete region by one-dimensional (1-D) element. The post-yielding behavior of steel rebar is captured using classical metal plasticity model involving von-Mises yield criterion with associated plastic flow and isotropic hardening. The behavior of reinforcing steel embedded in concrete is achieved from the plastic values of the stress-strain curve and the rate dependent parameters ($D$ and $n$) that define the failure of the material. For the present scenario, $D$ and $n$ are respectively assumed as 20 and 5 to simulate the damage in steel reinforcing bar. Full bonding is assumed between steel rebar and concrete, which is achieved using the tie constraint option.

The cascaded thermo-mechanical analysis of each RC wall panel exposed to fire subjected to pre-damage under blast loading is carried out in two phases, viz., Phase-1 and Phase-2. The effect of degradation in the form of pre-damage in Phase-1 is simulated using predefined field option available in ABAQUS® solver [1], which readily imports the pre-damage state of the RC wall panel to Phase-2. It must be noted that the use of predefined field becomes relatively limited when there is a change in geometry in both the phases.

The mesh of concrete geometry generated in both the phases, Phase-1 and Phase-2, of the FE analysis is created by coupled temperature-displacement continuum C3D8RT element with reduced integration.

The advantage of the CDP model is that the model can be used to define the failure of the material. For the present scenario, $D$ and $n$ are respectively assumed as 20 and 5 to simulate the damage in steel reinforcing bar. Full bonding is assumed between steel rebar and concrete, which is achieved using the tie constraint option.

The cascaded thermo-mechanical analysis of each RC wall panel exposed to fire subjected to pre-damage under blast loading is carried out in two phases, viz., Phase-1 and Phase-2. The effect of degradation in the form of pre-damage in Phase-1 is simulated using predefined field option available in ABAQUS® solver [1], which readily imports the predefined field into ABAQUS® solver [1].

The cascaded thermo-mechanical analysis of each RC wall panel exposed to fire subjected to pre-damage under blast loading is carried out in two phases, viz., Phase-1 and Phase-2. The effect of degradation in the form of pre-damage in Phase-1 is simulated using predefined field option available in ABAQUS® solver [1], which readily imports the pre-damage state of the RC wall panel to Phase-2. It must be noted that the use of predefined field becomes relatively limited when there is a change in geometry in both the phases.

The mesh of concrete geometry generated in both the phases, Phase-1 and Phase-2, of the FE analysis is created by coupled temperature-displacement continuum C3D8RT element with reduced integration. As the C3D8RT element has DOFs 1, 2, 3, and 11, DOF 11 is suppressed for Phase-1 dynamic stress analysis, whereas in the subsequent Phase-2 coupled temperature-displacement analysis, DOF 11 is allowed to obtain the response related to the temperature DOF. Truss element T3D2T is used to mesh the geometry of steel rebar in both the phases, which includes a temperature DOF required for the cascaded thermo-mechanical analysis in Phase-2. Each RC wall is fixed at the bottom and clamped on other three sides. Thus, the boundary conditions in each RC wall panel are given as: (i) bottom face: DOFs - $u_x (1) = u_y (2) = u_z (3) = \phi_y (4) = \phi_n (5) = \phi_n (6) = 0$, and (ii) remaining sides: DOFs-$u_x (1) = u_y (2) = u_z (3) = 0$. Here, DOFs (1, 2, 3, and 11) are standard notations used in ABAQUS® solver [1].

The blast loading on each RC wall panel is applied as pressure loading at the external surface of exposure because it is assumed that the blast event is external. On the other hand, the thermal loading is applied at the internal face as it is assumed that the blast event may trigger compartment fire due to combustibles inside the room (Fig. 4). The entire RC wall panel under thermo-mechanical scenario is subjected to an ambient temperature of 20 °C. Finally, each thermo-mechanical analysis is carried out up to a duration of 120 min and the response at each node is stored for post-processing.

