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Fire Fragility Functions for Community Resilience Assessment

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ABSTRACT

This work provides a framework to evaluate the response of buildings in a community to fire following earthquake. As part of the framework, the paper discusses two methodologies: (1) how to develop fire fragility functions; (2) how the fire fragility functions can be used in conjunction with an original fire ignition model to estimate the potential losses in a community from fire following earthquake. The paper focuses in particular on the development of fire fragility functions for an entire building to measure the probability of reaching a damage state given a fire scenario. Next, the paper proposes an ignition model to evaluate the probability of fire ignition after an earthquake. The ignition model together with fragility functions measure the probability of damage from fire following earthquake given an earthquake scenario.

INTRODUCTION

Community resilience to extreme events is an issue of increasing concern in our interconnected and urbanized societies. Meanwhile, cascading multi-hazard events, such as fires following an earthquake, can cause major social and economic losses in a community. The problem of evaluating the response of a community to an extreme event involves uncertainties at different levels, and therefore, is generally approached with probabilistic methodologies and risk management formulations. The process can be divided into three steps: evaluating the frequency of a hazard, measuring vulnerability of a community, and quantifying consequences of the event. The three steps together quantify the risk, which provides an estimate of potential losses.

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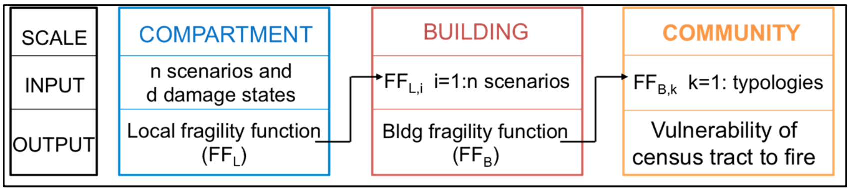


Figure 1. Overview of fire fragility curves for community resilience assessment

This work provides a framework to evaluate the response of buildings in a community to fire following earthquake. It focuses in particular on the development of fire fragility functions for an *entire building* and their application for measuring the resiliency of a community to fire following earthquake. The work is novel as the fragility functions are developed for the entire building, not just an element or component within the building. The paper includes two parts: (1) how to develop fire fragility functions; (2) how the fire fragility functions can be used in conjunction with an original fire ignition model to estimate the potential losses in a community from fire following earthquake. Fig. 1 provides an overview of the three scales involved (compartment, building, community of buildings) and how fragility functions fit within these scales. In this framework, local fire fragility functions at the compartment level are combined to obtain fragility functions for entire buildings. Different building topologies in a community demand different fragility functions. Such collection of fragilities can be used to evaluate vulnerability of a community.

FRAGILITY FUNCTIONS

In earthquake engineering, fragility functions are well established to quantify the structural damage due to an earthquake. A fragility function provides the probability of exceeding a damage state for a given intensity measure of a given hazard [1, 2]. The damage states (or limit states) are generally related to the structural performance level. Adopting a similar idea, fire fragility functions can be developed to measure the expected losses based on performance of a building structural system, rather than a single component. Then, different functions can be developed for buildings with different typologies (e.g. high-rise steel with moment resisting frame, low rise concrete with shear walls) to evaluate the vulnerability at the scale of a community.

One important parameter in defining a fragility function is the selected intensity measure for a given hazard. The intensity measure that is used to quantify the effect of an earthquake range from peak ground acceleration (*PGA*), pseudo displacement (S_d), permanent ground deformation (*PGD*) and etc., depending on the type of structure. In the case of a fire scenario, this paper proposes to take the average fire load (in MJ/m^2 of floor area) as the intensity measure, because: (i) the fire load is one of the main parameters affecting the intensity of a fire [3], (ii) the expected value of fire load changes depending the occupancy type, and (iii) it can be easily understood by the different stakeholders involved in fire safety. Therefore, in this paper, a fire fragility function refers to the probability of exceeding a damage state (e.g., column failure, excessive beam deflection, etc.) given the average fire load in a building. Fragility functions yield probabilities conditional to the occurrence of a hazard. In developing the fire fragility functions, it is assumed that a structurally significant fire, i.e. one that is able to endanger the structure, occurs in the building.

