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A framework for probabilistic fire loss estimation in concrete building structures

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ABSTRACT

A framework is proposed for the probabilistic estimation of yearly economic losses due to fire in concrete building structures. The fire loss estimation accounts for the uncertainties in the occurrence and growth of a fire as well as the response of the building. The assessment performs a fire hazard analysis, response analysis, damage analysis, and loss analysis. The response analysis relies on three-dimensional finite element modeling of the building structure. The expected direct loss for the building is determined by summing the expected losses under fires in different locations, weighted by the annual probabilities of fire occurrence in each location. To achieve this goal, we propose fire-specific engineering demand parameters (EDP) that are measurable and associated with damage states. One EDP addresses section damage due to temperature penetration, while a second EDP addresses component damage linked to deformations. We also define a set of fragility functions and consequence functions based on the selected damage states. The presented framework is applied to a case study of a five-story reinforced concrete frame building. Direct losses are evaluated at about 188 k$ for scenarios of single-compartment fire, conditional to the occurrence of severe fire. Losses are mostly related to nonstructural components and content. Although the case study focuses on single-compartment fires, losses in case of fire spreading within the building can be incorporated as well using event tree analysis with the conditional probability of the respective fire scenarios. The yearly fire loss framework presented in this paper can be adopted for other types of buildings and can be integrated into the workflow for the hazard vulnerability assessment of a community.

1. Introduction

Natural and man-made hazards can damage the built environment with a high yearly cost for societies [1–3]. Civil engineers seek to mitigate this cost by developing building codes and design provisions that adequately protect the built environment. For some hazards such as earthquake, methodologies have been developed to estimate yearly losses in order to support decision making [4–8]. However, no such methodology is available for fire, because so far fire protection has focused on life safety without consideration of the aftermath of a fire event. Communities can suffer great loss following a fire due to direct damage, content loss, and loss of functionality of the damaged buildings and infrastructures. At the scale of individual structures, statistics show that many businesses that experience a large fire eventually go into bankruptcy due to downtime and other indirect losses [3]. At the scale of a community, disasters such as wildfires [9], fire following earthquake [10], and explosion in industrial areas can disrupt an entire community.

Several challenges exist in the development of a fire loss estimation framework. The behavior of a building under fire involves fire development, heat transfer, and the corresponding structural response. Therefore, the measurement of building damage due to fire is complex and (unlike other structural loading such as earthquake) multi-physics by nature. This implies that the methodology developed for earthquakes cannot be straightforwardly adopted for the fire loss estimation, but requires adjustments to capture thermal damage states. Another challenge lies in the fact that the test data about structural/nonstructural components under fire are limited and rarely representative of the real behavior of the components under fire [11] due to the simplifications adopted in defining test boundary conditions and thermal exposures (e.g., standard fire tests do not contemplate the possible failure of members during the cooling phase [12]). As a result, little empirical data is available to construct fire fragility and consequences functions.

Previous studies have modified and applied the PEER PBEE framework to structures in fire [13–16]. However, there remain challenges in addressing the multi-physics characteristics of fire damage when defining engineering demand parameters, classifying damage states, and

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developing fragility and loss functions. The frameworks proposed in previous works mainly classified damage into a binary criterion of failure/no failure. While this may be suitable for some cases, a finer degree of granularity in defining fire damage is required in other cases, e.g. for RC structures. Although most of RC structures do not collapse under fire [17–20], the damage related to heat penetration or deformation may require significant repair efforts, hence influencing the loss assessment. Another ongoing challenge is related to the computational demand in applying a probabilistic method. Due to the computational cost, most previous works applied the framework to either isolated structural members or studied 2D frames. The study by Lange et al. [13] focused on floor systems. The study by Gernay et al. [15] focused on both beams and columns, analyzed as part of a 2D steel frame. The study by Memari and Mahmoud [14] focused on columns also in the context of a 2D steel frame. Yet, depending on the structural typology of the studied building, it might be necessary to capture the three-dimensional structural response to assess the damage to fire.

Therefore, the objective of this paper is to propose a framework for the probabilistic estimation of yearly economic losses due to fire for building structures. Building on previous research [21–26], this paper introduces new fire-specific engineering demand parameters that are defined from the results of nonlinear thermo-mechanical finite element analyses. The FEA quantifies the response of the entire structural system (3D building) to capture the interactions between frame-members and floors under fire exposure. The classification of damage states addresses gradual levels of repair efforts of components in the building in addition to binary failure/no failure criteria. From there, fragility and consequence functions are used within a step-by-step procedure that results in an economic loss evaluation, as detailed hereafter. In sum, the framework quantifies the direct cost of fires to buildings while accounting for the uncertainties in the fire occurrence and growth as well as in the building response. This effort can pave the way to performance-based design optimization [27–29] and cost-benefit analysis of fire safety measures [30,31]. This paper focuses on reinforced concrete structures, but the method can be adopted for other types of structures, as well as being integrated into regional hazard workflows (such as developed by NHERI SimCenter [32]) to analyze the impact of regional fire hazards on the built environment in support of fire resilience decision making.

2. Overview of methodology

The general PEER's probabilistic assessment methodology for buildings under earthquake hazard is shown in Fig. 1a [33]. The methodology divides the seismic performance assessment into four steps: (1) hazard analysis, (2) structural response analysis, (3) damage analysis, and (4) loss assessment. The hazard analysis quantifies the hazard in terms of an intensity measure, IM. The structural analysis measures the response in terms of engineering demand parameters (EDP), conditioned on the intensity measure IM. The damage analysis assesses the damage through component fragility functions that yield the probability to reach or exceed a particular damage state, DS, as a function of the calculated EDP. Each damage state is described by certain damage measures, DM. The different damage states are indicative of the corresponding efforts needed to repair a structural/ nonstructural component. Finally, the loss analysis is the probabilistic estimation of system performance measures, referred to as decision variables (*DV*), conditioned on damage. The loss analysis aims to include the overall loss including the monetary cost of repairs or replacement, possible casualties, and downtime, etc.

Unlike with an earthquake, the damage caused by a fire to a building depends on the fire location of ignition and development within the building. Therefore, the assessment needs to consider different possible fire scenarios within the building corresponding to different locations of ignition of the fire. Then once the fire location is fixed, a parallel can be

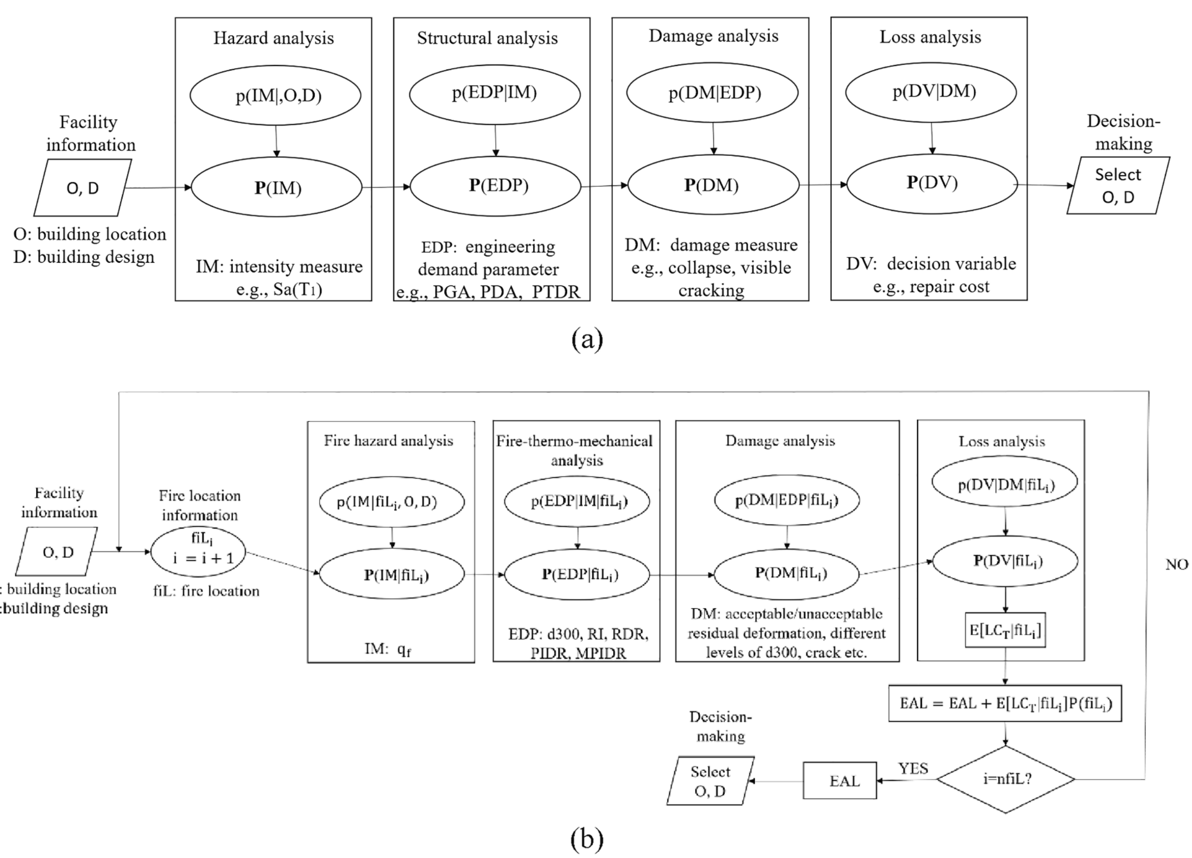


Fig. 1. Probabilistic loss assessment for buildings (a) under earthquake [33], and (b) under fire.

drawn with the PEER methodology, and the framework for fire hazard is also described in terms of the four analysis steps. This sequence of four analyses is repeated for different fire locations as indicated by the loop in Fig. 1b. Since this paper focused on the expected annual fire loss of a building, the expected annual fire loss is emphasized in Fig. 1b and the discussion in the following also focused on the expected annual fire loss. However, it is noteworthy that more information besides the expected annual loss could be determined from the proposed framework for decision making.

Therefore, the methodology proposed here for fire hazard calculates the expected annual loss due to fire as a sum of weighted estimated losses for different fire locations. More precisely, the expected annual loss for a building,EAL, is the summation of the products of the expected fire loss in each location and its corresponding probability of fire occurrence, as shown in Eq. (1). In the latter, $E[LC_T|fiL_i]$ is the expected loss conditioned on a fire occurrence in the location i, while $P(fiL_i)$ is the probability that the fire occurs in this location i. nfiL is the total number of possible fire locations in the building with a certain fire region, mainly determined by the fire compartmentation and function layout of a building. It is noteworthy that the probability of fire occurrence in one specific location depends on a lot of factors, e.g., its occupancy type; a discussion on the effect of different ignition likelihoods, and their combination in a weighting method, on the fragilities can be found in [24]. For instance, $P(fiL_i)$ is possibly higher for a utility room compared to that of a bedroom. Equivalently, the expected annual loss is the product of the expected loss of the building conditional on the fire occurrence in the building ($E[LC_T|fi]$) by the annual probability of fire occurrence in the building (P(fi)).

$$EAL = \sum_{i=1}^{nfiL}[E[LC_T|fiL_i]P(fiL_i)] = E[LC_T|fi]P(fi) \tag{1}$$

The damage analysis takes into account the possibility of complete collapse of the building under fire. In case of collapse, the loss is taken as the cost of demolition and reconstruction and the replacement of content. However, previous studies and observation from real events suggest that fires in concrete structures rarely result in complete structural failure and that many of the damaged structures can be successfully reinstated [17–20]. In case of no-collapse, the damage is evaluated based on defined damage states of components or assemblies (including structural components, nonstructural components, and content). This evaluation relies on EDP and DS specific to fire hazard that is introduced in this work to capture thermal and mechanical damage. It is still possible for a damage state to exceed a threshold of safety and/or affordable ability to rehabilitate (e.g. excessive overall residual deformations), in which case the building will be demolished and reconstructed.