**Fig. 2.** Stress-strain curves of concrete under (a) compression and (b) tension at different temperatures.
5.2. Numerical solution scheme

For the present FE study, two different numerical solution schemes are implemented for two different phases, which are, explicit dynamic analysis for Phase-1 and coupled displacement-temperature analysis for Phase-2. The dynamic analysis of the RC wall panel under blast loading is performed in explicit domain as it is more efficient for solving wave propagation problems, whereas, standard domain in ABAQUS® is found more efficient for solving smooth nonlinear thermo-mechanical problems. In both explicit and standard domains, the nonlinear response is obtained incrementally. The discretized equilibrium equation in FE environment is expressed as,

\[
\{P\}^t - \{I\}^t = [M]\{\ddot{u}\}^t
\]

(3)

where, \(\{P\}^t\) is external force vector, \(\{I\}^t\) is internal force vector created from element stresses, \([M]\) is diagonal lumped nodal mass matrix, \(\{\ddot{u}\}^t\) is

Table 5
Parameters used for the concrete damaged plasticity (CDP) model.

<table>
<thead>
<tr>
<th>Dilation Angle ((\psi))</th>
<th>Eccentricity ((e))</th>
<th>(f_{b0}/f_{co})</th>
<th>(K_c)</th>
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</thead>
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<td>36.31</td>
<td>0.1</td>
<td>1.16</td>
<td>0.667</td>
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</tbody>
</table>

Fig. 3. 3-D view of wall panel showing blast and fire exposure along with sectional cross-section view showing reinforcement details.

Fig. 4. 3-D and sectional view showing blast and fire exposure.
nodal acceleration at beginning of the increment, and \([M](\ddot{u})\) is force vector due to material inertia.

Explicit central difference integration scheme is used to update the velocity and displacement at each node, which is expressed as,

\[
\dot{u}_i^{n+1} = \dot{u}_i^n + \left( \frac{\Delta t}{2} \right ) (\ddot{u}_i^n - \ddot{u}_i^n) + \left( \frac{\Delta t}{2} \right ) (\ddot{u}_i^{n+1} + \ddot{u}_i^n).
\]  

(4)

\[
\ddot{u}_i^{n+1} = \ddot{u}_i^n + \left( \frac{\Delta t}{\Delta x} \right ) (\ddot{u}_i^{n+1} - \ddot{u}_i^n) - \left( \frac{\Delta t}{\Delta x} \right ) (\ddot{u}_i^{n+1} + \ddot{u}_i^n).
\]  

(5)

The central difference integration scheme is conditionally stable and the solution becomes unstable and diverges rapidly if the time increment is too large. Hence, the time increment must be smaller than the stability limit for the operator in terms of Eigen value is given by,

\[
\Delta t \leq \frac{2}{\omega_{\text{max}}} \sqrt{1 + \varepsilon_t^2 - \xi_t^2}
\]  

(6)

where, \(\omega_{\text{max}}\) represents maximum Eigen value of the element, and \(\xi_t\) represents fraction of critical damping in the highest mode. The stable time increment can be expressed as, \(\Delta t \leq \min \left( \frac{L_e}{c_d} \right)\), where, \(L_e\) is characteristic length of the smallest element in the domain, \(c_d = \sqrt{\frac{\varepsilon_t^2 + \xi_t^2}{\rho}}\) is dilatational wave speed, where \(\lambda\) is first Lamé constant, \(\mu\) is shear modulus, and \(\rho\) is the density of the element, chosen automatically to satisfy the user-defined critical time step. Here, the damping is introduced solely to reduce the stable time increment for the solution in evaluating the dynamic response. In this case, a small amount of damping in the form of bulk viscosity is introduced to control high frequency oscillations. The stress parameters are not influenced by the introduction of the bulk viscosity since this is incorporated as a numerical effect and not as a part of constitutive law for material. The bulk viscosity depends on the dilatational mode of each element and the fraction of critical damping of each element, which is given by,

\[
\xi_t = b_1 - b_2 \frac{L_e}{C_d} \min \left( 0, \varepsilon_{vol} \right)
\]  

(7)

where, \(b_1\) and \(b_2\) are damping coefficients having values of 0.06 and 1.2, respectively, and \(\varepsilon_{vol}\) is the volumetric strain-rate.