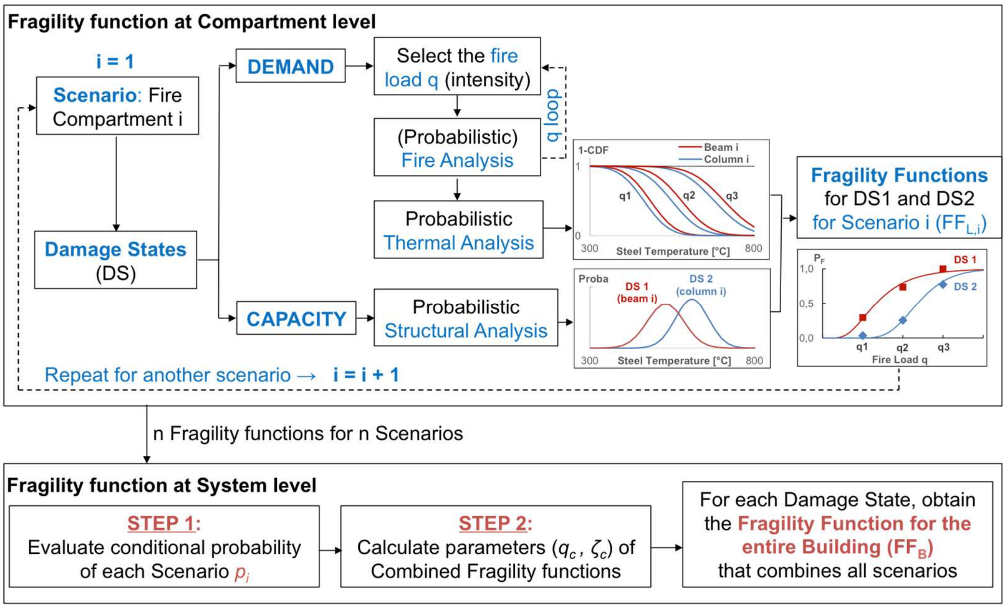


Figure 2. Development of fire fragility curves for a building

METHODOLOGY

This section provides an overview of the proposed methodology to construct a fire fragility function for a steel building. Developing a fragility function involves the probabilistic assessment of performance of the structure. Designing a member or a structure is traditionally based on comparing demand and capacity. In the context of fragility function, the demand is related to the temperature reached in the structure due to fire, while the capacity for a given damage state is associated to the exceedance of a certain temperature threshold in the sections of a structure, which is referred to as the critical temperature.

The proposed methodology incorporates uncertainties in demand (thermal analysis) and capacity analyses while taking advantage of the critical temperature. The concept of critical temperature is used to decouple the thermal analysis (demand side) from the capacity analysis. Thermal analysis provides the distribution of maximum possible temperatures reached in a section, while the capacity analysis yields a probability distribution function (*PDF*) for the critical temperature associated with a damage state. After both sets of analyses are completed, the results from demand and capacity can be combined and compared to find cases that experience the damage state (demand larger than capacity). The methodology is illustrated in Fig. 2.

Compartment level

For a given compartment in the structure, and focusing on a given damage state, the following procedure is followed to derive fire fragility functions at the compartment level:

(1) A value of fire load is selected and the temperature-time curve of fire is formulated using existing procedures (such as the Eurocode1 parametric fire curves).

(2) Heat transfer analysis is performed considering uncertainties in random variables such as thermal properties of steel or insulating materials, if applicable. The cumulative distribution function (*CDF*) of the steel temperature in the section is obtained.
(3) Structural analysis at elevated temperature is performed to find the critical temperature in the section. Uncertainties in mechanical properties of steel and the applied gravity loads are considered. The *PDF* of the critical temperature is obtained.
(4) The conditional probability of failure can now be computed using Eq. 1, by convolution of the PDF of capacity and the complementary *CDF* of demand [4].

