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FRAGILITY ANALYSIS OF A STEEL BUILDING IN FIRE

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ABSTRACT

Community resilience to extreme events is an issue of increasing concern in our interconnected and urbanized societies. This work provides a framework to evaluate the response of a community of buildings to fire following earthquake, a potentially highly destructive cascading multi-hazard event. In a previous part of the work, a model has been developed to predict the probability of ignition in a building due to an earthquake. Given an ignition in a building, the probability of the structure exceeding certain limit states must be evaluated in order to quantify the expected damage loss. Adopting an approach similar to that used in seismic engineering, fragility functions can be developed for structures subjected to fire. The methodology is described here for a prototype nine-story steel frame building. In developing the fragility functions, uncertainties in the fire model, the heat transfer model and the thermo-mechanical response are considered. In addition several fire scenarios at different locations in the building are studied. The demand on and capacity of the system are assessed probabilistically in terms of critical temperature. The developed fire fragility functions yield the probability of exceedance of predefined damage states as a function of the fire load in the building. Future works will aim to implement fire fragility functions into a GIS based risk assessment software platform for assessment of the expected risk and cost associated with fire following earthquake for a community of buildings.

1. INTRODUCTION

Recent extreme events have emphasized the need for disaster-resilient communities. Among the major threats to the built environment, cascading multi-hazard events such as fires following an earthquake can cause major social and economic losses in a community as observed for instance in the 1989 Loma Prieta and 1994 Northridge events. Structural engineering has a key role to play in the evolution towards more resilient communities, by addressing multi-hazard analysis and resilient building design.
This research project focuses on the response of a community of buildings to fire following earthquake. In a previous part, a model has been developed to predict the probability of ignition in a building due to an earthquake [1]. This model estimates, based on the intensity of an earthquake and the characteristics of a community, the total number of ignitions expected in the community as well as their distributions among the different structural types of buildings. As the next step, the present part of the work aims at predicting the expected structural damage due to fire in buildings in which an ignition was detected. This expected damage depends on many uncertain parameters related to the building structure, fire scenario, heat transfer processes and thermo-mechanical response. Adopting an approach widely used in seismic engineering, we propose to develop fragility functions for different typologies of structures to characterize their vulnerability to fire.
This paper presents a framework to develop fire fragility functions for a steel building. The methodology is described and applied to a practical example consisting in a prototype nine-story steel frame building.

2. METHODOLOGY

A methodology to construct fire fragility functions for steel buildings is presented in this Section.
Fire fragility is a conditional probability statement describing the vulnerability of a system subjected to a given fire intensity. In developing the fire fragility functions, it is assumed that a fire that is able to endanger the structure has started; such fire is referred to as structurally significant fire. Hence, the factors that influence the probability of a structurally significant fire to happen, such as the presence of fire detection or sprinkler systems, have no effect on the fragility functions.
A system vulnerability to a certain hazard depends first and foremost on the intensity of this hazard. In seismic engineering, it is common to choose the peak ground acceleration g as the intensity measure; this parameter appears thus on the x-axis

of the seismic fragility curves. In fire engineering, a wise choice could be the average fire load (in MJ/m² of floor area), because: (i) the fire load is one of the main parameters affecting the intensity of a fire [2], (ii) it may vary in a significant range, and (iii) it has a straightforward definition that is easily understood by the different stakeholders involved in fire safety. Consequently, the fire load is chosen as the intensity measure for the fire fragility curves. Given the fire load, the fragility functions yield the probability of exceedance of predefined damage states. These damage (or limit) states are specific to the structure under study; they must be defined to represent properly successive levels of damage such as moderate, severe or complete damage.

The development of fragility functions requires the probabilistic assessment of the capacity of the structure, relative to predefined damage states, and the probabilistic assessment of the demand placed on the structure due to fire. It must incorporate explicitly the uncertainties in the fire model, the heat transfer processes and the thermo-mechanical response.

For structural steel members in fire, the exceedance of a damage state depends on the exceedance of a certain temperature threshold in the section, referred to as a critical temperature. This concept of critical temperature is convenient because it allows defining the structural capacity purely as a function of the maximum temperature reached in the section. The structural (capacity) analysis can thus be decoupled from the thermal (demand) analysis. The capacity analysis of the structure yields a probability distribution function (pdf) for the critical temperature associated to a given damage state. The demand analysis yields a pdf for the maximum temperature reached in the sections of the structural members, as a result of the fire and thermal analyses. The two problems are treated separately and, in the end, the outputs of both analyses are compared. Failure occurs when the temperature reached in the section (demand analysis) exceeds the critical temperature (capacity analysis). This procedure is illustrated in the flowchart of Figure 1.