The methodology is presented in detail next. Section 3 discusses the fire hazard analysis, focusing on the probabilities of fire occurrence and on the fire load as intensity measure (IM). Section 4 focuses on the selection of the Engineering Demand Parameters (EDP), definition of Damage States (DS), and fragility functions. The response analysis and damage analysis are treated together in Section 5, for structural and nonstructural components. This results in the assessment of conditional probabilities for structural/non-structural damage states as a function of the fire load, given the fire occurrence of fire in a specific location. Section 6 deals with the loss analysis and results in an expected annual loss for the whole building due to fire, which constitutes a decision variable (DV) for the fire safety strategy of the building, for instance for use within a cost-benefit analysis.

3. Fire hazard analysis

The fire hazard analysis includes two parts. First, given a fire starts in a building, the hazard analysis needs to assess the relative probabilities of occurrence in each compartment, $P(fiL_i)$ in Eq. (1). Then, for any given fire location, the hazard analysis deals with the physical variable chosen to represent the intensity measure (IM) of the hazard, which depends on the building design and occupancy.

3.1. Probabilities of fire occurrence

Herein, the fire probabilities refer to the occurrence of severe fires that grow uncontrolled despite the possible presence of sprinklers or other active fire protection measures. This means that only the fires that have grown to the point where they achieve flashover and challenge the structure are counted. The framework is compatible with any method for calculating the annual probability of occurrence of a severe fire; in particular, risk-based and statistical methods accounting for the specific community fire resources can be embedded in the framework [34,35]. Here, the method used to derive the provisions in the Eurocode [36] is adopted; this method has been used in other studies for probabilistic fire safety assessments [13,15]. The annual probability $P(fiL_i)$ of occurrence of a severe fire in a compartment i which has a floor area A is calculated using Eq. (2). This equation is the one used for the development of the design values for the fire load densities in Eurocode [36]. The term $P_{1,EN}$ is the probability of having a fire to start and grow to a severe fire, per m^2 of floor and per year, including the effect of occupants and standard public fire brigade. Additional reduction factors are applied to this annual frequency to account for active fire protection measures that limit the probability for a fire to grow to severe fire, as detailed in [36]. These factors can vary between fire compartments within a building.

$$P(fiL_i) = (P_{1,EN}P_{2,EN}P_{3,EN}P_{4,EN})A \tag{2}$$

Using Eq. (2), one can evaluate the annual probability of severe fire occurrence in a single compartment of a building, or (through simple summation of the probabilities in individual compartments, assuming independence) in a particular story, or in the whole building. For example, the annual frequency is calculated for a story of an office building that comprises 25 compartments with identical area of 49 m^2. This building is exactly the same building described in Section 7. For that building, $P_{1,EN} = 3.0 \times 10^{-7}/m^2/year$. It is assumed that the time between the alarm and the intervention of a professional fire brigade is between 10 and 20 min ($P_{2,EN} = 0.10$), and that all fire compartments are equipped with automatic fire detection by smoke ($P_{3,EN} = 0.06$), but no sprinkler protection system ($P_{4,EN} = 1.0$). The resulting probability of a severe fire in a compartment is 0.92×10^{-7}/year, while that of the whole story is 2.3×10^{-6}/year. Now, if this building has five identical stories, the annual frequency of a structurally significant fire for the overall building would be equal to 1.15×10^{-5}/year. Conversely, the conditional probability to have the fire in any of the compartment, assuming a fire starts in the building, can be evaluated using a similar approach.

3.2. Probability distribution of fire load

A physical variable must be chosen to represent the intensity measure of the hazard to which the vulnerability is assessed. In earthquake engineering, the hazard is usually characterized in terms of acceleration g. The development of fragility functions is thus conducted considering different levels of acceleration, to cover the range of possible earthquake scenarios.

For structures in fire, the parameter that is best adapted to characterize the hazard could be the fire load (q_f) in a compartment (in MJ/m^2 of floor area). Indeed, for a given fire location, the fire load is one of the main parameters affecting the intensity of a fire [37]. It may vary in a significant range and it has a straightforward definition that is easily understood by the different stakeholders involved in fire safety, making it a good candidate as IM when developing fragility functions for structures in fire [15]. That being said, it is clear that other parameters influence the severity of a fire, but the power of the PEER framework lies in the fact that it uses a single scalar from one analysis step to the next one, and the fire load appears as a good choice for the fire hazard analysis.

The Gumbel type I distribution is frequently adopted for the probability distribution of fire loads [36], for example in Eurocode 1 part 1–2 [38] which uses Eq. (3). The parameters depend on the occupancy; for an office, $\sigma = 126$ MJ/m^2 and $\mu = 420$ MJm^2.

$$p(q_f) = \frac{1}{\sigma} e^{-(z+e^{-z})} \quad \text{with } z = \frac{(x-\mu)}{\sigma} \tag{3}$$

4. Selection of engineering demand parameters, damage states and fragility functions

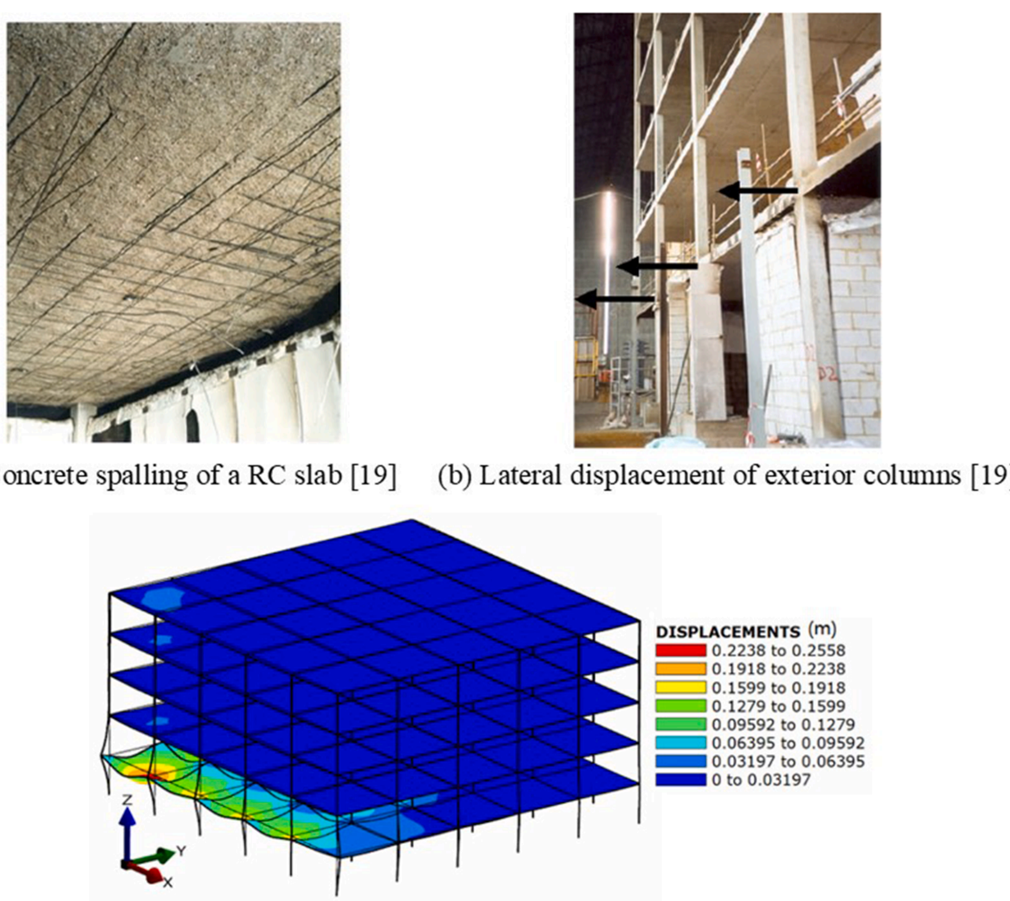
Engineering demand parameters (EDP) can be correlated to structural, non-structural, and content damage within a structure. The damage is quantified through the definition of discrete damage states (DS). Fragility functions relate the expected probability to reach or exceed each damage state to the intensity measure of the fire. The following sections discuss the EDP, DS, and fragility function adopted for the damage analysis of components under fire.

4.1. Structural components directly exposed to fire

Fire exposure may lead to damage in different forms including concrete spalling and cracking (Fig. 2a), rebar buckling, or permanent deformations caused by restrained thermal expansion effects (Fig. 2b). Fig. 2b shows the residual lateral displacements of exterior columns from a full-scale test on a seven-story RC building in Cardington [19]. Fig. 2c shows the residual deformation of a RC frame building after fire exposure as predicted by the nonlinear finite element software SAFIR [39].

The fire-thermo-mechanical analysis needs to quantify the response in terms of EDP that can be correlated to the types of damage mentioned above. It is proposed to introduce two EDP's for reinforced concrete structures in fire. The first one addresses material damage of the sections due to temperature penetration, while the second one addresses structural damage of the assemblies due to thermal deformations.

The first EDP relates to the damage of the material at the section level, due either to micro- or macro-cracking, spalling, or irreversible loss of properties following heating–cooling. Existing research on the degradation of concrete under fire has shown that a considerable reduction in material properties occurs during a fire and, after cooling, this may amount to 30% for compression strength and 40% for elastic modulus after exposure to a temperature of 300 ℃ [41–43]. The Eurocode specifies that the elevated temperature strength of siliceous concrete has dropped by 15% at 300 °C and, after cooling down to ambient temperature, the drop amounts to 23.5% (i.e. a retention factor of 0.9 × 0.85 according to EN1994-1–2, Annex C). The color of concrete changes from grey to pink when the concrete temperature exceeds 300℃ [41]. In engineering practice, the measurement of the penetration depth of pink coloring is often used to estimate the fire severity and inform the repairs process. Concrete which has experienced a peak temperature greater than 300℃ (as evaluated from its color after the event) will be removed and replaced; the reinforcement steel bar in that region will be strengthened due to possible permanent degradation of mechanical properties (that may occur at temperatures exceeding about 600 °C [44]) and/or permanent deformation due to buckling [41,45]. This



(a) Concrete spalling of a RC slab [19] (b) Lateral displacement of exterior columns [19]

(c) Residual deformation of a RC building after fire exposure [40]

Fig. 2. Types of damage in RC structures after fire exposure.

practice is based on the assumption that the threshold of 300 °C is relevant for separating the member section into a part that is likely to have its residual area and properties permanently affected from cracking and spalling, and a part that can be assumed as unaffected by the fire. The same reasoning based on a 300 °C temperature threshold is adopted here. Hence, we select the penetration depth of the 300 °C isotherm in the concrete section, noted d300, as EDP for the damage evaluation of RC components after fire exposure. The definition of d300 from the results of heat transfer analysis is shown in Fig. 3, where the dash line separates the section into two parts based on the experienced peak temperature of 300℃. The advantages of this EDP are that it is both reasonably correlated to experimental evidence on the residual concrete behavior and practically convenient as the 300 °C temperature is easy to assess on site based on coloring (and corresponds to current practice).