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

In Eq. 1, $P_{F|H_{fi}}$ is the probability of reaching a damage state conditional to the occurrence of a fire H_{fi}, $F_{D|H_{fi}}(.)$ is the *CDF* of the demand relative to the fire H_{fi}, and $f_C(\cdot)$ is the *PDF* of capacity.

The above procedure is repeated for a range of fire load densities (q values) in the same compartment. Repeating the operation for each fire load yields to several outputs relating the fire load q (intensity measure) and the conditional probability of reaching the damage state as shown in Fig. 2. The fragility function is built by fitting a function to the obtained points, assuming a lognormal distribution:

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

where q is the fire load (MJ/m²) and $\Phi[\cdot]$ is the standardized normal distribution function. The two parameters c and ζ characterize the fragility function and are determined by the best fit to the data points. Finally, the same process is applied for deriving the fragility functions relative to the other damage states and other fire compartment locations. Further details about the proposed procedure are provided in [5].

Building level

In a multi-story building, different fire compartments can be defined. As a result, fire fragility functions should first be developed for each compartment, and then combined to derive a fire fragility function for the entire building. The building fragility function should represent the overall vulnerability of the building. Therefore, the procedure discussed at the compartment level should be repeated several times during the fragility analysis of a building, for varying scenarios (where the scenario *i* in Fig. 2 corresponds to a fire located in compartment *i*).

The method for combining fragility functions is adopted from [6] work where fragilities for similar structural attributes are combined. In this procedure, the combined fragility function is also a lognormal function (similar to Eq. 2). The two lognormal parameters for the combined function are calculated on the basis of the corresponding parameters for the individual fragility functions, taking into account the relative likelihood of each fire scenario (Fig. 2). Eq. 3 provides the mean of combined lognormal, where n is the number of "nominally identical but statistically different" fragility curves, c_i is the median associated with each individual fragility curve, and p_i is the conditional probability for a fire in compartment i, given that a fire occurs in the building. The standard deviation of the combined lognormal distribution, ζ_c, is calculated using Eq. 4, where $\boldsymbol{P}$ is the vector of the probabilities p_i, $\mathbf{Z}$ is the vector of the variances ζ_i^2 associated with each individual fragility function, $\boldsymbol{A}$ is the vector of the expected values (ln c_i), and $\boldsymbol{Q}$ is the matrix given by Eq. 5. The reader is referred to [5,6] for more comprehensive information about the procedure.

$$q_c = \prod_{i=1}^{n} c_i^{p_i} \tag{3}$$

$$\zeta_c^2 = \mathbf{P}^{\boldsymbol{T}}\mathbf{Z} + \mathbf{A}^{\boldsymbol{T}}\mathbf{QA} \tag{4}$$

$$\mathbf{Q} = \begin{bmatrix} p_1(1-p_1) & \cdots & -p_1 p_n \\ \vdots & \ddots & \vdots \\ -p_n p_1 & \cdots & p_n(1-p_n) \end{bmatrix} \tag{5}$$

In the proposed methodology, the probability values (p_i) are calculated using the formulation that is applied by Eurocode to develop the prescribed design values for fire load densities [7]. This formulation relates probability of having a severe fire to occupancy type, fire brigade, and active fire protections.

CASE STUDY

The proposed methodology, discussed above, is applied to a 9-story steel building prototype. The building is 45.72 m by 45.72 m in plan, with five bays at 9.144 m in each direction. The structure is composed of four moment resisting frames on the perimeter, and four interior gravity frames. 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 eight floors of 3.962 m high. 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. The beam sections consist of W21x44 on all floors except a W18x40 at the roof (9$^{\text{th}}$ floor). The column sections range from W14x43 at the 9$^{\text{th}}$ story to W14x109 at the first floor. Column splices are located at every two floors.