In a multi-compartment building, multiple fire scenarios are possible. The fire fragility functions of the building should encompass these different scenarios to capture the overall fire vulnerability. Therefore, the methodology illustrated in Figure 1 is in fact applied several times during the fragility analysis of a building, for varying scenarios (where the scenario *i* corresponds to a fire located in the compartment *i*).

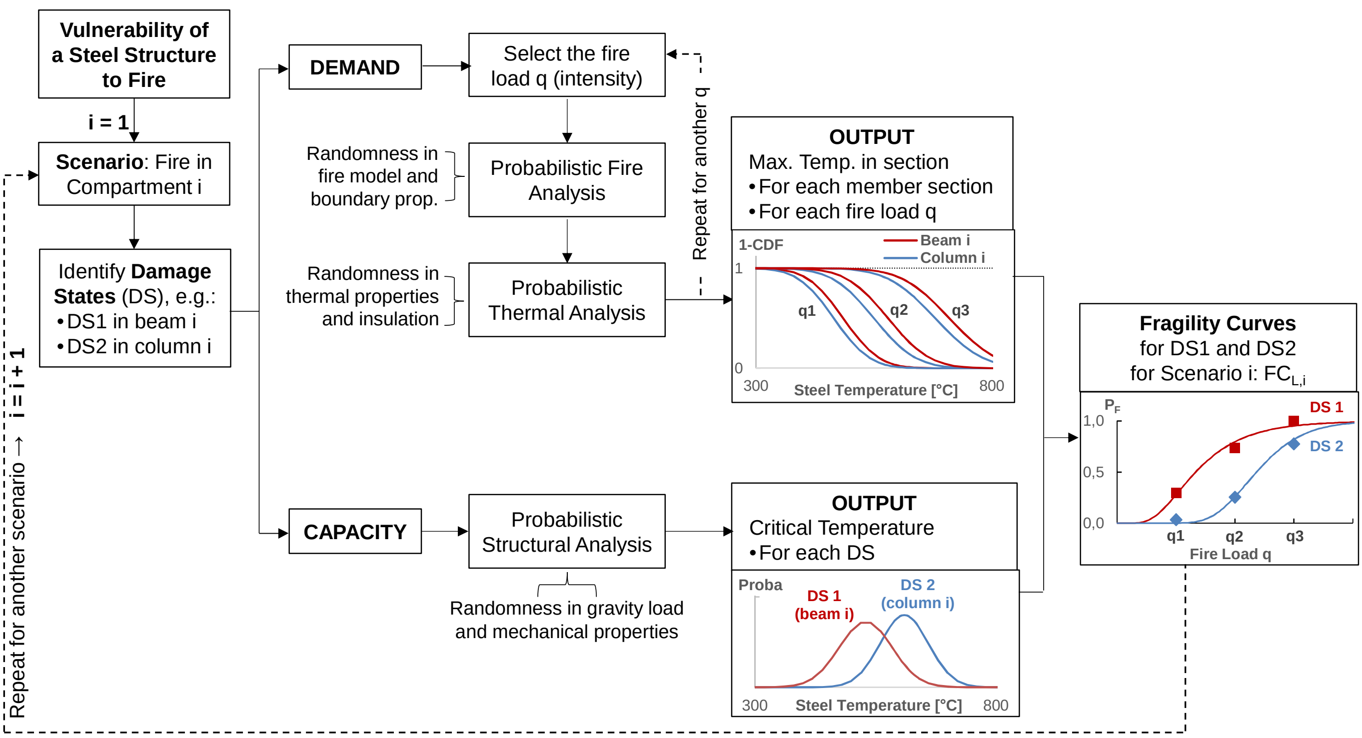

Figure 1. Flowchart for the development of fragility curves for a steel building.

Each time the methodology is applied (considering a specific scenario), it yields one fragility curve per damage state, representative of the vulnerability at the scale of the fire compartment. In the end, the fragility curves associated with a given damage state but different scenarios must be combined into a single curve, representative of the overall vulnerability at the scale of the building.

3. PROTOTYPE STEEL BUILDING

The methodology is applied to a building prototype that consists in a nine-story steel frame building. The building is 45.72 m by 45.72 m in plan, consisting of five bays of 9.144 m in the two directions. The structure is composed of four moment resisting frames on the perimeter, and four interior gravity frames, see Figure 2. The columns of the interior frames are continuous on the nine-story but the beams have pinned connections (statically determinate beams). The total height of the building is 37.182 m, divided between a first floor of 5.486 m high and the eight other floors of 3.962 m high. The sections of the beams and columns for the interior frame are given in Table 1.