The damage states associated to the d300 EDP correspond to different depth of penetrations. They are classified according to different levels of repairs needed for the section as described in [41,46,47] and summarized in Table 1. It is noteworthy that other characteristics of the fire-damaged concrete section have been proposed in the literature to define classes of damage states [41], including on-site observations on cracking or spalling, but these characteristics are not easy to map to numerical results from a finite element software, whereas a temperature penetration depth is directly accessible from numerical results.

While the first EDP (d300) addresses damage of the sections, a second EDP is needed to address the residual deformations in the structure. Indeed, section damage due to heat penetration and member residual deformations are not necessarily directly correlated. For instance, a column exposed to a severe fire can experience important cracking and loss of properties, but exhibit limited residual deformations due to symmetrical thermal exposure and restraint conditions and limited loading. Inversely, a column that is only tangentially exposed to the fire and therefore suffers limited sectional damage, can experience large permanent deformations generated by thermally induced forces of expansion and contraction coming from surrounding members [40]. These examples show the need to consider separately the two EDPs. It is true that the two EDPs are correlated to some extent, but more research is needed to capture this degree of correlation. In this study, the two EDPs are assumed to be uncorrelated.

Regarding residual deformation after fire, columns mainly experience lateral deformations while beams and slab mainly experience vertical deflections. The residual inclination (RI) is selected as the deformation-related EDP for columns. It is defined by the residual deformation (Δ_c) at the column top divided by the height of the column (h), as shown in Fig. 4a. The residual vertical deflection ratio (RDR) is selected as the deformation-related EDP for beams and slabs. It is defined by the residual vertical deformation at the center of the slab (Δ_s) divided by the square root of the product of spans in x and y directions ($l = \sqrt{l_1 l_2}$), shown in Fig. 4b. The RDR EDP is selected for the whole floor system because slabs and beams are often cast monolithically and their repair actions are highly coupled.

Practice shows that it is difficult to straighten or realign a RC structural component. The CIB report [41] noted that no attempt should be made to force a RC structural component back to its original position. If the residual drift is small, no corrective action on geometry is necessary for structural stability, and only the sectional repairs are conducted

Table 1
Fire damage states for the EDP of the penetration depth of the 300 °C isotherm in the sections of the RC members [41,46,47].

Damage states	EDP: Penetration depth of the 300 °C isotherm (d300)	Repair actions
DS0	Temperature did not exceed 300 °C in concrete	• No repair is required; redecoration if required
DS1	0 < d300 < c/10	• Chip, clean and patch the damaged region
DS2	c/10 ≤ d300 < c	• Remove damaged concrete in repair area to fully expose rebar • Clean by high water pressure • Blow off dust/debris with oil-free dry compressed air • Place repair material by wet shotcrete process to full depth of repair • Carefully trim excess material and steel trowel finish without overworking • Cure with sprayed membrane curing compound
DS3	c ≤ d300 < d/4 (d is the side dimension of the cross section)	• Remove concrete in repair area to fully expose reinforcing bars and to the depth of 300 °C • Final clean by high water pressure • Blow off dust/debris with oil-free dry compressed air • Placement of supplemental rebar • Place repair material by wet shotcrete process to full depth of repair • Carefully trim excess material and steel trowel finish without overworking • Cure with sprayed membrane curing compound
DS4	d/4 ≤ d300 ≤ d/2	• Demolish and reconstruct

* c = concrete cover to the edge of the rebar.

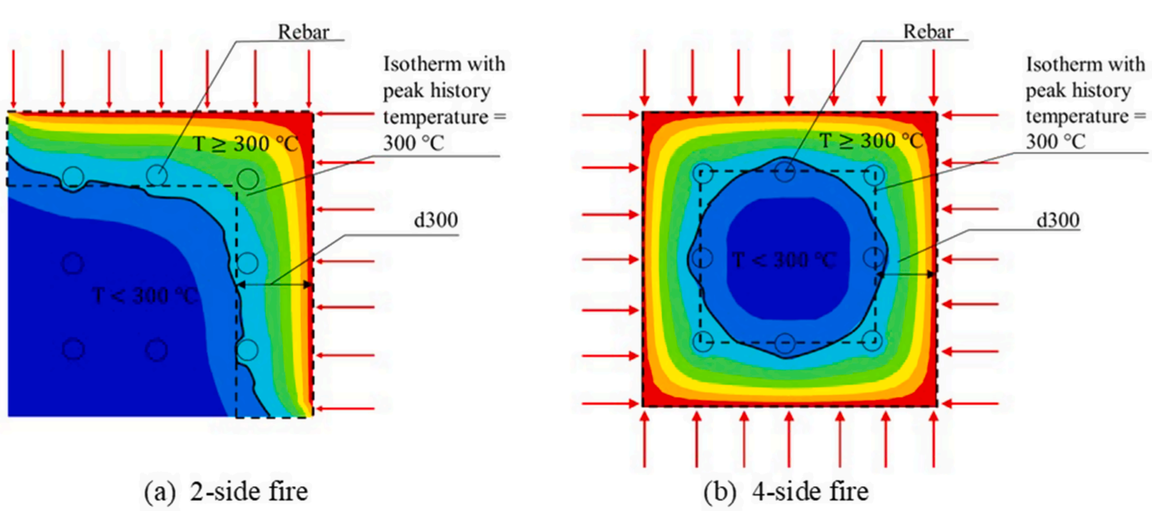


Fig. 3. Definition of heat penetration depth d300 from results of thermal analysis for evaluation of damage states in the RC sections.

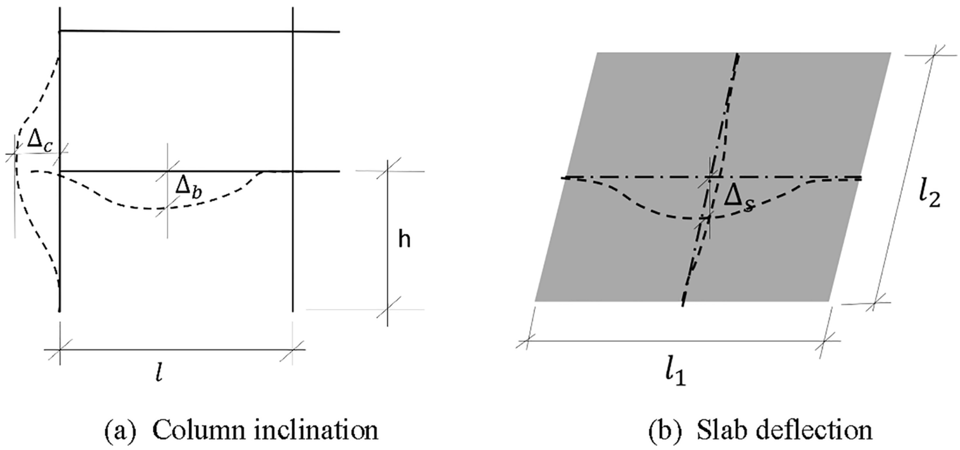


Fig. 4. Residual deformation in structural components.

as discussed for the d300 EDP. However, if the residual drift exceeds a threshold that affects functionality, construction tolerance and safety, the structural component has to be demolished and reconstructed. The damage states associated to the EDP of residual deformation correspond thus to a threshold in the values of RI and RDR.

For columns, the research done by McCormick et al. for earthquake [48] has proposed a threshold of 0.005 rad, considering functionality, construction tolerance, and safety. Here, "safety" means that the residual drift of a structural component will not impede the escape of occupants, e.g., the residual drift of the surrounding structural members around a door system should not lead to the damage of the door system. They noted that no structural realignment is required for a column with residual inclination up to 0.005, while repairs are required beyond this value. Although this was proposed for buildings after earthquake, similar concerns about the structural safety and building functionality exist in buildings after fire hazards. Therefore, the residual inclination of 0.005 is selected as the threshold in RI separating the two damage states for the RC columns after fire (Table 2).

For floors (slabs and beams), the thresholds can be based partly on the limiting values in design codes for vertical deflections. For instance, the CSA A23.3–14 [49] includes two limiting values, where L/360 limits the immediate deflection of a floor under the live load, while L/240 limits the long-term deflection of a floor under the dead load and the sustained live load. Those limits are proposed to limit the deflection of the floor system under service states. Here, four damage states are adopted based on the value of residual vertical deflections. The definition of the damage states and their corresponding repair actions are summarized in Table 3.

Relationships are proposed between the damage states and the EDP in Tables 1–3. Based on these, probabilities associated with different damage states for a structural component after fire can be calculated from the EDP which are direct outputs of the probabilistic fire-thermo-mechanical analysis. This is convenient because it avoids the difficulty associated with obtaining empirical fragility functions given the limited amount of test data for post-fire damage of structures. Meanwhile, the adopted thresholds are imperfect representation of a complex state of damage. In practice, engineers responsible for post-fire inspection would have their own judgement on the damage states of a structural component after fire exposure. To incorporate the uncertainty and variability inherent to the process of post-fire evaluation, the thresholds for the damage states are adopted as probabilistic variables. Data is lacking to quantify these probabilistic variables and further research is required. Here, it is assumed that those variables follow a lognormal distribution with mean values equal to the values from Tables 1–3 and coefficients of variation equal to 0.1. The lognormal distribution of the lower threshold of each damage state (as shown in Tables 1–3), as a probabilistic function of the EPD, is the fragility function that represents the probability of a component to reach or exceed the damage state, as shown in Fig. 5. Follow-up research is required to validate or modify the assumption here. Since the lower threshold value of DS1 for heat penetration is 0, the fragility curve for DS1 is the lognormal distribution with a mean value very close to zero, as shown in Fig. 5b.

Table 2
Damage states based on residual inclination of RC columns [41,48].

Damage state	EDP: Residual inclination (RI)	Repair method
DS0	$\Delta_c/h < 0.5\%$	• No repair is required
DS1	$\Delta_c/h \geq 0.5\%$	• The structural component should be demolished and reconstructed

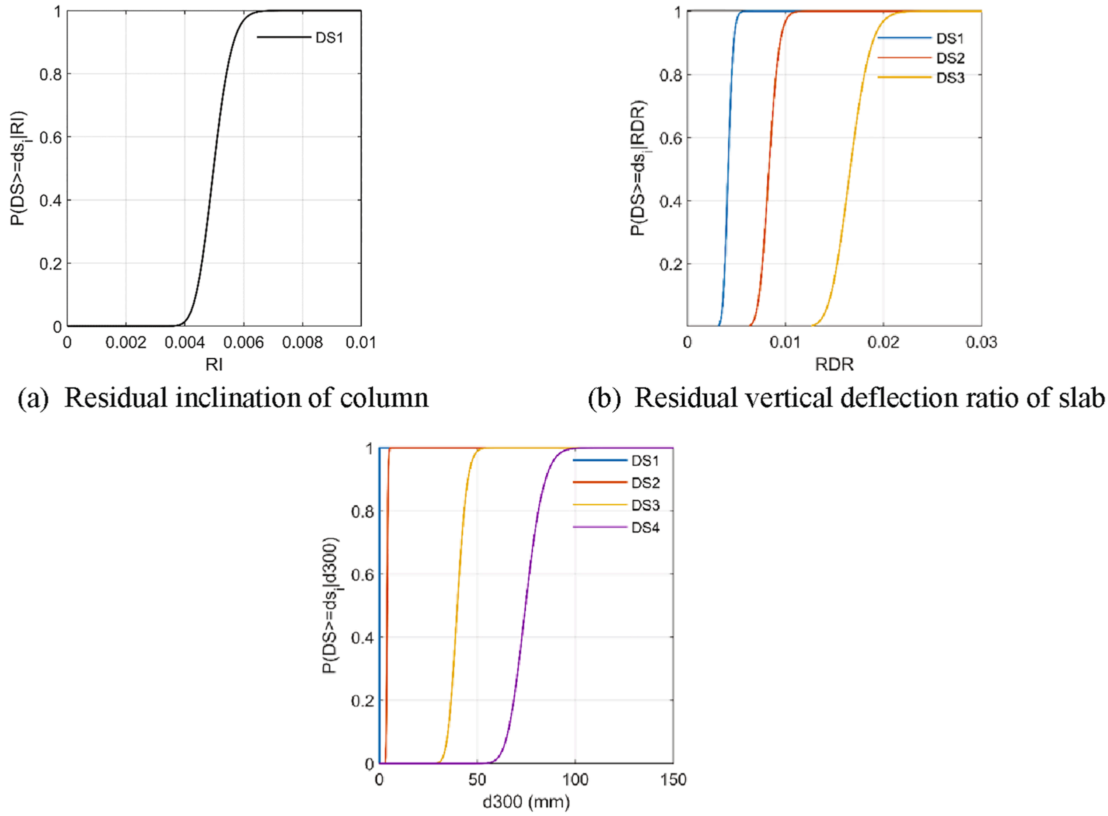
Table 3
Damage states based on residual vertical deflection of RC slabs [46,49,50].