In the subsequent coupled temperature-displacement analysis (Phase-2) for the RC wall panel, the temperature history is applied to the exposed surface of the RC wall through convection and radiation considering damage induced by the blast loading. The heat is transferred within the RC wall by means of heat conduction and heat transferred to the wall is through heat convection and radiation mechanism. The transient heat conduction equation along with the boundary condition and the convection condition can be expressed as,

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

(8)

\[
(-k \nabla T) \cdot n = h_l (T^l - T_i)
\]  

(9)

where, \(Q\) is overall heat transferred from outside, \(\rho\) is density of material (kg/m³), \(c\) and \(k\) are the specific heat capacity (J/kg K⁻¹) and thermal conductivity (W/m K⁻¹) respectively, \(n\) is unit vector outer normal to the boundary, \(h_l\) is convective heat transfer coefficient, and \(T_i\) and \(T^l\) are ambient temperature and temperature at reference surface, respectively. It can be observed that the transient conduction problem follows parabolic relation with time dependence and elliptic behavior with spatial coordinates. In order to have smooth control over the analysis technique, it is ensured that local instabilities, such as material instability or local buckling are not developed throughout the different phases of the analyses. Hence, damping is introduced throughout the model in such a way that the generated viscous forces are sufficiently large to prevent instantaneous buckling without altering the stability. In the present study, ‘automatic stabilization scheme’ with a constant damping factor available in ABAQUS® [1] is used to typically subside the instabilities and to eliminate rigid body modes without having a major effect on the solution.

It is critical to note that in such explicit nonlinear simulations in Phase-1, the structural elements undergo very large deformations and distort the FE mesh ending in a point of providing inaccurate results due to numerical reasons. In this regard, it is necessary to increase the mesh size to minimize the distortion in the structure. On contrary, for implicit nonlinear simulation techniques in Phase-2, the increment size depends on the rate of convergence and in such highly nonlinear problems due to slow convergence, the increment size needs to be decreased. Therefore, the mesh size in the RC wall geometry should be chosen appropriately to counter the extreme nonlinearity under both the extreme hazards, viz., high strain-rate loading due to blast and the subsequent thermo-mechanical loading.

6. Numerical study

The current study involves investigation of 3-D RC wall panels under PBF by first exposing the RC wall panels to dynamic blast loading and subsequently to PBF loading. Mesh convergence trials are conducted to determine optimum mesh size for obtaining significant results, and in this regard, the mesh size obtained for the current analysis is 60 mm × 60 mm × 60 mm. The validation for modeling strategy of the RC wall panels independently under fire and blast loadings is appropriately conducted using the studies of Ngo et al. and Jain et al. [30, 43], respectively. The structural response obtained for the PBF loading are compared against the NF. The thermo-mechanical response of the RC wall panels are studied in terms of deflection (\(\delta_b/\delta_b\)), rotation (\(\theta\)), principal stress (\(\sigma_{11}\)), tensile damage (\(d_1\)), and plastic strain (\(\varepsilon_p\)) at the center of the RC wall panels; where, \(\delta_b\) denotes out-of-plane deflection under blast loading, and \(\delta_f\) denotes deflection under NF/PBF loading. Meanwhile, Tables 6 and 7 provide the complete tabulated data for the

<table>
<thead>
<tr>
<th>Properties</th>
<th>Parameters</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean Values</th>
<th>CoV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanical properties (concrete and steel)</td>
<td>Strength of concrete/steel</td>
<td>MPa</td>
<td>Lognormal</td>
<td>(Van Coile et al., 2013)</td>
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</tr>
<tr>
<td>Elastic modulus of concrete/steel</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coefficient of expansion ((a))</td>
<td>/C</td>
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<td>Deterministic</td>
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<td>Reinforcement details</td>
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<td></td>
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<td>Thermal properties (concrete and steel)</td>
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<td>Normal [53]</td>
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<tr>
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<td>[17]</td>
<td></td>
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<tr>
<td>Stefan-Boltzmann Constant</td>
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<td>Convection</td>
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<td></td>
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<td>Radiation/emissivity</td>
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</table>
probabilistic study, which is based on the deterministic parameters. Moreover, the sets of blast and fire loadings are presented in Fig. 5 to show the range of the peak reflected overpressure and maximum temperature attained. Lastly, Fig. 6 is presented to demonstrate the algorithm to conduct the cascading hazard analysis under PBF scenario.