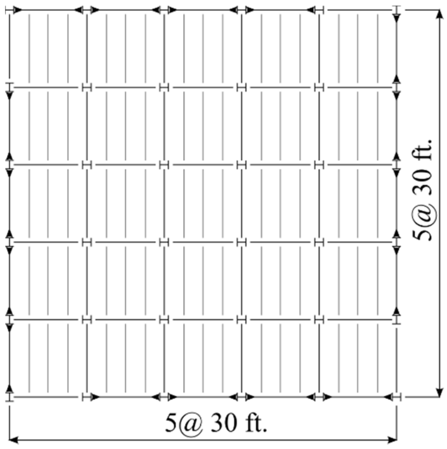

Figure 2. Plan view of the prototype 9-story steel building.

The perimeter frames, designed for seismic resistance, are made of relatively heavy protected steel sections. For instance, the moment frame columns sections range from W14x342 to W14x665. As such, they are not likely to be affected significantly by a fire, and this has been confirmed by a previous study [3]. In contrast, gravity frames have a higher utilization ratio, and they are most likely to reach their critical temperatures first [4]. As a consequence, this work focuses on the effect of the fire on the gravity frames only.

Table 1. Sections of the structural members for the gravity frame.

Level	Beam	Column
9	W18x40	W14x43
7-8	W21x44	W14x53
5-6	W21x44	W14x68
3-4	W21x44	W14x82
1-2	W21x44	W14x109

The concrete slab is 102 mm depth. The steel sections (beams and columns) are protected with a sprayed fire-resistive material (SFRM) of nominal thickness 39 mm. The nominal values of the steel yield strength and Young modulus are 345 MPa and 200,000 MPa, respectively. The concrete compressive strength is 28 MPa.

4. DAMAGE STATES

The level of structural damage of the building due to fire will be assessed based on predefined damage states. For the prototype steel frame building with beams with pinned connections, two structural damage states are considered, one relative to the beams and one relative to the columns:

- DS1: Maximum bending resistance of the beam, when the bending capacity of the beam is exceeded and the mid-span vertical deflection increases dramatically;
- DS2: Maximum resistance of the column, when the column fails with a sudden increase in transversal deflection, whether due to exceedance of the buckling resistance of the column or exceedance of the section plastic capacity under combined compression and bending.

These damage states are illustrated in Figure 3.

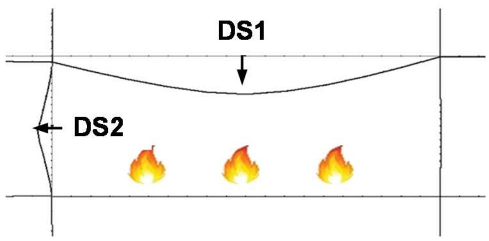

Figure 3. Damage states for the steel frame building.

5. PROBABILISTIC FIRE ANALYSIS

For a steel building in fire, the demand placed on the structure due to fire can be expressed in terms of the maximum steel temperatures reached in the sections of the structural members. In order to evaluate these maximum steel temperatures, a fire analysis must first be conducted to predict the evolution of temperature in the compartment.

The development of the fire in a building depends on many uncertain parameters. For instance, it depends on the ignition location, the fire load, the openings in the compartment and the thermal properties of the boundary of enclosure. All these parameters have sources of uncertainty, and they influence the spatial and temporal variability in the gas temperature. Since the fragility analysis of the building aim at evaluating its structural reliability in fire, this analysis should deal explicitly with the uncertainties that affect the system. However, it is neither practical nor relevant to consider all possible configurations and sources of uncertainties. Based on literature and engineering judgment, only the most significant sources of uncertainties are selected, considering a trade-off between computational efficiency and accuracy.

The location of the fire is a priori unknown. This is a major source of uncertainty and the vulnerability of the building may depend significantly on this parameter. Consequently, different analyses will be conducted to consider the different possible locations (i.e. fire compartments) and evaluate the response of the structure in each case (see the main loop in Figure 1).

For a given fire location, the development of the fire depends on random parameters that govern the evolution of gas temperature with time in the compartment. Among these, fire load has a paramount importance and was chosen as the intensity measure of the fire hazard. Thus, different levels of fire load, ranging from 100 to 2000 MJ/m² of floor area, are considered in the analysis (see the secondary loop in Figure 1). Other parameters, such as the opening factor or the thermal properties of the boundary of enclosure, are assumed here to be deterministic, based on a typical compartment of the prototype building. It is assumed that the walls and ceiling of the prototype building are lined with gypsum plaster board, with the following properties [5]: conductivity k_g = 0.48 W/mK; specific heat c_g = 840 J/kgK; density ρ_g = 1440 kg/m³. This assumption is conservative compared to concrete walls, because the latter result in a higher thermal inertia. Of course, other assumptions could be made and would not modify the presented methodology.