Damage state	EDP: Residual vertical deflection (RDR)	Repair method
DS0	$\Delta_s/l < 1/240$	• No repair is required
DS1	$1/240 \leq \Delta_s/l < 1/120$	• Clean the surface by high-pressure water • Blow off dust/debris with oil-free dry compressed air • Place repair material • Cure with sprayed membrane curing compound
DS2	$1/120 \leq \Delta_s/l < 1/60$	• The slab should be demolished and reconstructed, while beams can be reused
DS3	$1/60 \leq \Delta_s/l$	• The whole floor system, including slabs and beams, should be demolished and reconstructed

4.2. Structural components not exposed to fire

For structural members not directly exposed to fire, the EDP of heat penetration (d300) is irrelevant. However, these members may experience damage associated with the EDP of deformation. Indeed, structural members not directly exposed to fire may be subject to forces and displacements exerted by the surrounding heated members which undergo thermal expansion.

Therefore, EDPs similar to the ones discussed in Tables 2 and 3 are considered. However, when assessing the costs associated with the repair actions for these damage states, a different estimate is made for the components not directly exposed to fire, compared with those exposed to fire. This is because, for the components exposed to fire, the repair costs for the deformation DS exclude the repair costs for sectional



(a) Residual inclination of column (b) Residual vertical deflection ratio of slab

(c) Heat penetration of sections (for column with d=300mm)

Fig. 5. Probabilistic relationships between the EDP and the DS for structural components directly exposed to fire.

damage, which are already covered through the damage state of heat penetration in Table 1 and should not be double counted. In contrast, for components not directly exposed to fire, the only repairs costs that are estimated are those associated with deformation. Since it is reasonable to assume that significant deformations are likely to be associated with material damage (e.g. cracking/damage of the section), some adjustments are required.

For columns not exposed to fire, the adopted approach is taken from previous research on columns under earthquakes. Researchers have proposed different EDP for columns under lateral loads, e.g. the deformation damage index (DDI) and the peak interstory drift ratio (PIDR). Associated fragility functions and loss functions can also be found in the literature [4,5]. This work adopts the PIDR fragility and loss functions for the columns which were outside the fire area, for which no direct damage due to heating has to be considered.

For the floor members, the residual vertical deflection (RDR) is selected to represent the EDP on the floor systems. The definitions of damage states in Table 3 are used to evaluate the damage. Yet, the cost associated with each of these damage states listed in Table 3 is increased for floor members in non-fire region as explained to cover surface reparation, i.e. the actions required to repair the cracking of the surface, which was included through the EDP d300 for the floor members exposed to fire.

4.3. Nonstructural components and content directly exposed to fire

It is assumed that all the non-structural components and content in the compartment directly exposed to fire are irreversibly damaged. Indeed, the combination of elevated temperatures, smoke and combustion products, and possible use of firefighting measures, can be assumed to result in the complete loss and need for replacement of all nonstructural components and content in the compartment. Therefore, loss estimates are based on the assumption that the entirety of the non-structural components and content are lost in the area where the fire developed.

4.4. Nonstructural components and content not exposed to fire

The content outside the area directly exposed to fire is assumed to be unaffected by the fire. This assumption has limitations as, in reality, the fire event may result in some loss of content outside the directly affected area, for instance, due to the spread of smoke. However, this is very hard to quantify and, in this study, these effects are neglected.

For the nonstructural components outside the fire area, some degree of damage needs to be contemplated due to the fire-induced deformations in the structure. According to studies in the seismic field [4,5], components such as exterior walls, interior partition walls, and windows are sensitive to the peak transient drift ratio. These components can be assumed to be similarly sensitive to the peak interstory drift ratio during a fire event. However, the lateral displacements at each point in a given floor are not equal under fire, which is different from what happens in a building under earthquake. The mean peak interstory drift ratio (mPIDR) of one-bay one-story frame in one direction is recommended as the EDP for exterior walls, interior partition walls and windows which are part of that one-bay one-story frame. Although out-of-plane damage may also occur, only the damage due to in-plane drift is considered in this paper. The definition of damage states and their corresponding repair methods and fragility functions are directly taken from those proposed for the damage analysis of nonstructural components under earthquake [4,5,51]. Meanwhile, for components such as ceiling systems, automatic sprinklers, plumbing, and elevators, seismic studies indicate that damage is primarily sensitive to the peak acceleration of the diaphragms. In fire, deformations develop at a slow rate; hence it is assumed here that the latter components are not damaged if they are not directly located in the zone where the fire occurred.

5. Fire-Thermo-Mechanical analysis and damage analysis

5.1. Overview

The fire-thermo-mechanical analysis is performed to evaluate the probabilities associated with the engineering demand parameters (EDP) in the structure. These probabilities are conditional on the fire load (q_f) and the location of the fire (fiL_i). The damage analysis then relates the EDP to the damage states as discussed in Section 4. Fig. 6 shows an overview of the response and damage analysis of a building, conditional on the fire occurrence in the compartment fiL_i with fire load q_f. For each fire location, probabilistic fire-thermo-mechanical analyses of the structure are conducted for different values of the fire load q_f, considering uncertainties stemming from the fire, heat transfer, and mechanical models. By varying q_f, the fragility functions for the structural and nonstructural components can be determined. Those functions are inputs for the loss analysis.

For some combination of the parameters, the building may collapse (C). The analysis quantifies the probability of collapse, noted $P\left(C|q_f, fiL_i\right)$. In concrete structures, $P\left(C|q_f, fiL_i\right)$ is low. The loss estimation for the collapse cases is straightforward as the building has to be reconstructed entirely. For cases of non-collapse, the loss estimation requires the evaluation of the EDP and the damage states as discussed in Section 4. The results from the non-collapse (NC) cases are used to determine the probability density distributions of the EDP, based on which the probability of a structural/nonstructural component at a specific damage state are calculated, conditional on the non-collapse of the building under the occurrence of fire in a specific location fiL_i with fire load q_f.

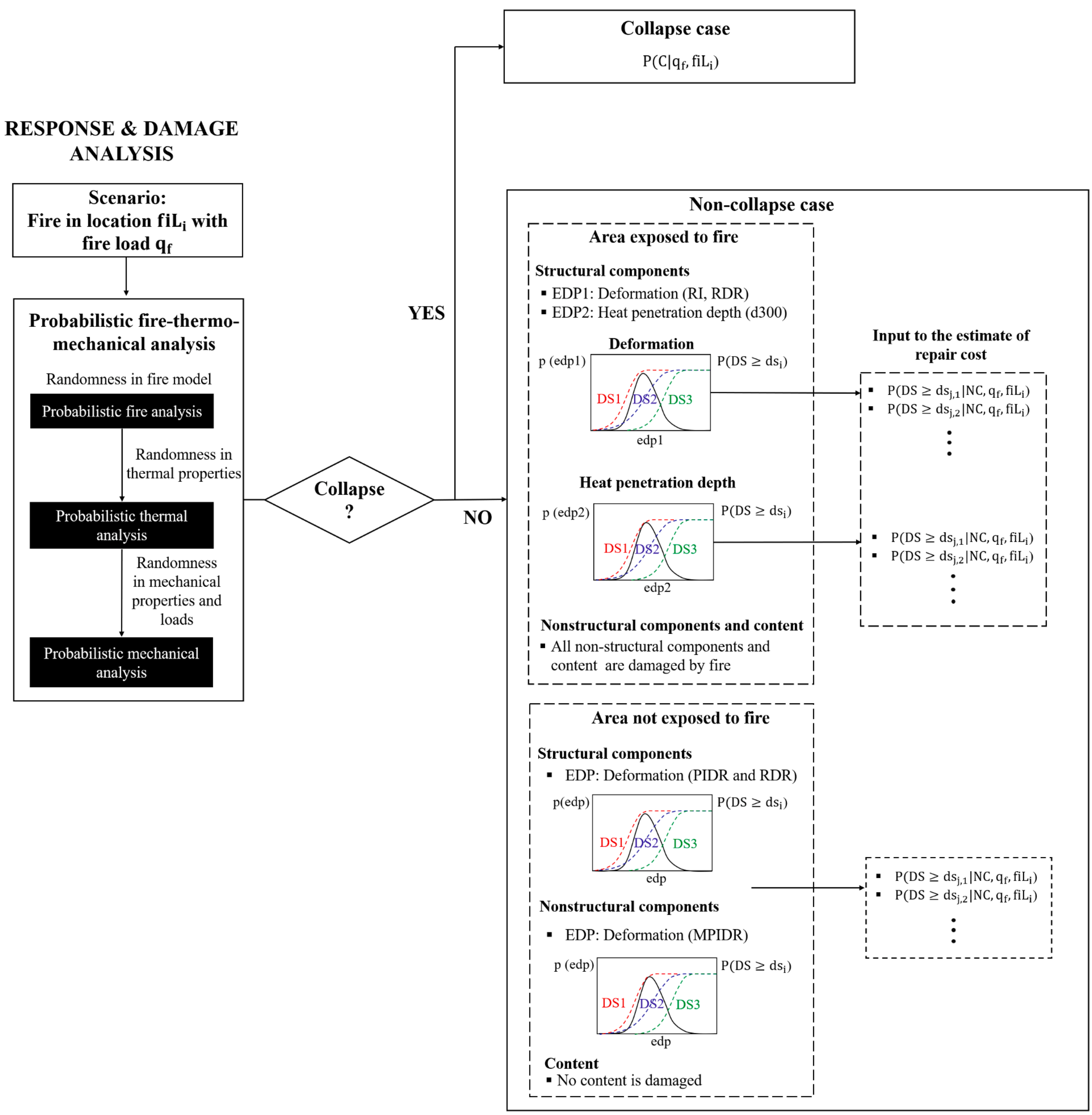


Fig. 6. Overview of the response analysis and damage analysis.

5.2. Probabilistic response evaluation from fire-thermo-mechanical analysis

The response analysis sequentially includes a fire analysis, heat transfer analysis, and mechanical analysis. Numerical analysis by nonlinear finite element method can be used, see e.g. [40]. The analysis takes into account the parameter uncertainties related to the fire model, the heat-transfer model, and the mechanical model. Considered uncertain parameters may include, for instance, the opening factors, thermal conductivity of materials, thickness of concrete cover, mechanical loads, mechanical properties of materials at elevated temperature, etc. [21,26,52]. The fire load uncertainty is accounted for separately, through the use of this parameter as IM.