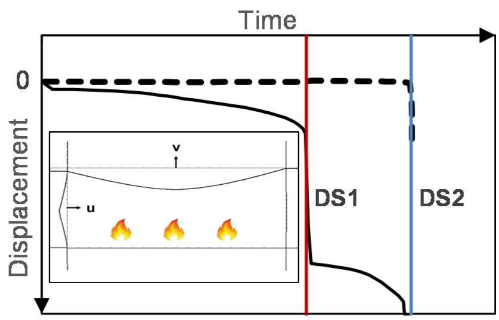
The procedure to develop a fire fragility function for the building is applied to the prototype building. In order to illustrate the procedure, two structural damage states are considered, one relative to the beams and one relative to the columns:

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

The uncertainties in both demand and capacity analyses are considered. On the demand side, the SFRM thickness is assumed to follow a lognormal distribution with a mean value equal to the nominal value of 39 mm plus 1.6 mm and a coefficient of variation of 0.2 [8]. The probabilistic model proposed by Elhami Khorasani et al. [9] is adopted for the SFRM conductivity. On the capacity side, randomness in the gravity loads and mechanical properties of steel is 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 [10]. The reduction in steel mechanical properties with temperature, are modeled using the probabilistic model from [9].

The thermal analyses are performed using the finite difference formula of EN 1993-1-2 Section 4.2.5.2 [11]. Monte Carlo Simulations are conducted using the Eurocode formula and varying the thermal properties of the insulation material (thickness and conductivity) [12]. On the capacity side, the building structure is modeled in the non-linear finite element software SAFIR [13] developed at University of Liege. SAFIR allows conducting a thermal analysis of the sections of the structural members, followed by structural analysis of the building at high temperature. The critical temperature is independent of the particular time-temperature evolution curve in the section. The concept of critical temperature, which was discussed in the previous section and illustrated in Fig. 2, is prescribed in Eurocode [11], and is validated for the specific structure under study. The reader is referred to [5] for details of the study. Therefore, the temperature evolution used as an input in the structural FE analysis, can be any time-temperature relationship. In this work, the ASTME119 fire with no thermal protection on the steel members is used as the input.

Based on the proposed methodology and the inputs, the combined fragility curves associated to the two damage states for the entire building are shown in Fig. 3. Assuming an average fire load of 600 MJ/m² in the building, the figure shows that the probability of exceeding the beam damage state (DS1) is 88% and the probability of exceeding the column damage state (DS2) is 15%. Therefore, the probability of exceeding the damage state in the beam (DS1) but without collapse of the column is 73% (0.88-0.15). 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. 12%.



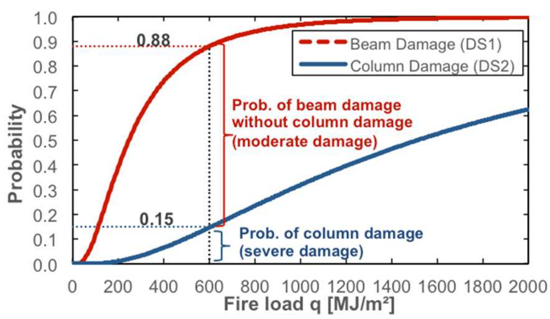


Figure 3. (a) Damage states related to beam and column, (b) Combined fragility curves for the prototype nine-story steel frame building

IGNITION MODEL

The paper so far discussed a methodology to quantify the vulnerability of a building given that a structurally significant fire occurs in the building. This section provides a model to evaluate the probability of ignition in a building after an earthquake. The two models together can be combined to evaluate the probability of damage due to fire given an earthquake scenario.

The proposed ignition model is developed based on historical ignition data, all of which led to structural ignition fires. The model is based on seven historical earthquake events, all of which occurred in California, U.S.A., between 1983 and 2014: 1983 Coalinga, 1984 Morgan Hill, 1986 North Palm Spring, 1987 Whittier Narrows, 1989 Loma Prieta, 1994 Northridge, and 2014 Napa. Compilation of the inventory of historical data is similar to the work completed by Hazus [14] but updated to include data from the recent Napa earthquake in 2014. Meanwhile, the proposed model takes a different approach than the existing FFE ignition model in Hazus and is based on a probabilistic approach.