The model adopted to generate the time-temperature evolution in the compartment under study is the natural fire model developed by Quiel and Garlock [6]. This model is based on the study of the real fire that developed in the One Meridian Plaza (1MP) Building of Philadelphia. The natural fire curve is dependent on the maximum fire temperature and can be scaled accordingly (Figure 4). To determine the maximum fire temperature, the method from Annex A of Eurocode 1991-1-2 is adopted [7] and applied for different levels of fire load between 100 and 2000 MJ/m².

As a result of the probabilistic fire analysis, a set of time-temperature curves is thus generated for each fire compartment under study.

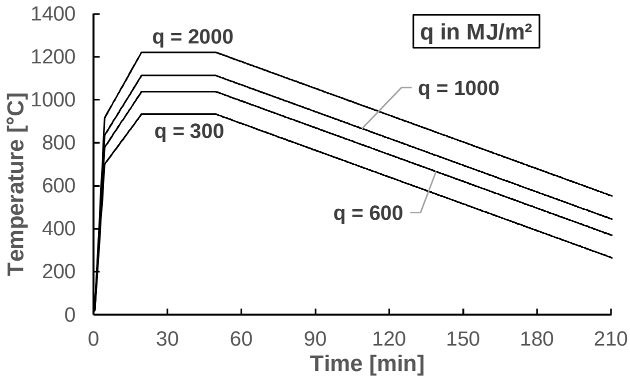

Figure 4. Natural fire model [6] scaled as a function of the maximum fire temperature.

6. PROBABILISTIC THERMAL ANALYSIS

The next step consists in assessing the temperature evolution in the sections of the structural members. This is done by conducting heat transfer analysis, using the time-temperature fire curves plotted in Figure 4 as an input.

The heat transfer processes depend on the thickness and thermal properties of the insulating material, the thermal properties of steel and the section geometry. These parameters have sources of uncertainties that will influence the temperatures in the sections. Sensitivity analyses for steel members protected with SFRM have shown the prevailing importance of the thickness and conductivity of SFRM [8]. As a result, the latter are treated as random parameters in the model, whereas SFRM density and specific heat are treated as deterministic. For the SFRM thickness, a lognormal distribution is assumed with a mean value equal to the nominal value of 39 mm plus 1.6 mm and a coefficient of variation of 0.2 [9]. Regarding the SFRM conductivity, the probabilistic model proposed by Elhami Khorasani et al. [10] is adopted. On the other hand, thermal properties of steel are treated as deterministic due to their relatively low variances; the properties are taken from Eurocode [11].

Different methods can be employed to conduct the heat transfer analyses, ranging from advanced numerical methods (FEM) to simple calculation models that have been validated for prediction of the temperature evolution in protected steel sections. One advantage of the proposed methodology, in which the thermal part (demand assessment) and the structural part (capacity assessment) are treated separately, lies in its flexibility: the method used to solve the thermal problem does not need to be the same as for the structural problem. Hence, the most efficient approach for each analysis can be adopted.

Here, the thermal analyses are performed using the finite difference formula of EN 1993-1-2 Section 4.2.5.2 [11]. This formula, also referred to as lumped mass approach, yields the uniform temperature in the cross-section of a steel member at each time step and it can be used for insulated and bare steel members. It is chosen for its computational efficiency and its wide acceptance among the structural fire engineering community.

For a given structural cross-section and a given time-temperature fire curve, Monte Carlo Simulations are conducted using the Eurocode formula and varying the thermal properties of the insulation material (thickness and conductivity). For each fire curve, 1000 realizations are computed. The process is then repeated for the same compartment using a different fire load, yielding the distribution of maximum temperature in the steel section corresponding to each fire load. The same methodology is then applied to each different cross-section type in the building.

As a result, the distribution of maximum temperature reached in the sections of the structural members is obtained, e.g. see Figure 5 for the column of the fourth floor. The result is presented in the form of the complementary cumulative distribution function of the maximum steel temperature.