In this study, the adopted fire model is the parametric fire model from Eurocode [38]. This model has been derived for the situation of fully developed fires in a compartment with three main parameters: 1) fire load density (q_f) which accounts for the fire load in a compartment; 2) opening factor (O) which accounts for the ventilation conditions in a compartment; and 3) thermal inertia of the enclosure boundaries (b) which accounts for the nature of the walls, ceilings and floor of a compartment. Yet, any fire model with cooling phase can be incorporated into the framework developed in this paper.

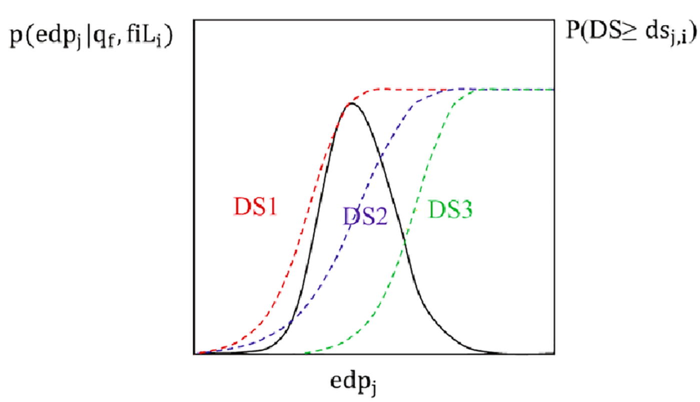
The probabilistic fire-thermo-mechanical analysis can be run using Monte Carlo simulations. Although Monte Carlo simulation is an unbiased method, it requires a large number of model evaluations and is therefore computationally expensive when large nonlinear calculation models are required, as is common in fire safety engineering. The computational effort can be reduced through the use of efficient techniques, such as surrogate models, stratified sampling, maximum entropy methods, etc. Further, the number of studied fire locations (scenarios in Fig. 6) can be reduced if the layout and compartmentation of a building presents a symmetry, as will be discussed in the case study of Section 7.

5.3. Damage analysis

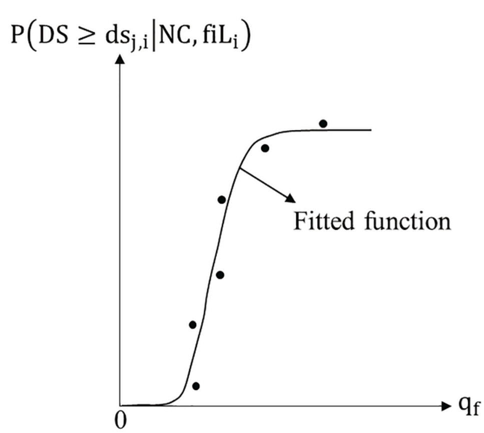
For cases where no global collapse occurs, the analyses of Section 5.3 determine the probability of reaching or exceeding the damage states in each component (as a function of the intensity measure q_f). The damage analysis of components is then based on both these probability density distributions of the EDP and the fragility functions. The PDF of an EDP is the output of the fire-thermo-mechanical analyses. The fragility functions of a given component are the probability that the component reaches or exceeds a specific damage state, given the EDP (Fig. 5). The convolution of the fragility function and the PDF of EDP leads to the conditional probability that the component j exceeds a specified damage state i, as given by Eq. (4) and illustrated in Fig. 7a. The output of Eq. (4) is a scalar.

$$P(DS \geq ds_{j,i}|NC, q_f, \mathrm{fiL}_i) = \int_0^{\infty} \mathrm{P}(\mathrm{DS} \geq ds_{\mathrm{j,i}}|\mathrm{edp_j})\,\mathrm{p}\left(\mathrm{edp_j}|NC, q_f, \mathrm{fiL_i}\right)\mathrm{dedp_j} \tag{4}$$

The procedure behind Eq. (4) must be repeated for different levels of intensity measure. By varying the fire load and repeating the procedure,



(a) PDF distribution of EDP and the corresponding fragility functions



(b) Probability of damage as a function of q_f

Fig. 7. Probability density distribution of EDP and damage analysis.

different points relating the fire load and the probabilities that the component reaches or exceeds the different damage states are obtained. A function can then be fitted on these points as shown in Fig. 7b. The probability of the component j at one specific damage state i when it is subjected to a specified fire load is estimated as the difference between two successive damages states, as given by Eq. (5). In Eq. (5), i begins with 0. The damage state $ds_{j,0}$ is the state that a component is almost undamaged, and no repair efforts are required, as shown in the first state in Tables 1–3. The corresponding probability for this state is the probability that a component is almost undamaged under fire, which is one minus the probability that a component reaches or exceeds $ds_{j,1}$.

$$P\left(ds_{j,i}\middle|NC,q_f,\mathrm{fiL}_i\right)=1-P\left(DS\geq ds_{j,1}\middle|NC,q_f,\mathrm{fiL}_i\right)\qquad \text{for } i=0$$

$$P\left(ds_{j,i}\middle|NC,q_f,\mathrm{fiL}_i\right)=P\left(DS\geq ds_{j,i}\middle|NC,q_f,\mathrm{fiL}_i\right)-P\left(DS\geq ds_{j,i+1}\middle|NC,q_f,\mathrm{fiL}_i\right)\qquad \text{for } 1\leq i<m \tag{5}$$

$$P\left(ds_{j,i}\middle|NC,q_f,\mathrm{fiL}_i\right)=P\left(DS\geq ds_{j,i}\middle|NC,q_f,\mathrm{fiL}_i\right)\qquad \text{for } i=m$$

6. Loss analysis

The loss analysis is conducted next for the building after a fire. In Section 6.1, loss functions corresponding to the repair methods of structural components or non-structural components at different damage states are presented. Based on these loss functions, Section 6.2 presents the method to calculate the expected annual loss of the building under fire hazard, accounting for the probability of building collapse and the probabilities of components at different damage states. In this work, only direct losses are considered; hence "loss" and "repair cost" are used as synonyms. In future work, indirect losses can be incorporated into the framework.

6.1. Loss functions

The loss cost (LC) for a component j having reached a damage state i is noted $LC_j|ds_{j,i}$. In literature [4,5], loss functions are usually assumed to follow a cumulative lognormal distribution to estimate the probability of a certain level of loss in a component when a certain damage state has been observed, as given by Eq. (6).

$$P\left(LC_j|ds_{j,i}\right)=\Phi\left[\frac{\mathrm{Ln}(lc)-\mu_{LnLC_j|ds_{j,i}}}{\sigma_{LnLC_j|ds_{j,i}}}\right] \tag{6}$$

where Φ is the standard cumulative normal distribution, while $\mu_{LnLC_j|ds_{j,i}}$ and $\sigma_{LnLC_j|ds_{j,i}}$ are the logarithmic mean and logarithmic standard deviation of the loss in component j under the damage state i, respectively. The probabilistic parameters can be calculated from the relationships between the mean and the standard deviation of $(LnLC_j|ds_{j,i})$ and $(LC_j|ds_{j,i})$.

The expected mean repair cost, $E\left[LC_j|ds_{j,i}\right]$, has to be evaluated for each damage state. Data about repairs costs are given in the RSMeans data, a construction cost database [46]. The repairs procedure corresponding to each damage state is identified first, based on Tables 1–3. The cost associated to these repairs is then evaluated from the RSMeans database. The coefficients of variation of the structural cost of rehabilitation, COV= $\sigma\left[LC_j|ds_{j,i}\right]/E\left[LC_j|ds_{j,i}\right]$. The COV ranges from 0.6 to 1.1 for typical construction cost [53]; according to FEMA 276, the COV of the cost of rehabilitation for typical structural components ranges from 0.6 to 0.8 [54].

6.2. Loss estimation

6.2.1. Repair cost for the non-collapse scenarios

The repair cost is first evaluated for the cases where the building has survived the fire (non-collapse in Fig. 6). The discussion treats successively the cost due to the damage in the area directly exposed to fire and then in the area not exposed to fire. For the area directly exposed to fire, the estimate includes the loss due to the damage to the columns, the floor system (slab and beams), the nonstructural components and the content.

For a column cj, the expected loss, $E\left[LC_{cj}|NC,q_f,\mathrm{fiL}_i\right]$, is computed as a function of the cost of repairing the component when it is in different damage states and the probability of being in each damage state. The two EDP of residual inclination (RI) and heat penetration into the section (d300) are considered simultaneously. These two EDPs are assumed independent in terms of the repair costs they generate, so that the expected loss can be obtained as the sum of the products of the expected value of the loss in each combination of damage state by the probabilities of this combination of damage states. Yet in the situation where one of the EDP results in the need for replacing the component, no cost associated with the other EDP is added (since the full cost of replacement is already considered).

For the floor system, the beams and the slab are evaluated separately with respect to the heat penetration depth while they are evaluated as an entire floor system with respect to the residual deformation. If the vertical deflection is so large that the slab or the floor system needs to be replaced, the repair cost of the replaced structural members associated with the EDP of d300 is set to zero. If the damage of the heat penetration to the slab is too severe (slab at the damage state $ds2_{s,4}$) while the deflection of the floor system has not reached the damage state $ds1_{f,3}$, the repair cost of the slab will be the expected replacement cost of the slab, while the corresponding repair cost due to the deflection will be zero.

The expected total loss due to the fire damage in the zone directly exposed to fire is obtained by summing the repair cost in columns, in the entire floor system (including beams and slab), plus the expected replacement cost of all the nonstructural components and the content in the fire-exposed region, see Eq. (7) where the condition $|NC,q_f,\mathrm{fiL}_i$ is omitted for brevity.

$$E\left[LC_{fire,T}\right]=\sum_{cj=1}^{nc}E\left[LC_{cj}\right]+\sum_{fj=1}^{nf}E\left[LC_{f,j}\right]+\sum_{nsj=1}^{nns}a_{nsj}+\sum_{nsj=1}^{ncont}a_{contj} \tag{7}$$

where a_{nsj} is the expected replacement cost of the nonstructural component nsj; a_{contj} is the expected replacement cost of the content contj; nc is the number of columns exposed to the fire; nf is the number of floor systems exposed to the fire; while nns and ncont are the number of nonstructural components and content items exposed to the fire.

For the area not exposed to fire, the expected loss in a column j, $E\left[LC_{cj}|NC,q_f,\mathrm{fiL}_i\right]$, is only associated with repairs needed for the EDP of deformation; no repair cost for the heat penetration is relevant. Therefore, the cost estimate is the sum of the product of the mean repair cost associated with the EDP of deformation by the probabilities to reach the different damage states for this EDP. For these columns not directly exposed to fire, the selection of EDP, the fragility functions and the corresponding loss functions refer to the studies of column damage under earthquake loads, as discussed in Section 4. Similarly, for the floors outside the fire compartment, the evaluation for the slab and its surrounding beams relies on the EDP of residual vertical deflection.

The loss due to the damage of non-structural components in the building is also considered. It is evaluated by summing the products of the expected repair cost of the nonstructural components at each damage state by the probabilities to reach these damage states. The conditional probability of being in each damage state for the nonstructural components is computed based on the probability density distribution of its EDP (peak interstory drift ratio for drift-sensitive nonstructural component) and fragility functions. The fragility functions and the corresponding loss functions for drift-sensitive non-structural components in the non-fire region can refer to the studies of drift-sensitive nonstructural components under earthquake loads.