6.1. Result and discussion based on deterministic study

Herein, deterministic study is conducted for the RC wall panels considering the mean values of assumed distribution. Moreover, the NF and PBF deflection response for the singly-reinforced panel is compared with the doubly-reinforced panel, wherein the RC walls have been designed with a layer of reinforcement mesh in the compression side under the in-plane (axial) load. Each structural panel is embedded with two layers of steel reinforcement in concrete having clear cover of 25 mm on both sides, as shown here in Fig. 7. The reinforcement type and spacings have been kept as that of the singly-reinforced RC wall to facilitate comparison. Response histories are plotted for blast, NF, and PBF deflection for the singly-reinforced panel is compared under the scenario-based blast loading. The peak deflections are obtained as 538.17 mm, 305.65 mm, and 178.22 mm, respectively for the 75 mm, 100 mm, and 125 mm thick panels. With increase in thickness of the panels by 25 mm, the peak blast deflections reduce by ~70%–80%. Moreover, residual deflections for the wall panels are also investigated to show the degree of damage induced in the panels. The residual deflections are obtained as 505.45 mm, 223.18 mm, and 108.15 mm, respectively for 75 mm–125 mm panel thicknesses under the deterministic blast load. The residual response increase by ~ 2.0–4.7 times on decreasing the panels from 125 mm to 75 mm, which indicates that the blast loadings has induced ~2.0 to 4.7 times more damage in 75 mm and 100 mm as compared to the 125 mm thick panel. Furthermore, stress ($\sigma_{11}$) history at the unexposed side of the wall panels show that the maximum stresses developed in the 75 mm thick panel is substantially more under the considered blast scenario. The peak stress increases from ~ 1.5 to 2.2 times for the panels with thicknesses increasing from 75 mm to 125 mm. Therefore, based on such exposure of blast intensity, it is recommended to use at least 125 mm thick wall panels in civil-structural engineering applications.

Fig. 9 illustrates the relative out-of-plane deflections ($\delta_b$) obtained at center of the singly-reinforced wall panels under the NF and PBF scenarios. The deflection profile in case of PBF scenario is considered relative to the origin of axis, although the final deflection state under the previous blast scenario shows a significant amount of deformation. The deflection profiles for the RC wall panels under the NF are compared with PBF exposure, which indicate a significant difference in the behavior of RC wall panels eventually affecting the fire resistance of the panels. It is interesting to observe that the deflection profiles under the PBF scenario do not follow the same path as compared to that under the NF scenario. This is due to the damage inflicted in the panels which causes reduced strength and stiffness, thus altering the deflection path. Moreover, the panels under the PBF exposure show lower thermal bowing as compared to the NF indicating the reduced capacity of the pre-damaged panels. The peak response for the panels from 75 mm to 125 mm under NF are 150.91 mm, 83.01 mm, and 35.70 mm, respectively, whereas the peak response under the PBF are respectively obtained as 186.46 mm, 105.27 mm, and 31.06 mm. The deflection profiles are also used to define the fire resistance ratings considering the threshold horizontal out-of-plane deflection value obtained as $h/100 = 43$ mm for RC wall panel under compressive in-plane load; where, $h$ represents the height of the member [48]. The fire rating decreases by ~18% and ~53% respectively for the 75 mm and 100 mm thick wall panels. Therefore, the importance of considering the cascading hazard scenario needs to be understood and addressed explicitly in such regions where extreme threats are significant.