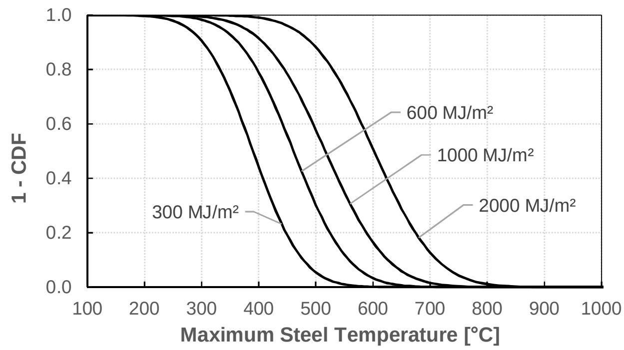

Figure 5. Distribution of maximum steel temperature in the fourth floor column for different levels of fire load, considering variability in SFRM thickness and thermal properties.

The curves of Figure 5 represent probabilistically the thermal demand placed on the system due to fire. They will be used to construct the fragility curves.

7. PROBABILISTIC STRUCTURAL ANALYSIS

This section investigates the structural response of the building subjected to fire, in order to assess the probabilistic capacity of the building with regards to the predefined damage states.

Following the methodology of Figure 1, the capacity is assessed in the temperature domain. Hence, the objective is to define the pdf of the critical temperatures associated with each damage state for the prototype steel frame building.

The capacity of the system depends on parameters with uncertainty, among which the most significant are the mechanical properties of steel at high temperature and the applied gravity loads. Regarding the reduction of the steel mechanical properties with temperature, the probabilistic model from [10] is adopted. Randomness in the gravity loads is also considered. The factors applied to the dead and live loads are respectively 1.05 and 0.24 and these factors are weighed by probabilistic load factors according to [8].

The capacity assessment is done using Monte Carlo Simulations (MCS) based on non-linear FE structural analyses. Although more computationally efficient methods (such as simple calculation models) would be preferable in the framework of probabilistic analysis, the choice of the FE method for the structural (capacity) assessment is dictated by the complexity of the response for the studied structure in fire. The gravity loads and mechanical properties of steel are taken as random variables.

7.1 FINITE ELEMENT MODEL

The building structure is modeled in the non-linear finite element software SAFIR [12] developed at University of Liege. SAFIR allows conducting a thermal analysis of the sections of the structural members, followed by a structural analysis of the building at high temperature. Here, the response of one interior frame is studied in its plane, meaning that the model is built in two dimensions.
First, a two-dimensional thermal FE analysis is conducted for each heated member (beams and columns) using cross-sections that are discretized in fibers. The modeling of the beam section includes a 2.3 m effective width of concrete slab, i.e. one quarter of the span, see Figure 6.

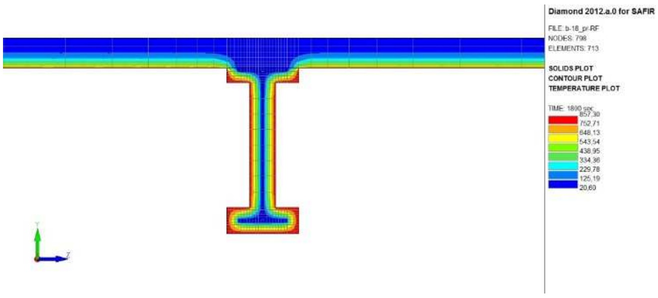

Figure 6. Thermal analysis of the protected beam.

Then, a structural analysis is performed using three-noded, two-dimensional beam elements (Figure 7). The time-temperature evolution in each fiber results from the previously conducted thermal analysis. The structural analysis takes into account geometrical and material non-linearity, including large deflections. The composite effect of the concrete slab is taken into account in the structural analysis assuming a full transfer of horizontal shear at the steel-concrete interface.

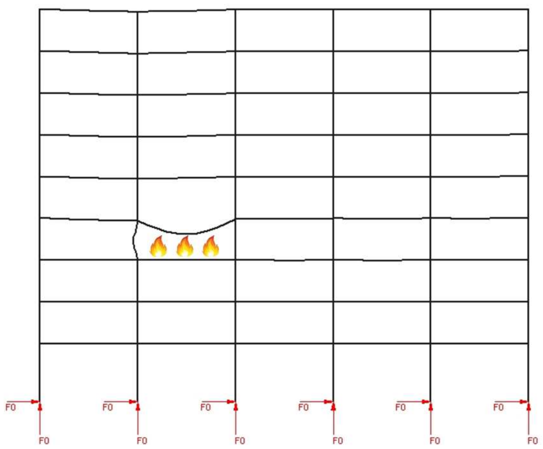

Figure 7. Structural model of the building.

The stress-strain relationship for steel at high temperature is adopted from Eurocode [11] and the relationship from concrete is taken from Gernay et al. [13]. The evolution of yield strength and Young modulus with temperature is evaluated using probabilistic models [10], to account for the uncertainties in these parameters.
Figure 7 presents the structural model, with the deflected shape at collapse (amplified two times) for a fire in the second bay of the fourth floor.