The expected total loss due to the damage of structural/nonstructural components that have not been directly exposed to fire is the sum of the losses in each individual columns, the entire floor system (including beams and slab), and the repair cost of all the nonstructural components as given by Eq. (8). The condition $|NC, q_f, fiL_i$ is omitted for brevity. No damage is assumed for content that has not been directly exposed to fire.

$$E[LC_{nonfire,T}] = \sum_{cj=1}^{nc} E[LC_{cj}] + \sum_{fj=1}^{nf} E[LC_{fj}] + \sum_{nsj=1}^{nns} E[LC_{nsj}] \quad (8)$$

where nc, nf and nns are the number of columns, floor systems, and nonstructural components outside the fire compartment.

The repair cost for the entire building, $E\left[LC_T|NC, q_f, fiL_i\right]$, is the sum of the repair costs of structural/nonstructural components inside and outside the fire compartment. It is thus obtained by summing Eqs. (7) and (8). This total repair cost is conditional on the occurrence of a fire, in the compartment i, and with fire load q_f, which does not lead to a global collapse of the building.

6.2.2. Repair cost for the building including both collapse and non-collapse cases

In the cases where the building collapses under fire, the repair cost is the cost of demolishing and reconstructing the building, $E[RE_T]$. Assuming that $P(C|q_f, fiL_i)$ is the conditional probability that the building collapses, the expected total cost for the building conditional on the fire occurrence in this specific compartment with fire load of q_f is given by Eq. (9), which combines the cases of collapse and non-collapse (from Fig. 6).

$$E\left[LC_T|q_f, fiL_i\right] = E[RE_T] \times P(C|q_f, fiL_i) + E\left[LC_T|NC, q_f, fiL_i\right] \times (1 - P(C|q_f, fiL_i)) \quad (9)$$

6.2.3. Repair cost for the building accounting for the fire load distribution

Eq. (9) yields the repair cost for the building, conditioned on fire occurrence in one specific location, as a function of fire load q_f. Indeed, the fire load was taken as the IM. Now, by adopting a PDF for this IM, it can be integrated into the cost estimation. Here, the fire load in a compartment is assumed to follow a Gumbel type I distribution. The cost estimates are convoluted with the PDF of q_f to yield the probabilistic loss of the building given the fire occurrence in the considered compartment, see Eq. (10).

$$E[LC_T|fiL_i] = \int_0^\infty E\left[LC_T|q_f, fiL_i\right] p(q_f) dq_f \quad (10)$$

It is worth mentioning that the total repair cost $E[L_T|fiL_i]$from Eq. (10) may be greater than or close to the cost of demolishing and reconstructing the building. In this case, a decision should be made about whether the building should be repaired or should be demolished and reconstructed. Besides the cost, the time of operations can be relevant in making this decision, i.e. either the repairs or demolition and rebuilding may be faster, allowing the building to recover its functionality more quickly. This could be quantified in an extended framework accounting for indirect costs, which is outside the scope of this research. Here, it is assumed that if $E[L_T|fiL_i]$ is greater than E(RE) (the expected cost of demolishing and reconstructing), E(RE) is substituted to $E[L_T|fiL_i]$ in the following cost calculation.

6.2.4. Expected annual loss accounting for the fire likelihood

Finally, the expected repair cost of the whole building can be estimated by considering the different possible fire locations and their associated probabilities. This is expressed by Eq. (11).

$$E[LC_T|fi] = \sum_{i=1}^{nfiL_i} E[LC_T|fiL_i] P(fiL_i|fi) \quad (11)$$

where $P[fiL_i|fi]$is the conditional probability of the fire occurring in the compartment i provided a (structurally significant) fire starts in the building, and $nfiL_i$ is the number of possible fire locations in the building. The calculation of these probabilities can make use of Eq. (2).

Finally, it remains to account for the probability to have a fire start and grow to significant in the building. The expected annual loss (EAL) can be calculated as the product of the annual frequency of a structurally significant fire for the overall building, P(fi), with the expected loss conditional on fire occurring $E[L_T|fi]$. This is expressed by Eq. (12).

$$EAL = C_I C_L E[LC_T|fi] P(fi) \quad (12)$$

where the factors C_I and C_L account for inflation and location, respectively [446]. Since the expected loss of individual component has accounted for contractor overhead and profit, no factor for contractor overhead and profit will be included in Eq. (12).

In the discussion above, the focus is on the calculation of the expected annual fire loss from the expected repair cost of each component. However, it is noteworthy that the proposed framework can yield more information for decision-making, besides the expected annual fire loss, when the loss estimations are done probabilistically using the loss functions discussed in Section 6.1.

7. Case study

7.1. Description of the prototype building

A benchmark code-conforming five-story reinforced concrete space frame building is used as a prototype building to illustrate the application of the methodology. This building is adapted from [55] but redesigned by the authors for a less intense seismic load. The frame building consists of moment-resisting frames in both orthogonal directions with 5 bays in each direction, having a 7.0 m span length, resulting in a 35 m by 35 m square floor plan (as shown in Fig. 8). The story height is 4.0 m. The building was designed based on the 2010 NBCC seismic requirements with accompanying CAS Standard A23.3–04 "Design of Concrete Structures" used for proportioning and detailing of members [55–57]. The design dead load included a superimposed dead load of 1.33 kPa in addition to member self-weight. The live load was 2.4 kPa. The characteristic compressive strength of concrete is 30 MPa while the characteristic yield strength of the steel rebar is 400 MPa. Table 4

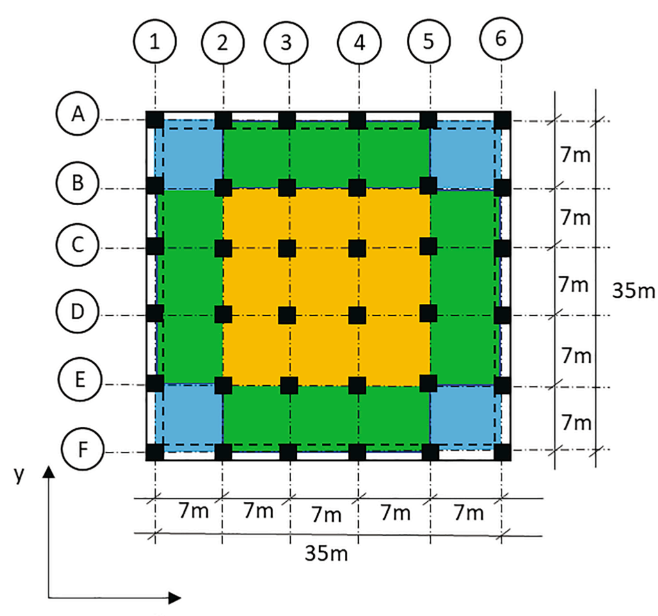


Fig. 8. Floor plan and different areas exposed to fire.

Table 4
Dimension of structural members.

		Size (mm × mm)	Steel reinforcement
Column	Corner Column 1–5	300 × 300	8–25 M
	Ext Column 1–2	350 × 350	8–30 M
	Ext Column 3–5	350 × 350	4–30 M + 4–25 M
	Int Column 1–2	450 × 450	16–30 M
	Int Column 3–5	450 × 450	4–30 M
Beam	Ext Beam	300 × 500	Top: 3–30 M Bottom: 2–25 M
	Int Beam	300 × 550	Top: 3–35 M Bottom: 3–25 M

provides the design details for each member. The slab is 200 mm thick with two layers of reinforcement.

The IBC classifies the building as a Type II A construction [58]. The fire-resistance rating requirements are one hour for the primary structural frame, the floor construction and associated secondary members, and the roof construction and associated secondary members. No requirements are specified for exterior and interior partition walls. The design concrete cover thickness for the primary members and slab meets the fire resistance requirements. The interior partition wall is a gypsum with steel studs (with both top and bottom fixed) and the exterior wall system is the 4.5 in-thick precast concrete panel. Information about the fire detection system, sprinkler system, and available fire brigade is the same to that described in Section 3.1.

7.2. Cost data

The total replacement cost of the building is estimated according to the data in the Square Foot Costs (RSMeans, 2019) [59]. The mean cost per square foot is $176 according to the occupancy and size of the building [60], inclusive of both structural and nonstructural components. Therefore, the total replacement cost of the building is $11,607,628. It is assumed that 38% of this cost comes from structural components and 62% from nonstructural components [61]. The demolishing cost of the building is $358,686, estimated from data in the Concrete & Masonry Costs (RSMeans, 2019 [46]). An assumption has to be made regarding the value of the content. Here, the replacement cost for the content in a building is assumed to be equal to the total replacement cost ($11,607,628) of all structural and nonstructural components in the building, based on the recommendation in [62]. It is further assumed that each of the 125 fire compartments in the building has the same nonstructural components and content. Therefore, the replacement cost of the nonstructural components for each fire compartment is $57,574 (obtained as $7,196,729 / 125) and the replacement cost of the content for each fire compartment is $92,861 ($11,607,628 / 125).

For structural components exposed to fire, the repair cost is estimated from the repair actions for the corresponding damage state, based on the Concrete & Masonry Costs (RSMeans, 2019 [46]) and FEMA 58 [51], and summarized in Tables 5–7. For each damage state, the repair cost is calculated according to each step of its corresponding repair actions (as shown in Tables 1–3). The cost for each step includes labor, materials, equipment, overhead and profit. If a component has to be replaced, the cost includes demolition and reconstruction of that structural component. In Table 5, the cost of scaffolding and forming is included in the construction cost, while the cost of shoring is included in the demolishing cost. It is assumed that the entirety of the non-structural components and content are lost in the area where the fire developed; the costs for the nonstructural components and content in fire region are their corresponding replacement costs, given above.

The classification of damage states and the corresponding repair cost for structural (columns, and floor systems) and nonstructural (interior walls, exterior walls and windows) components outside the fire area refer to [46,51], as summarized in Table 8. The cost for exterior walls

Table 5
Replacement cost for structural and nonstructural components.

	Construction Cost ($)	Demolition cost ($)	Total replacement cost ($)
Structural components			
Corner column	1,767	2,892	4,660
Exterior column 1–2	2,447	5,531	7,978
Exterior column 3–5	2,123	3,643	5,766
Interior column 1–2	4,824	10,708	15,532
Interior column 3–5	1,501	6,886	8,387
Exterior beam	2,490	1,243	3,733
Interior beam	2,672	1,788	4,459
Slab	11,371	3,779	15,150
Floor system(1)	17,604	a(1)	21,383 + a(1)
Nonstructural components			
Interior walls	–	–	2,088(2)
Exterior walls	–	–	49,150(2)

(1) The demolition cost for the floor system includes that of the slab plus that of the four perimeter beams; for corner compartments, a is equal to the demolition cost of two exterior beams and two interior beams; for exterior compartments, a is equal to the demolition cost of one exterior beam and three interior beams; for interior compartments, a is equal to the demolition cost of four interior beams.
(2) Data for interior walls and exterior walls are estimated from [51] which only gives the total replacement cost.

Table 6
Repair cost for the EDP of heat penetration (d300) for components exposed to fire.

	DS1 ($)	DS2 ($)	DS3 ($)	DS4 ($)
Corner column	85	418	704	4,660
Exterior column 1–2	99	500	805	7,978
Exterior column 3–5	99	500	762	5,766
Interior column 1–2	127	610	1,271	15,532
Interior column 3–5	127	610	826	8,387
Exterior beam	311	1,036	1,171	3,733
Interior beam	331	1,057	1,290	4,459
Slab	2,725	8,316	14,245	15,150

Table 7
Repair cost for the EDP of residual deflection ratio (RDR) for floors exposed to fire.

	DS1 ($)	DS2 ($)	DS3 ($)
Floor system	1,746	15,150	21,383 + a(1)

(1) See note in Table 5.