Fig. 10 presents the out-of-plane deflections at the center of the doubly-reinforced wall panels under the normal fire (NF) and post-blast fire (PBF) scenario. The deflection profiles of the RC walls reinforced on two sides demonstrate similar behavior as compared to the deflections of the RC walls having single side reinforcement (singly-reinforced). Primarily, there is a decrease in the maximum deflection in the RC walls

---

**Table 7**

<table>
<thead>
<tr>
<th>Loading</th>
<th>Parameters</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean Values</th>
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</tr>
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<td>Scaling factor due to ground</td>
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</tr>
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<td>Fire loading</td>
<td>Thermal inertia (k)</td>
<td>J/ (m³/²)</td>
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<td>0.1</td>
</tr>
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<td></td>
<td>Fire load density (g)</td>
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<td>Duration of fire loading (d)</td>
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<td></td>
<td>Opening factor (O)</td>
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</tbody>
</table>

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![Fig. 5](image-url) Sets of blast and time temperature history curves used in the study.
with two-sided reinforcement (doubly-reinforced, as plausible from the increase in the capacity of the structural systems. From the PBF deflections, it is observed that the walls experience lesser thermal bowing as compared to the NF deflections. The peak response of the 75 mm, 100 mm, and 125 mm thick wall panels under the NF scenario are obtained as 170 mm, 95 mm, and 12 mm, respectively, whereas, the peak response under the cascaded fire are obtained as 150 mm, 57 mm, and 8 mm, respectively. The fire rating has also increased for the doubly-reinforced RC walls. For the 75 mm thick wall panel, the fire rating decreased from 1.30-h to 1.14-h (14%).

Fig. 11 illustrates tensile damage response ($d_t$) at the unexposed faces of singly-reinforced RC wall panels under the NF and PBF scenarios. The PBF scenario causes significantly more damage as compared to the NF scenario. Under the NF scenario, the damage level for the RC wall panels does not exceed 50%; however, the damage at unexposed face of 75 mm thick wall exceeds 50% and up to 74%, although the exceeded region is extremely small. On contrary, under the cascading fire scenario, the tensile damage does not exceed 72%; however, the damage in 75 mm
thick wall is relatively more which exceeds 72% and goes up to 99%, although the exceeded region is similarly small. The tensile damage due to PBF increases by around 22% at the center, and by around 25% near the edges. Such relative increase in damage due to the effect of dynamic blast loading significantly alters the performance criteria of structures under PBF loading, which requires reconsideration of design guidelines for structural safety in regions where such extreme events occur.

6.2. Result and discussion based on probabilistic study

The influence of uncertainty in the obtained response (θ, δf/δi, σ11, and εp) is investigated from the box plots, probability density function (PDF) plots, and fragility curves. Fig. 12 shows the extent and state of damage induced in the singly-reinforced wall panels under the uncertain blast loading. The damage states are obtained from the Department of Defense manual of the United States of America (PDC-TR-06, 2008) The 75 mm thick wall singly-reinforced panel is induced mostly with moderate and heavy damage, with more than 75% of the response falling under hazardous damage, and more than 20% demonstrate blowout damage. Similar observation is carried out for the 100 mm and 125 mm thicker wall panels. Furthermore, the mean and deterministic response are compared to observe the difference in degree and nature of damage induced in the wall panels. The mean and deterministic response for the 75 mm thick panel demonstrate hazardous failure. Interestingly, the mean response for 100 mm panel demonstrates heavy damage, which on contrary, the deterministic response demonstrates hazardous failure. Similar observation is seen for the 125 mm thick panel, which demonstrates superficial damage for mean response, and moderate damage for deterministic response. Therefore, it is evident that the prescriptive guidelines utilizing the deterministic approach are relatively lesser rational and may overestimate the design values, which may not essentially evaluate the structural response accurately.

