



Severity Measures and Stripe Analysis for Probabilistic Structural Fire Engineering

Mayank Shrivastava* , Anthony K. Abu , Rajesh P. Dhakal, and Peter J. Moss, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch 8140, New Zealand

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Abstract. This paper presents modifications to the adoption of a Performance-Based Earthquake Engineering (PBEE) framework in Probabilistic Structural Fire Engineering. Potential Fire Severity Measures, which capture significant characteristics of fire scenarios, are investigated. A suitable Fire Severity Measure (FSM), which best relates fire hazard intensity with structural response, is identified by satisfying efficiency and sufficiency criteria as described by the PBEE framework. The study also implements a new analysis method called Fire Stripe Analysis (FSA) to obtain the relationship between FSM and the structural response. In order to obtain the annual rate of exceedance of damage and repair cost/time for an office building, an occurrence model and an attenuation model for office structure fires are generated for both Christchurch city and New Zealand. The process is demonstrated with the help of a case study performed for a steel–concrete composite beam. Structural response is recorded for the beam exposed to several fire profiles which are generated by varying fuel loads from 200 MJ/m² to 1000 MJ/m² and ventilation factors from 0.02 m^{1/2} to 0.08 m^{1/2}. FSA and dispersion curves of structural response are plotted for every fire severity measure. Cumulative incident radiation is found to be the most efficient and sufficient FSM. The mean annual rate of exceedance of given levels of fire severity and structural response are evaluated for both New Zealand and Christchurch city. It is found that Christchurch city has a 15% less probability of exceedance of the given fire severity level in comparison to the whole of New Zealand. The extension of this work would facilitate designers/insurers to evaluate the probability of damage or failure of a structure due to a probable fire hazard.

Keywords: Probabilistic engineering, Structural fire, Exceedance probability, Fire severity measure

1. Introduction

A paradigm shift in structural fire design from prescriptive design to performance-based design not only allows the designer to think beyond code provisions to assess compliance but also produces structures with quantifiable levels of safety that meet other design objectives beyond life safety. This paradigm shift results in

* Correspondence should be addressed to: Mayank Shrivastava, E-mail: mayank.shrivastava@pg.canterbury.ac.nz



the provision of only the required elements of construction—in some cases this results in reducing the amount of required fire protection, while keeping or improving the desired performance level [1, 2]. Performance-based design aims to ensure satisfactory structural performance, along with meeting general objectives of fire safety design. For any hazardous event, life safety is the foremost objective, whereas property protection and low repair cost/time are secondary objectives for stakeholders. Current structural fire design practice within a performance based design approach requires multiple fires which occur in different locations of the building, or different fires at the same location. Multiple analyses are performed for the probable fire scenarios to capture the thermal and the subsequent mechanical response of the structure. These fires are not guaranteed to occur in a building, therefore a more reliable approach is probabilistic methodology. A probabilistic assessment of structures accounts for the probability of different fire scenarios and provides predictions of structural response in terms of temperature, deflection, failure time, etc. It provides a measurable sense of reliability or risk of the structure for the multiple realistic fire scenarios which may occur in the structure. Several probabilistic approaches are available in structural fire engineering. These are: probabilistic risk analysis, reliability analysis and performance based structural fire engineering, based on the Pacific Earthquake Engineering Research (PEER) framework [3, 4].

In a probabilistic risk analysis (PRA), risk is a subjective matter and requires agreement between different stakeholders to set acceptable risk standards or follow socially acceptable risk. Risk due to various design alternatives are compared in PRA [5]. De Sanctis et al. [6] recommend a risk-based methodology based on Bayesian probability network. On the other hand, the same situation can be analysed considering the level of safety required or reliability, which is the core of reliability analysis. A pertinent methodology of measuring the reliability of a structure is to estimate the probability of failure. Probability of failure is not only a reliable indicator but also a valuable tool from a design point of view. The probability of failure can be calculated with the help of a First-order reliability method (FORM), second-order reliability method (SORM) or Monte Carlo approach. FORM underestimates the failure probability results but the observed error is not significant. The failure probability can be improved by second order approximation with SORM. On the other hand, the Monte Carlo approach is time-consuming but it can incorporate uncertainty at various levels of the framework [7–11].

A new approach is proposed to deal with member temperature estimation and protection thickness estimation for all possible scenarios in order to prevent structural members from exceeding their critical temperatures within the required period [12, 13]. The method calculates the required fire resistance based on time equivalence, which is known to have its flaws. The Monte Carlo approach is used for probabilistic investigation and risk/failure/reliability is calculated, whereas an accurate estimation of fire severity for a structure requires thermal and structural analysis.

An approach which not only designs structures for life safety with a linear framework but also aids in evaluating probable damage and losses is the perfor-

mance-based probabilistic design approach which is well established in earthquake engineering, and known as Performance-Based Earthquake Engineering (PBEE) [14].