Table 8
Repair cost for the structural and nonstructural components outside the fire area.

	DS1 ($)	DS2 ($)	DS3 ($)	DS4 ($)
Corner column	177	884	3,535	4,660
Exterior column 1–2	180	899	3,595	7,978
Exterior column 3–5	156	780	3,119	5,766
Interior column 1–2	214	1072	4,288	15,532
Interior column 3–5	67	334	1,334	8,387
Exterior wall	1024	4096	24,575	49,150
Interior wall	538	1217	2088	–
Floor system	2095	18,180	21,383 + a(1)	–

(1) See note in Table 5.

includes the cost of windows. As discussed in Section 4.2, the repair cost of a floor system listed in Table 7 is increased by an empirical percentage, assumed as 20%, to cover surface damage in a floor system out of the fire area. It should be noted that no additional 20% cost is added to the cost for a floor system in DS3, since the whole floor system in DS3 will be replaced.

The costs listed in Tables 5–8 are comparable to those listed in studies for seismic loss estimations [4,5]. Finally, a 1.02 factor is applied to the final repair cost of structural and nonstructural components to account for miscellaneous costs, such as dust barriers. All the cost data above are the current values based on the national averages for materials and installation; therefore, no location adjustment and inflation adjustment are applied.

7.3. Response analysis and damage analysis

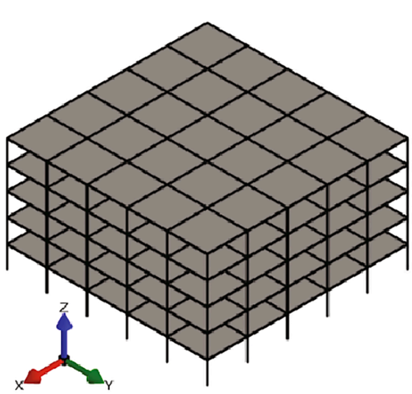
The nonlinear finite-element software SAFIR [39] is used to conduct the heat transfer analysis in the structure, followed by the transient structural analysis. In the thermal analysis, the section of each member is modeled as 2D solid models; the material models are STEELEC3EN for steel and SILCON_ETC for concrete. The thermal properties of those material models follow the recommendation of Eurocodes [63,64]. In the transient structural analysis, the columns and beams are modeled using beam elements while the slab is modeled using shell elements, shown in Fig. 3. The material models for beam elements are SICOPRBWE for concrete [26] and STEC3PROBA [65] for steel while the material models for shell elements are SILCOETC2D for concrete and STEC3-PROBA for steel. SILCOETC2D is based on the plasticity-damage formulation developed by Gernay et al. [66], with the temperature dependency of the properties follow the recommendations of EN 1992-1-2 [63]. The temperature dependency of the properties of SICOPRBWE is also in accordance with Eurocodes except the compressive strength. The compressive strength of SICOPRBWE is described as a Weibull distribution to incorporate the strength uncertainties at high temperature; however, the mean value of the temperature-dependent mean value is very similar to that recommended by Eurocodes. The two concrete models (SICOPRBWE and SILCOETC2D) incorporate the transient creep strain explicitly [67], with the reduction of concrete strength during the cooling phase following the recommendation of EN 1994-1-2 [68] that incorporate an additional reduction in strength during the cooling process, compared to the value at peak temperature. The STEC3PROBA material has the same expression of the stress–strain relationship as the steel of EN 1993-1-2 [64] but the reduction of yield strength with temperature follows the logistic EC3-based probabilistic model proposed in [65]. Perfect bond between steel reinforcement and concrete is assumed in the analysis. The inputs for those material models are described in [40]. Fig. 9a shows the full building model in SAFIR.

To reduce computational time, the model is simplified into that of Fig. 9b, where the slab above the fire compartment is modeled with shells and the columns directly connected to the fire-exposed story are modeled with beams using a relatively fine mesh, while the other RC frame members were modeled with beams using relatively coarse mesh without slabs. Fig. 9b shows the simplified model when a fire scenario on the third floor is studied, but other models are used when the fire is on another floor. This simplified method has been validated in a building very similar to the prototype one of this paper [40]. The deformations of the full building model and the simplified model are almost the same for that building under natural fire exposure.

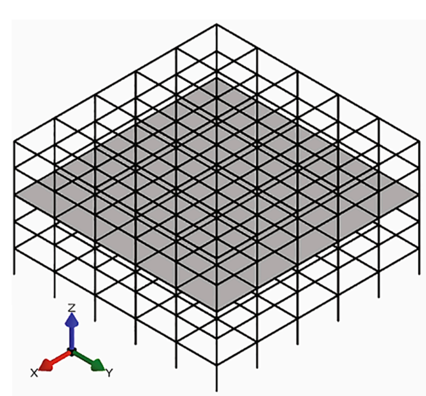
Analyses were conducted for several levels of fire load (IM) ranging from 200 MJ/m^2 to 1000 MJ/m^2, to cover the range of realistic fire loads in an office building. The fire load multiplied by a combustion factor 0.8 is the one used to calculate the gas temperature. The fire compartment is 7 m by 7 m (Fig. 8). To reduce the number of computations, the symmetrical layout is taken advantage of, meaning that at a given floor only one scenario is studied for the corner compartments (highlighted in blue in Fig. 8), for the exterior non-corner compartments (in green), and for the interior compartments (in yellow). This reduces the number of (single-compartment) fire locations from 125 to 15.

In this case study, a single random parameter is adopted in the probabilistic fire-thermo-mechanical analysis, namely the opening factor. This is in addition to the fire location and fire load (IM). These assumptions are adopted here because the purpose of the case study is to exemplify the methodology. Further studies will focus on the full probabilistic analysis of different prototypes to establish their expected annual fire loss. The uncertainty in the opening factor is adopted from the JCSS [69] model, $O = O_{max}(1 - \zeta)$, where O_{max} is the maximum possible opening factor assuming all the window glass is immediately broken when fire breaks out. The value $O_{max} = 0.0439\ m^{1/2}$ is assumed for each fire compartment. ζ is a random variable that follows a truncated (cut off at 1.0) lognormal distribution with mean 0.2 and standard deviation 0.2. A stratified sampling method is adopted to select the opening factor for each fire scenario based on the probability density distribution of opening factor; five opening factors are selected for each fire scenario. Again, this is a very low number of variations for the random parameter, but the objective is simply to illustrate the procedure. The gas temperature curve is calculated according to the parameter fire model in EN 1991-1-2 [38].

Given the fire occurrence in the compartment i with the fire load q_f, the PDFs of the EDPs are determined from the numerical simulations of the building response under fires with opening factors selected by the stratified sampling. Fig. 10a–c show the fitted PDF of the residual inclination of the corner column (RI), the heat penetration depth of the column section (d300) and the residual vertical deflection of the RC slab (RD), respectively. These figures deal with a fire in the corner compartment in the first story, with the fire load equal to 800 MJ/m^2. According to the fitted PDF curves of the EDPs and their corresponding fragility functions, the conditional probability of one component reaching or exceeding a specified damage state is calculated according to Eq. (4). By varying the fire load, the relationship between the

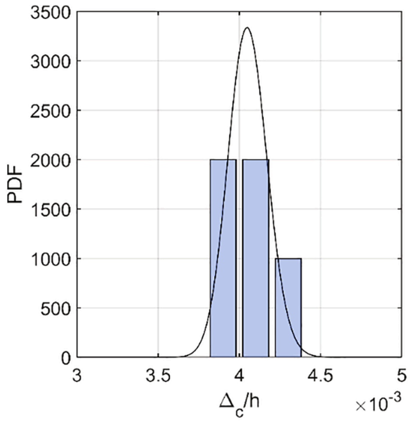


(a) Full building model

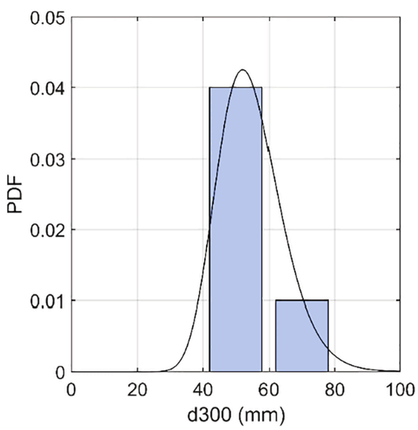


(b) Simplified model for fire scenarios at the third floor

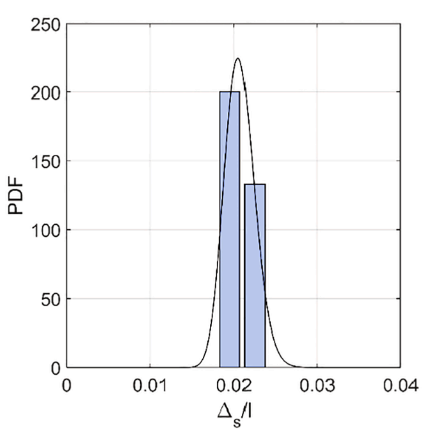
Fig. 9. Numerical model of the building in SAFIR.



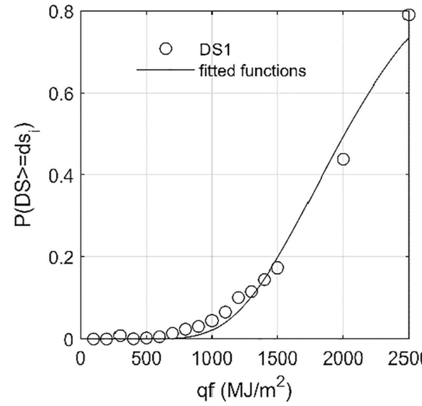
(a) PDF curve of the EDP of residual inclination (RI) for corner column



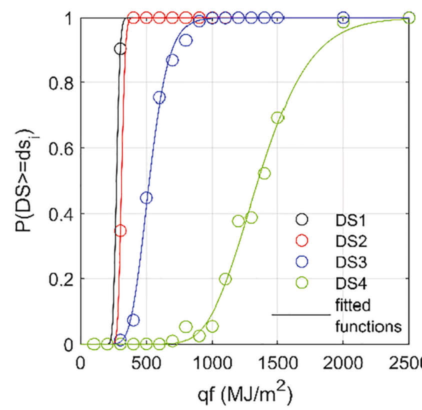
(b) PDF curve of the EDP of heat penetration depth (d300) for corner column



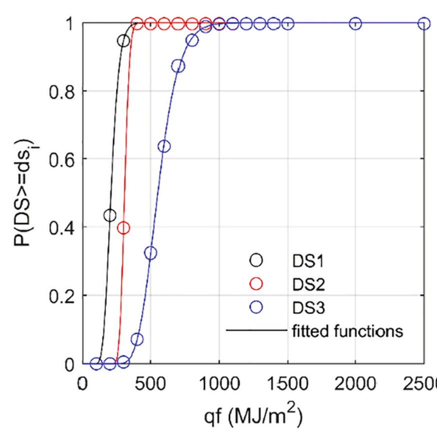
(c) PDF curve of the EDP of residual vertical deflection (RD) for floor system



(d) Probability of reaching or exceeding each DS, as a function of the fire load, for corner column RI



(e) Probability of reaching or exceeding each DS, as a function of fire load, for corner column d300



(f) Probability of reaching or exceeding each DS, as a function of fire load, for floor system RD

Fig. 10. Probability of reaching or exceeding the DS associated with different EDP for the corner columns and floor system under a fire scenario in the corner compartment of the first story.

probability of reaching or exceeding one specified damage state ($P\left(DS \geq ds_{j,i}|NC, q_f, fiL_i\right)$) and fire load ($q_f$) can be identified. This relationship is then fitted by lognormal cumulative distribution functions, as shown in Fig. 10d–f.