Fig. 13 presents the box plots to show the degree of uncertainty in the response of the mechanically loaded singly-reinforced RC wall panels under the NF and PBF scenarios.
The peak displacement response from 5% to 95% for the 75 mm panel response of the 125 mm thick panel is the smallest, which is expected. Moreover, the stress ($\sigma_{11}$) and plastic strains ($\varepsilon_p$) for the singly-reinforced RC wall panels of different thicknesses under NF and PBF scenarios. Furthermore, the stress ($\sigma_{11}$) and plastic strains ($\varepsilon_p$) for the doubly-reinforced RC panels of different thicknesses under NF and PBF scenarios. The peak deflections for the 75 mm thick panel are marginally skewed, whereas the same for the 100 mm thicker panel has substantial degree of skewness. This is because the mechanical loading has greater influence in determining the deflection behavior of the RC wall panels. Furthermore, the stress ($\sigma_{11}$) and plastic strains ($\varepsilon_p$) at the unexposed side decrease with increase in thickness of the panels under NF scenario. The extent of uncertainty is also observed relatively higher for all the panels. On contrary, the response obtained under the PBF show a significant level of uncertainty as observed from the difference in the positions of mean and median values. This is because of the extent of uncertainty in the blast loading which contributes to the response obtained for the PBF scenario. Therefore, the structural capacity, heat transfer properties, and mechanical, blast, and fire loadings induce significant uncertainties in the response of RC wall panels under the thermo-mechanical loading scenarios.

Fig. 10. Comparison of the relative out-of-plane deflections of the doubly-reinforced RC panels of different thicknesses under NF and PBF scenarios.

Fig. 14 shows the out-of-plane deflections ($\delta_i$) for the singly-reinforced 75 mm thick RC wall panel under NF and PBF scenarios. The response histories are plotted to show the extremities of the curves including the response statistics obtained due to the uncertainties involved in the system. It is to be noted that the response histories for the PBF scenario are plotted relative to the initial position. This is because the residual displacements due to the previous blast scenario have extremely wider range, for which the comparison of the response in absolute scale becomes less meaningful. The figures indicate mean response (50th percentile) along with 84th and 95th percentile response showing the significance of data set. It is observed that the fire resistance of the RC wall panel lies from 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) while considering a practical range of uncertainty. On the other hand, the fire resistance for the mechanically loaded RC wall panel under PBF is observed in between 0.136-h (~9 min) to 1.728-h (~104 min). The mean and deterministic values are respectively obtained as 0.948-h (~57 min) and 1.02-h (~57 min). The probable reason for obtaining extremely lower fire resistance is due to accumulation of significant residual forces under the blast scenario, which in turn is inflicted with substantial damage, thus lowering the fire resistance extensively.

It is also observed that in few cases, there occurs abrupt failure indicating premature failure of the system with no load carrying capacity. Moreover, with degradation in strength and stiffness, most of the response do not experience thermal bowing effect (observed mostly under NF scenario), as the combined action of fire and mechanical loading is significant on the degraded structural member. This can also be observed from the slope of the mean response, which is steeper than the response under NF. The differences in the fire resistance durations range from 8% to 82%, indicating the degree of uncertainty involved in the system. The current guidelines and design standards conforming to fire safety of structures recommend achieving fire rating through specific thickness, aggregate ratio, and concrete cover. Moreover, the guidelines do not mention regarding the extent of decrease in fire resistance under such cascaded action, which should be helpful to have a knowledge of the response time for evacuation. Hence, it is imperative to consider the uncertainty in the system to obtain reliable design values for structures expected to counter multiple hazards in design life.

Fig. 15 shows the PDFs of maximum out-of-plane deflections at the center of singly-reinforced RC wall panels under NF and PBF. The response follow lognormal distribution as observed from the skewness of the data set. The peak deflections for the 75 mm thick panel are significantly larger as compared to the other walls. The mean deflection under the NF is obtained as 137.27 mm, whereas the mean deflection under PBF considering pre-damage is obtained as 150.35 mm. Similarly, for the other two panels with increasing thicknesses, the mean deflections when exposed to the NF are 80.23 mm and 42.56 mm, respectively, whereas the response under the PBF are respectively obtained as 94.06 mm and 42.56 mm. The relative peak displacement, in such scenario under the PBF, increases by ~9%–~14%.

Moreover, the mean values are further compared with the deterministic values to observe the effect of uncertainties in the system. The difference in mean and deterministic response are observed to be in the range from ~9% to ~16% under the NF exposure, whereas the same
quantity is obtained to be in the range from ~6% to ~18% under PBF. This shows that the effect of uncertainty is relatively more pronounced in the case of PBF. Finally, the coefficient of variations (CoVs) of the deflection response under the NF exposure is obtained as 10%–18%, whereas, the CoVs are observed to be in the range of ~32%–~87% under the PBF.