The proposed model relates the probability of ignition to the peak ground acceleration *PGA* (as a measure of earthquake intensity), type of building material (number of wood buildings N_W, mobile homes N_{MH}, non combustible buildings N_{NC}), and the main features of the environment in which the buildings are located (the total square footage *SF* and the population density *PD*). The ignition model outputs probability of ignition at a census tract and at individual buildings (Eqs. 6 to 8). Fig. 4 shows the step-by-step procedure to use the ignition model. Eq. 6 uses the characteristics of the area under study and *PGA* to estimate probability of ignition at each census tract P_{Ig_tract}. Then, Eq. 7 uses the complement probability rule to back-calculate probability of ignition P_{Ig} in each building type from P_{Ig_tract}. Finally, Eq. 8 provides the expected number of ignitions in a collection of census tracts given the ignition probability in individual buildings. The model is validated against historical FFE events and shows good agreement with the historical data [15]. In addition, the proposed model has the advantage of providing the breakdown in the number of ignitions for different considered building types.

$$P_{Ig\_tract} = \frac{\exp(-6.755+8.463\times\mathrm{PGA}+98.4\times10^{-6}\times PD+152.3\times10^{-6}SF)}{1+\exp(-6.755+8.463\times\mathrm{PGA}+98.4\times10^{-6}\times PD+152.3\times10^{-6}SF)} \tag{6}$$

$$P_{Ig\_tract} = 1-[(1-0.471P_{Ig})|\mathrm{PGA}]^{N_W}\times[(1-1.0P_{Ig})|\mathrm{PGA}]^{N_{MH}}\times[(1-0.411P_{Ig})|\mathrm{PGA}]^{N_{NC}} \tag{7}$$

$$\text{No. of Ignitions} = \sum_{i=1}^{m}\left[N_W\times\left(0.471P_{Ig}\right)+N_{MH}\times\left(1.0P_{Ig}\right)+N_{NC}\times\left(0.411P_{Ig}\right)\right]_i \tag{8}$$

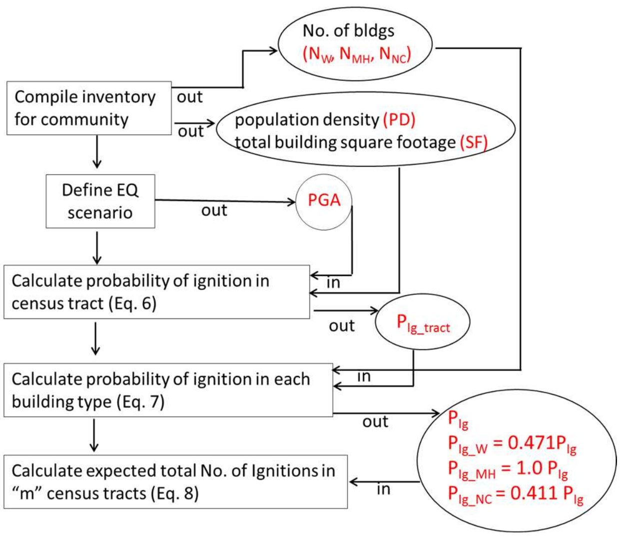


Figure 4. The step-by-step procedure for using the ignition model

CONCLUSION

This paper provided a methodology to develop fire fragility functions for buildings. The fragility functions are first derived at the compartment level, and then combined to obtain the fragility function for the entire building. The proposed framework employs the concept of critical temperature to decouple the thermal analysis (demand side) from the capacity analysis. The compartment fragility function is obtained by convolution of the PDF of capacity and the complementary *CDF* of demand.

The paper also proposed an ignition model to evaluate probability of fire ignition after an earthquake. The ignition model, combined with the fragility functions, can be implemented in a Geographic Information System (GIS) based risk assessment platform to evaluate social and economical losses in a region from fire following earthquake. The ignition model provides the probability of fire ignition while the fragility function measures the expected structural damage given a fire ignition. The two models together measure the probability of damage from fire following earthquake given an earthquake scenario.

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