7.2 STOCHASTIC FE SIMULATIONS

In the FE simulations, a set of values is randomly selected for the gravity loads and the evolution of steel mechanical properties with temperature. Then, the temperature in the section of the structural members is increased over time, which leads to a decrease in their load bearing capacity and an increase in the displacements. This temperature increase is conducted until the predefined damage states are reached. At the time when a damage state is reached, the temperature in the corresponding structural member is recorded as the critical temperature. The procedure is then repeated for a new set of values for the random parameters.
The critical temperature is independent on the particular time-temperature evolution curve in the section. Obviously, the time at which a structural damage state is reached depends, amongst others, on the physics of the fire and thermal properties of the structure (e.g. level of thermal protection); yet the temperature at which this damage state is reached is independent on these parameters. This critical temperature concept, which is at the base of the methodology illustrated in Figure 1, is for instance prescribed in Eurocode [11], and its validity for the specific structure studied here has also been verified by FE simulations.
As a result, the temperature evolution in the sections of the structural member, used as an input in the structural FE analysis, can be any time-temperature relationship. In this work, the evolution of temperature in the sections is computed by a deterministic thermal FE analysis, considering the standardized ASTME119 fire and no thermal protection on the steel members. Since this fire is monotonically increasing, so is the temperature in the sections, so that any subsequent structural analysis can be run until complete failure, i.e. until attainment of all damage states. Note that

this methodology cannot be used to study damage states specific to the cooling phase (such as tension failure in the connections). It is important to note that the thermal FE analysis is run only once, in order to generate a temperature history in the sections; this temperature history is then used in all the structural FE analysis run in the MCS. Another temperature history could be used and would lead to the same results in terms of pdf of critical temperature.

7.3 PDF OF CRITICAL TEMPERATURE

Considering a fire in the building second bay of the fourth floor (see Figure 7), 40 realizations are computed using the software SAFIR and the probabilistic distributions for gravity loads and steel mechanical properties.
For each structural analysis, the time at which the damage state is reached in the beam and in the column is recorded. The evolution of temperature with time in the sections of the beam and the column is known as a result of the thermal FE analysis. Hence, the time corresponding to the attainment of a damage state can be mapped to the average temperature in the section of the member at this time.
As a result, the pdf of capacity related to each damage state is obtained in terms of critical temperature in the steel section. For instance, Figure 8 shows the pdf of the critical temperature at which the damage state in the beam (DS1) and in the column (DS2) are reached, assuming a fire in the second bay of the fourth floor.
For other fire locations, the distribution of critical temperature (i.e. the capacity) will be different, because of different cross-sections of the structural members or applied gravity loads. Therefore, the probabilistic structural analysis must be repeated for each fire location in the building for which the significant parameters have different nominal values.

8. FRAGILITY CURVES

The probability distributions for demand and capacity obtained in Sections 6 and 7 allow for deriving analytical fragility functions for the building.

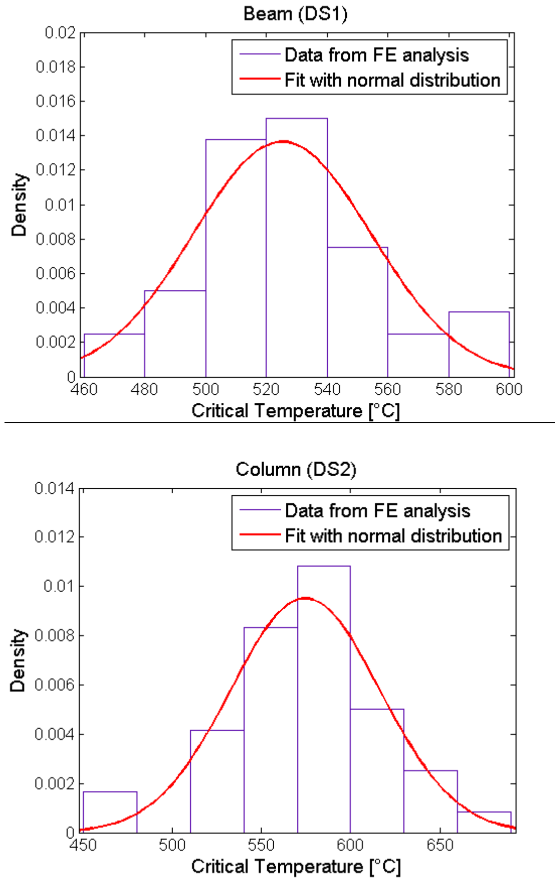

Figure 8. Distribution of the critical temperature for the beam damage state (DS1) and column damage state (DS2), considering a fire in the 2nd bay of the 4th story.