The PBEE approach, developed by the Pacific Earthquake Engineering Research (PEER) center [3], is a probabilistic methodology for the assessment of the performance of a structure under seismic conditions. PBEE is characterized by four analysis stages—hazard analysis, structural analysis, damage analysis and loss analysis. Hazard analysis is performed to quantify the annual rate of exceeding a certain seismic shaking intensity which is usually quantified by an Intensity Measure (IM) such as peak ground acceleration or spectral acceleration. For a given hazard condition, structural analysis is performed to evaluate the structure's critical response. It is expressed by a variable called Engineering Demand Parameter (EDP), such as maximum inter-story drift ratios, maximum inelastic component deformations or strains, and peak floor acceleration. Using the response from the structural analysis, a damage assessment is conducted at the damage analysis stage and to deduce Damage Measures (DM). Lastly, loss analysis (in which information about repair cost or the repair time for the extent of damage predicted in the previous stage) is expressed by a Decision Variable (DV) [14–16].

Both earthquake and fire are low-probability and high-consequence hazards. The application of the Performance-Based Earthquake Engineering (PBEE) framework to structural fire engineering has been explored by a number of researchers [17–21]. The process of probabilistic structural fire engineering (PSFE) follows similar analysis stages except the hazard here is fire. The pinch variables at each stage (IM, EDP, DM and DV) are identified here as well in the context of fire engineering to establish the process. As the two hazards are similar (low probability-high consequence) in nature the PBEE framework has so far been directly applied in PSFE. However, research shows that there are problems with this direct application, as the quantification of the two hazards and their associated structural response are different [20, 22]. The hazards in seismic engineering occur outside the building and manifest themselves in motions of the entire building, necessitating structural response only at room temperature. In fire, the hazard may occur in one compartment within the building, and its detrimental effects require structural response in both thermal and mechanical terms. The temperature of the compartment is dependent on the surface linings. If surface linings absorb more heat, then the temperature rise within the compartment will be less severe. This temperature–time relationship may be obtained by simple or complex calculations. Simple calculations employ a unique thermal inertia of each lining while a complex model will monitor thermal exchanges between each lining and the compartment throughout the fire. Thus, the thermo-mechanical response of the building in a fire is more complex than the more straightforward mechanical analysis in earthquake engineering. As a result of these fundamental differences in the hazard and associated building response, it has been observed that although the broad framework of PBEE is still applicable to PSFE, it needs to be extended in detail to produce meaningful results for fire engineering design.

1.1. Probabilistic Structural fire Engineering

The first stage of PSFE is referred to as Fire hazard analysis by the authors in order to highlight the type of hazard for which the analysis is performed and also to differentiate it from PBEE. In earthquake engineering, the hazard happens outside the building and therefore its distinct amplitude (i.e. intensity) at the location of the building is very important because intensity reduces with distance, ground condition and magnitude of the earthquake. In fire the hazard happens inside the building. The severity of the fire is therefore affected by how much ventilation is available. Intensity of a fire typically refers to the temperature of the fire at a given time. An intense short duration fire has less effect on a protected steel beam or concrete beam than a less intense long duration fire. Therefore, it is not the “intensity” of the fire which affects the structure but its overall severity. Therefore, the authors propose to call it a Fire Severity Measure (FSM) instead of an Intensity Measure (IM). FSM apprehends the significant characteristics of the fire scenario which affect the response of the structural system. Examples of FSMs are maximum fire temperature, fire duration, area under the fire curve and cumulative radiant heat, among others. Furthermore, in PBEE, the structural response is calculated for a given hazard, i.e. PGA, by performing only (room-temperature) structural analysis whereas in PSFE, the hazard is represented by a temperature–time curve. Once you generate the hazard then a two-phase analysis is required to get the complete response of the structure. First is a thermal analysis which estimates the temperature history of the structure for the given fire profile. Secondly, a mechanical analysis is performed to evaluate the response of the thermally affected member to the applied loads. Keeping the same terminology, i.e. structural analysis, in PSFE does not clearly capture the inclusion of thermal effects on the structure. Therefore, the authors suggest the use of “response analysis” in order to clearly account for both thermal and structural analyses effects on the structure. Critical response parameters i.e. EDPs are identified at the response analysis stage. Some examples of structural response parameters (EDPs) are maximum displacement, maximum member temperature and maximum moment. The EDP needs to be correlated with a damage measure (DM), such as no damage, spalling, collapse, etc., which will be expressive of loss/cost. The adoption of the PBEE approach in PSFE is illustrated in Fig. 1.