Fig. 16 illustrates the PDFs of principal stresses for the singly-reinforced RC walls at unexposed sides under the NF and PBF scenarios. Similar to the PDFs for the deflection response, the stresses under the PBF are comparably higher for the 75 mm thick panel. Upon comparison with the mean response under the NF, it is observed that the stresses at the unexposed side increase from ~8% to 60%. This indicates that the degradation increases under the PBF resulting into increased deflections. The difference in the mean and deterministic values are also studied, which shows that the effect of uncertainty creates a difference from about 5% to 83% in the values. The CoVs of the stresses under the PBF are significantly higher and ranges from ~34% to ~62%, which effectively demonstrates the influence of input uncertainties in the output response of the structural members.

From the PDFs it can be concluded that owing to increased response in the structure under the PBF, the level of protection in terms of structural design need to be carefully assessed. Finally, it is also concluded that based on the extent of uncertainty as observed in the responses, the traditional way of prescriptive design approach may not provide robust structural solutions where multiple hazards dominate.

Fig. 17 shows the probability of failure ($p_f$) curves for the singly-reinforced RC wall panels under NF and PBF scenarios based on fire load density ($q$) as intensity measure parameter. The fragility curves indicate that the vulnerability increases with decrease in thickness of the panels and the RC wall panels exposed to PBF show higher vulnerability as compared to the RC wall panels under NF. Moreover, the failures for the RC wall panels of same thickness under NF and PBF are almost same at relatively higher fire load density because irrespective of the damaging conditions, the panels can barely withstand any further load causing significant failure. For example, the 75 mm RC wall panel under both the fire exposure has almost similar failure at $q = 780 \text{ MJ/m}^2$, whereas the similar failure for 100 mm thick panel starts occurring at
Further, the fragility curves also demonstrate that the fire load density required to initiate failure for the 75 mm thick panel under NF is obtained as 120 MJ/m$^2$, whereas under the PBF, the fire load density required is \(\sim 40\) MJ/m$^2$. Similarly, the fire load density required to initiate failure for the 75 mm and 100 mm thicker panels under NF are respectively obtained as 470 MJ/m$^2$ and 630 MJ/m$^2$. In contrary, the fire load density required for the PBF are observed as 330 MJ/m$^2$ and 520 MJ/m$^2$. In this regard, \(\sim 17\%–67\%\) lesser fire load density is required to initiate failure when the structure is already damaged to certain extent, which is quite significant. Apart from the explosion load, structural damage can initiate from already imposed mechanical loading and lifecycle degradation, which should alter the fire resistance of the structure. Hence, it is imperative to address the increased damage under such cascading hazard scenario to make robust design guidelines for assessment of structures.