In Section 8.1, the methodology to derive a fragility curve corresponding to a given damage state (e.g. the column damage state) and a given fire location (e.g. the compartment located in the second bay of the fourth floor) is illustrated. The same methodology applies to the other fire locations and damage states.
In Section 8.2, the fragility curves corresponding to different fire locations in the building are combined, in order to derive a single fragility curve per damage state for the prototype building.

8.1 FRAGILITY CURVES FOR ONE SPECIFIC FIRE LOCATION

The fire is assumed to develop in the second bay of the fourth floor and the focus is here on the fragility curve for the column in this compartment (section W14x82). The pdf of capacity and the complementary cdf's of demand have been plotted in Figure 9(a) for this column. The different curves for demand correspond to different levels of fire load.

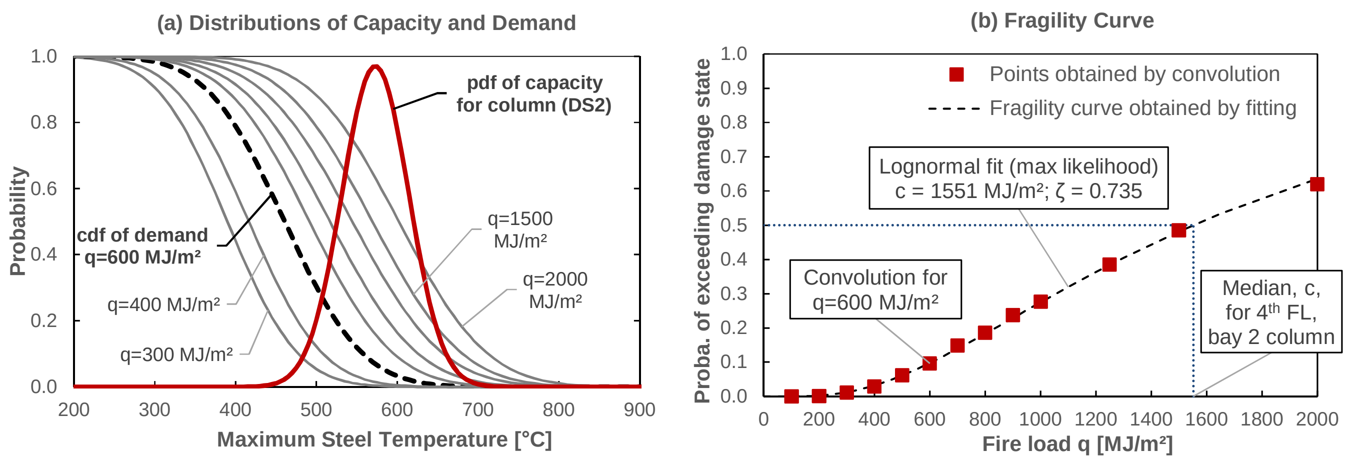

Figure 9. The fragility points are obtained by convolution of pdf of damage state and complementary cdf of demand.

For a given fire load, the conditional probability of failure can be computed using Eq. 1, i.e., by convolution of the pdf of capacity and the complementary cdf of demand corresponding to this fire load.

$$P_{F|H_{fi}} = \int_0^\infty \left[1 - F_{D|H_{fi}}(\alpha)\right] f_C(\alpha) d\alpha \qquad \text{Eq. 1}$$

In Eq. 1, $P_{F|H_{fi}}$ is the probability of failure conditional to the occurrence of a fire H_{fi}; the demand D and capacity C are random variables characterized by their pdf $f_D(\cdot)$ and $f_C(\cdot)$; and $F_{D|H_{fi}}$ is the cdf of the demand relative to the fire H_{fi}.

Repeating the operation for each fire load level yields several points relating the fire load level and the conditional probability of failure, see Figure 9(b).

Then, the fragility function is built by fitting of the obtained points, assuming that it is a lognormal function in the form of:

$$F(q) = \Phi\left[\frac{ln(q/c)}{\zeta}\right] \qquad \text{Eq. (2)}$$

with q the fire load (MJ/m²) that characterizes the fire and Φ[·] the standardized normal distribution function. The two parameters c and ζ characterize the fragility function; they must be determined to maximize the best fit with the data points resulting from the analysis. This fit is performed using the maximum likelihood function.

The same process is applied for deriving the fragility functions relative to the other damage states and other fire compartment locations.