Collapse of the building does not happen in this case. Therefore, the probability of building failure in the fire is assumed to be negligible in this case study; this is supported by the conclusions in [17–20]. Future studies will consider a broader range of fire scenarios and probabilistic parameters, possibly incorporating failure cases.

7.4. Loss assessment for the building

The loss of the building due to a 7 m × 7 m compartment fire is calculated for each possible scenario. The final calculated loss of the building is the summation of the weighted estimated losses for different fire locations, summarized in Table 9. The numbers in Table 9 have been rounded to the nearest 100 dollars. Given the occurrence of a fire, the expected loss for the building is $187,900. This includes the damage loss of structural components ($27,100), damage loss of nonstructural components ($67,900), and damage loss of content ($92,900). It should be noted that those values are the fire loss conditional to the occurrence of structurally significant fire.

Fig. 11 plots the distribution of losses between structural components, nonstructural components, and content. Most of the loss is due to the fire damage to content (50%) in a fire compartment, rather than the structural (14%) or nonstructural (36%) components. It is noteworthy that assumptions were made regarding the content, including about the value of the content in a room, and its complete loss in case of fire. In an actual event, the content and nonstructural components exposed to fire might not be totally damaged, which may decrease the losses associated with these items and proportionally increase the losses due to structural fire damage. Nevertheless, it is notable that structural damage seems to account for a limited part of the fire losses in a RC building.

The fire losses due to structural damage are divided in approximately 89% for the floor systems and 11% for the columns. Most of the damage cost comes from the floor above the fire compartment. Another distinction can be made in terms of EDP. The damage cost to structural components in the fire-exposed area is at 71% related to the EDP of heat penetration (d300), while 29% is due to the EDPs of deformations. In cases with large fire load, the slab or the whole floor system had to be replaced due to the severe heat penetration, which contributed a lot to the final cost.

The compartment in which the fire develops did not have any significant influence on the estimated loss. Fires in interior compartments lead to slightly higher losses than exterior and corner compartments. In terms of floor, fires occurring in the fifth floor lead to slightly lower losses than for the lower floors, because in the latter case some damage develops in the upper stories.

The numbers above are conditional to fire occurrence. One can evaluate the estimated yearly losses for a community, using the likelihood of structurally significant fires and the number of buildings in the community. According to an NFPA report [70], the U.S. fire department responded to an average of 3,340 fires in office buildings per year during the five-year period of 2007–2011. For the sake of the discussion, let assume that 20% of those office buildings are RC frame buildings with compartments similar to those of the studied prototype. Further, according to NFPA 557, the average fraction ratio of fires that are “structurally significant” in office occupancies for fire-resistive construction is 6.75% [71]. Therefore, the total yearly direct loss due to structurally significant fires in RC frame office buildings could be estimated as 188k$ × 3340 × 20% × 6.75% = 8.5M$ for the U.S. According to the NFPA [54], the annual average cost associated to direct property damage from fires in offices (from any material and structural type) was 112 M$ from 2007 to 2011 (without including inflation).

Table 9
Results of loss estimation for the fire scenarios within a 7 m × 7 m compartment. The total estimated loss conditional to the occurrence of a structurally significant fire is $ 187,900.

Total estimated loss		$ 187,900
Area exposed to fire	Structural components	$ 19,600
	Nonstructural components	$ 58,700
	Content	$ 92,900
Area not exposed to fire	Structural components	$ 7,500
	Nonstructural components	$ 9,200

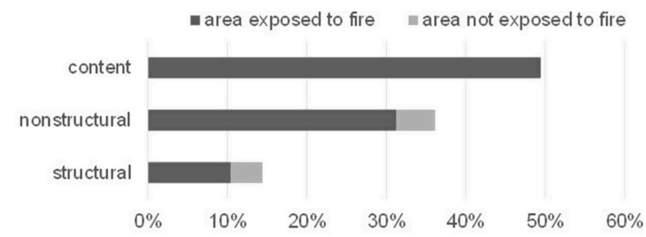


Fig. 11. Ratios of fire losses attributable to different parts of the RC building.

7.5. Loss estimation due to fire spreading to a larger area

In the study above, it was assumed that the fire remains confined in one compartment. However, a fire may spread to other compartments and lead to more damage to the building due to the failure of the compartmentation. Also, many modern offices use layouts that are characterized by large open space and hence less compartmentation, which would enable larger fire-affected areas. The influence of different parameters, such as sprinkler failure, in affecting the probability of fire spread, growth, and intensity, is also relevant in such a framework. In future works, it will be important to incorporate the potential spread of fire into the fire-loss estimation framework. This could be addressed through event tree analysis combining the different scenarios of fire locations and of single compartment versus fire spread in multiple compartments.

Fig. 12 shows the event tree for analyzing the conditional probabilities of fire confined in a single compartment, 7 m × 7 m, and fire spreading to a larger area, 21 m × 21 m. In this case, the fire begins in a single compartment. It is assumed that the probability of a fire remaining contained in its original compartment (7 m × 7 m) is 95% while the probability of that fire spreading to an area of 21 m × 21 m is 5%. According to the event tree, in any given compartment, it results in a conditional probability of 0.0076 of having a single compartment fire versus a conditional probability of 0.0004 of having the fire spread to 9 compartments. Those probabilities are conditional to the occurrence of severe fire (1.15×10^{-5} severe fires per year, calculated based on the assumption in Section 3). More exhaustive event trees including other fire spread scenarios can be adopted in the framework following a similar method. Once the probability of each fire scenario and their corresponding expected fire loss are estimated, the expected annual loss for a building,EAL, could be calculated, as described in Section 6.2.

8. Conclusion

This paper proposed a framework for the probabilistic estimation of yearly economic losses due to fire in concrete building structures. While similar performance-based loss assessment methods are used for seismic or wind hazards, no unified framework was available for fire. This paper aimed at filling this gap by adapting the well-known sequential PEER framework to the specificities of structures in fire. The following main conclusions are drawn from the study:

- The PEER seismic engineering methodology can be adapted to conduct fire loss assessment through sequential fire hazard analysis,

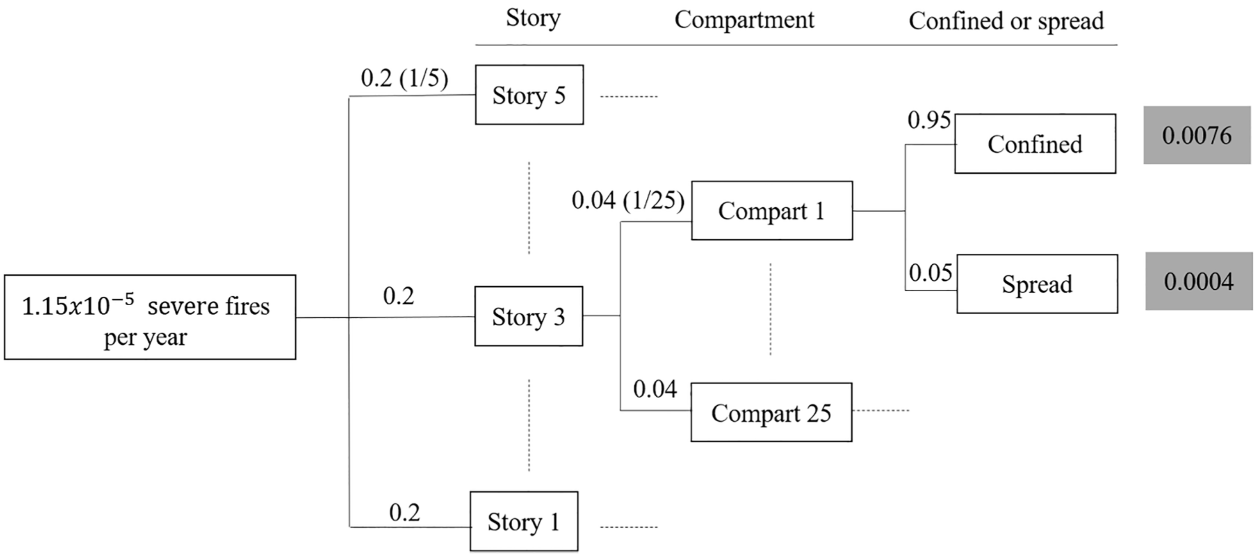


Fig. 12. Event tree for the conditional probabilities associated with each fire scenario.

response analysis, damage analysis, and loss analysis. This corroborates findings by other scholars [13], and is shown here at the scale of an entire building with new EDPs.

- A major difference in case of fire, compared with earthquake, is that the spatial–temporal evolution of the hazard within the building needs to be accounted for. This can be done by iterating the sequence of analyses for multiple fire scenarios, which cover the range of possible events including location of ignition and multi-compartment spread (besides being each dependent on their intensity measure). Event trees can be used to assemble the expected losses corresponding to different hazard scenarios.
- Another difference stems from the multi-physics nature of the structural fire problem, which can be overcome by defining damage measures that account for thermal and structural damage. For reinforced concrete structures, we proposed to adopt two types of engineering demand parameters (EDP) and associated damage measures: one for material degradation due to heat penetration at the section level, and one for structural degradation associated with deformations.
- The response analysis is performed using nonlinear FE modeling. This allows capturing thermally-induced deformations and restraints in structural assemblies, but hinders the possibility to perform full probabilistic analyses capturing uncertainties in fire scenarios and thermal-structural response of the building.
- Fragility functions and consequence functions can be defined for structural and nonstructural components. For components exposed to fire, we propose functions based on literature data for post-fire concrete repairs. For components outside the fire area, damage comes from deformations and therefore the functions can (under certain assumptions) be adopted from seismic engineering.
- A five-story RC frame building is presented as a case study. Direct fire losses in case of a 49 m^2 compartment fire are expected to be 188 k$ consisting mostly in content losses (50%) and nonstructural components damage (36%), with structural components damage at only 14%. For the structure, direct losses are mainly associated with floors (rather than columns) and with the thermal EDP (rather than the deformation one). Under a basic assumption for the ratio of RC buildings in the U.S. building stock, the case study predicts yearly direct fire losses in concrete office buildings of about 8.5 M$ for the U.S.
- Event tree analysis can be used for the proposed framework to incorporate losses associated with any fire scenario, including cases of multi-compartment spread within a building. The expected losses are weighed by the conditional probabilities associated with each fire event, to yield the expected annual loss for a building.

The developed framework is intended as a first step toward establishing a performance-based approach to evaluate the expected cost of fire and the optimum investments in fire safety for structures. At this stage, it still relies on engineering judgment for several important assumptions, which warrant further research. Current limitations include the lack of data for defining fragility functions and consequence functions for the fire damage of structural components; the prediction of the fire scenarios (including failure of compartmentation and fire spread); and the expected losses for nonstructural components and content (which were conservatively estimated in this study). The latter point is particularly important since the case study showed that content and nonstructural components contributed the most to direct fire losses. Another drawback relates to the computational cost associated with nonlinear thermo-mechanical analyses. In the case study, only a limited number of scenarios and parameter realizations were evaluated. For broader application of the methodology, advances in reduced-order modeling for structures in fire (e.g. surrogate models) are needed.

Declaration of Competing Interest

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: Under a license agreement between Gesval S.A. and the Johns Hopkins University, Dr. Gernay and the University are entitled to royalty distributions related to the technology SAFIR described in the study discussed in this publication. This arrangement has been reviewed and approved by the Johns Hopkins University in accordance with its conflict of interest policies.

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