Furthermore, effect of uncertainty in the system is also observed from the range of CoVs obtained for constructing the fragility curves. The CoVs of the curves are obtained in the range from 0.226 to 0.681, which indicates the influence of uncertainty governing the structural behavior of the RC wall panels. Finally, the expected failure of the RC wall panels is studied considering the deterministic fire load density for dwelling house systems ($q = 780$ MJ/m$^2$). The PBF exposure initiates significant failure in the RC wall panels as compared to the NF, for which the difference in the failure probability ranges from \(\sim 3\%\) to 60%. Moreover, the RC wall 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 under both the fire exposure. Hence, irrespective of the damaging concerns, it is recommended to use RC wall panel of 125 mm to ensure fire protection for the dwelling houses.
under the PBF scenario, which is almost same as that of the 100 mm thick RC wall panel. However, the failure probability for the 100 mm thicker panel is significantly lower than the 75 mm thicker panel with increase in duration, indicating the increase in failure is relatively lower for same increase in duration. The failure initiates at about 1.1-h (66 min) and 0.8-h (48 min) for the 125 mm thick panel respectively under NF and PBF scenarios. Therefore, the panels under PBF take about ~27%–~35% lesser time to initiate failure, which is quite substantial considering the fire safety of the structure. Hence, the decrease in time should be a warning for obtaining adequate time to control the fire by...
rescue teams and safely evacuate the inhabitants in the structure. Finally, an attempt is also made to obtain the fire resistance rating from the fragility curves at 50% probability of failure. The fire resistance of the RC wall panels under the NF scenario is obtained from 1.13-h (~68 min) to 2.41-h (144 min), whereas, the fire resistance under the PBF is obtained from 0.95-h (~57 min) to 2.14-h (~128 min). Hence, the fire resistance decreases by ~11%–23% for the panels under the cascaded hazard scenario. From the deterministic 2-h fire duration, the fire resistance decreases by ~11%–23% for the panels under the NF and PBF scenarios. The developed fragility curves are effective in predicting the structural failure corresponding to a predefined degree of damage. The reliability assessment in this case should further benefit the building community for investigating the probable failure when such initial damage is expected. This framework should further pave a way for the futuristic performance-based design of structures in PBF. Finally, the developed stochastic framework is also suitable to construct analytical fire fragility functions for other different typologies of the RC framed structures.

7. Conclusions

To conclude, a novel probabilistic framework is outlined to compute the vulnerability of RC wall panels under cascading fire scenario which is pre-damaged by blast explosion. The wall panels are modeled in finite element (FE) domain and analyzed considering material and geometric nonlinearities. Uncertainties are assumed in material properties, thermal properties, and loading (mechanical, blast, and fire) of the system to study the vulnerability of the RC wall panels. The fire resistance of the panels is discussed, thereby demonstrating the necessity of implementing the cascading hazard scenario in the existing design guidelines. Finally, the proposed modeling strategy is observed to be effective in simulating the complex post-blast fire effects in the RC panel members. In this case, the proposed approach can be straightaway adopted for investigating the post-blast fire behavior for other structural systems. Hence, based on the research conducted, the major conclusions drawn are as follows:

1. The panels under PBF require ~27%–35% lesser time to initiate failure, which is quite substantial considering the fire safety of structure. Hence, the relative decrease in time should be kept in mind by rescue teams for having adequate time to control the fire and safely evacuate the inhabitants from the PBF damaged structure. Therefore, this extent of increase in damage due to the blast scenario is extremely crucial and should be considered in the current design guidelines for resilient built infrastructure.

2. The fire resistance decreases by ~11%–23% for the panels under the cascading hazard scenario. Therefore, the importance of considering the cascading hazard scenario needs to be understood and addressed explicitly in such regions where extreme threats are significant.

3. The vulnerability increases with decrease in thickness of the RC wall panels and the panels exposed to PBF show relatively higher vulnerability. The probability of failure for the RC wall panels of same thickness under NF and PBF scenarios are almost same at relatively higher fire load density because irrespective of the damaging conditions, the panels can barely withstand any further load after significant fire exposure.

4. About 17%–67% lesser fire load density is required to initiate failure of the RC wall panels when the panels are already damaged to certain extent, which is quite significant. Apart from the explosion load, structural damage during design life can initiate from already imposed mechanical loading and lifecycle degradation, which should alter the fire resistance of the structure. Hence, it is imperative to address the increased risk under such pre-damaging scenarios for making robust design guidelines and assessment of structures.

5. The structural capacity, heat transfer properties, and mechanical, blast, and fire loadings induce significant uncertainties in the response of RC wall panels under the thermo-mechanical loading scenarios. In this regard, the prescriptive guidelines utilizing the deterministic approach are relatively lesser rational and may overestimate the design values, which may not essentially evaluate the structural response accurately.

6. Irrespective of the damaging concerns, it is recommended to use RC wall panel of 125 mm to ensure fire protection for the dwelling houses.

Author contributions

Tathagata Roy: Formal Analysis; Software; Data Curation; Investigation; Visualization; Writing - Original Draft. Vasant Matsagar: Conceptualization; Methodology; Project Administration; Resources;
Compliance with ethical standards (conflict of 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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