8.2 COMBINED FRAGILITY CURVES

Using the procedure of Section 8.1, fragility curves associated with the beam and the column damage states are constructed for each different compartment fire locations in the building. Due to the number of possible fire scenarios, this results in many different fragility curves associated with the column damage state and many different fragility curves associated with the beam damage state. In view of the process of fire disaster evaluation of a community of buildings, a single fragility curve per damage state should be used to model the vulnerability of an entire prototype building representative of a given typology.

A method has been proposed in the literature [14] for constructing combined fragility curves from individual fragility curves developed for structures with similar structural attributes. In this case, the individual fragility curves represent specific fire locations in the building. The objective is to combine them in order to derive a fragility curve (one per damage state) that does not depend on the location of the fire, but captures the overall vulnerability of the building.

The idea consists in assuming that the combined fragility curves can also be approximated by lognormal functions, in a form similar to that of Eq. (2). The two parameters, mean and standard deviation of the combined lognormal distribution, are calculated on basis of the corresponding parameters for the individual fragility curves, taking into account the relative likelihood of each fire scenario. The reader is referred to [14] for more

comprehensive information about the combination process.
In the end, the column DS fragility curves corresponding to different compartment fire locations are merged into one single column DS fragility curve for the entire building (and similarly for the beam DS fragility curves).
The combined fragility curves associated to the two damage states for the entire building are plotted in Figure 10. Based on the average value of the fire load that is expected in the building, the building fragility curves yield the probability of exceeding each damage state, conditional to the occurrence of a structurally significant fire in one compartment of the building.
In this probabilistic model, it is not necessary to assess in which particular compartment the fire develops. Instead, the model provides a probabilistic assessment of the degree of damage for a building similar to this prototype building, in which a compartment fire develops somewhere and, despite the active fire protection measures, reaches a point where it is able to endanger the structural stability. The effect of the passive fire protection (SFRM) are incorporated in the fragility curves.

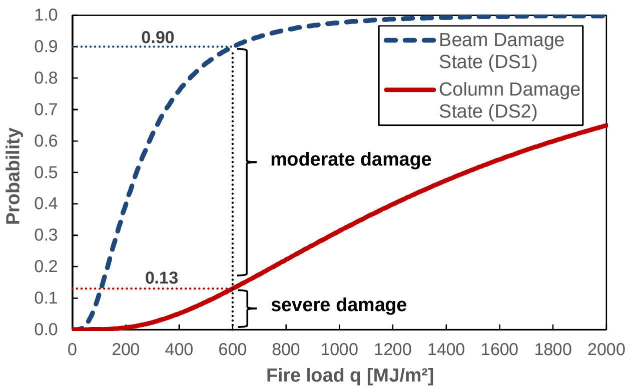

Figure 10. Combined fragility curves for the prototype nine-story steel frame building, representing the overall vulnerability of the building.

For instance, assuming that the fire load is equal to 600 MJ/m² (in average) in the building, Figure 10 shows that the probability of exceeding the beam damage state (DS1) is 90% and the probability of exceeding the column damage state (DS2) is 13%. For this specific building structure, the beam damage state is always reached prior to the column damage state. Therefore, the probability of exceeding the damage state in the beam (DS1) but without collapse of the column is 77% (0.90-0.13). The latter situation can be referred to as a "moderate damage" in the building due to fire. On the other hand, in case of failure of both the beam and the column, the structure is said to experience "severe damage". The probability of not reaching any of the two considered structural damage state is obtained as the complement of the probability of DS1, i.e. 10% for a q of 600 MJ/m².

9. CONCLUSIONS

This study proposes a novel methodology for developing fire fragility functions for steel buildings and applies it to a nine-story steel frame building.
The methodology developed in this work can be applied for constructing analytical fire fragility curves for other typologies of steel structures. One key aspect for steel structures lies in the separation between the thermal and the mechanical problem, taking advantage of the fact that the capacity and demand can be characterized in the temperature domain.
The fragility curves presented as an application characterize the vulnerability of a prototype multi-story steel frame building in fire. They can be applied in a probabilistic fire disaster assessment of a community of buildings. First, the probability of structurally significant fire in a community of buildings is estimated, per year or per accidental event (e.g. following an earthquake). Second, the fragility curves are used to predict the level of structural damage for each individual building subject to a fire as a function of the fire load in this building. These results eventually allow for an estimation of the expected damage loss due to fire in the community.
Buildings within a community are made of varied structural types and materials. Hence specific fragility curves are needed to characterize different types of structures. Further works shall focus on the development of reliable and accurate fire fragility functions for these different types of structures.

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