PSFE follows the PEER center equation based on a total probability theorem (Eq. 1). In Eq. 1 $\lambda(\text{FSM})$ relates the fire severity measure to its mean annual frequency (MAF) of exceedance; $dG(\text{EDP}|\text{FSM})$, signifies the conditional probability of exceeding a specified EDP when the structure has been subjected to a defined FSM. The mean annual rate of exceedance (MARE) of given levels of EDP, i.e. EDP hazard curve $\lambda(\text{EDP})$, can be estimated from combining $dG(\text{EDP}|\text{FSM})$ with the mean annual frequency of FSM, $\lambda(\text{FSM})$. $G(\text{DM}|\text{EDP})$ is classified as a fragility function, which models the conditional probability of a damage measure given the magnitude of an engineering demand parameter. $\lambda(\text{DM})$, i.e. mean annual rate of exceeding a specified value of DM, can be estimated by integrating the EDP MAF term, $\lambda(\text{EDP})$, with the conditional probability $dG(\text{DM}|\text{EDP})$. The selection of EDP and DM should be such that conditional probabilities are independent of

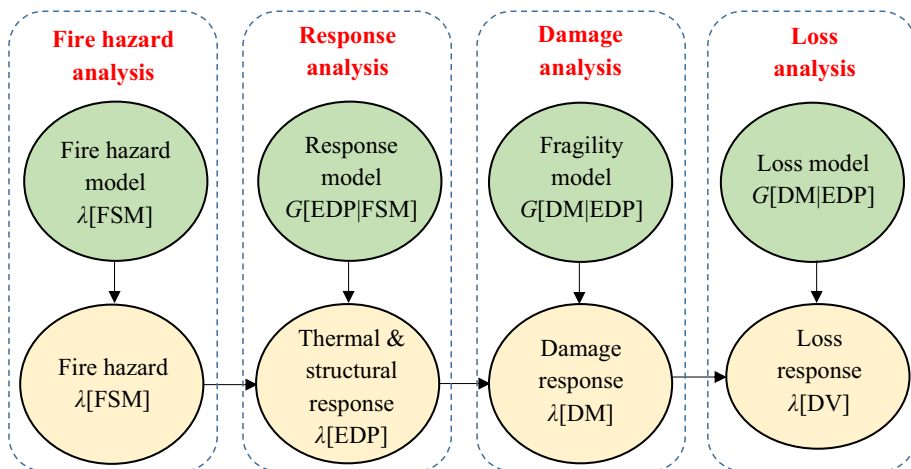


Figure 1. Probabilistic structural fire engineering framework.

each other and this conditional probability information should not propagate to the next level. Similarly, the mean annual rate of exceeding a specified value of DV, i.e. $\lambda(DV)$, can be obtained by integrating the conditional probability $dG(DV|DM)$ with DM MAF term $\lambda(DM)$. In PBEE, the mean annual value of a decision variable in the form of Expected Annual Loss (EAL) is derived by adding an additional integral to Eq. 1. A significant way of communicating vulnerability of a structure to stakeholders is in terms of dollars and EAL is capable of doing so [23].

$$\lambda(DV) = \iiint G(DV/DM)/dG(DM/EDP)/dG(EDP/FSM)/\lambda(FSM) \quad (1)$$

Equation 1 implies that PSFE has the capability to provide information on the annual rate of exceedance of damage and repair cost/time incurred to the structure during its lifetime. This requires an understanding of the probability calculation in two stages. The first is an occurrence stage, which provides information on the annual probability of occurrence of a fire hazard. The second is an attenuation stage, which indicates the probability of exceedance. The amalgamation of both probability stages produces the mean annual rate of exceeding any particular value of a given variable (e.g. FSM, damage state, or structural response) at any location (e.g. city or country).

Identifying a fire by a single characteristic may completely ignore other aspects of the fire. For example a “two-hour long fire” does not give any information about the peak temperature of the fire or how quickly it decays. Furthermore, if the severity of fire hazard has to be represented by a single parameter (i.e. by an FSM) then it needs to be ascertained that the parameter adequately accounts for the effects of the fire on the structure more efficiently than any other parameter. In parallel to PBEE, a suitable FSM is identified by several selection criteria such

as efficiency [24], sufficiency [25], scaling robustness [26], hazard computability [27] and predictability [28, 29]. The application of all these properties to select a suitable FSM leads to an accurate prediction of the structural response.

So far, only the efficiency criteria has been investigated in PSFE. Therefore, it is important to introduce more selection criteria in PSFE to produce suitable FSMs. Four FSMs available in literature are maximum fire temperature, fire duration, area under the time–temperature curve and cumulative radiant heat. Maximum steel temperature is also used as an FSM by Hamilton [18]. Principally, maximum steel temperature is a thermal response of the structure, which is a good indicator of steel structural performance in fire. Maximum steel temperature can be argued to be unsuitable as a fire severity measure (FSM), as FSM candidates need to be independent of the structure. As such they should be evaluated from the fire hazard curves or parameters contributing to the temperature–time curve. This indicates that very limited work has been done in investigating potential FSMs [17]. Previous works to identify efficient FSMs have been limited in the numbers of potential FSMs that were investigated. To obtain the response of any structure exposed to fire, thermal and structural analyses are required. However, one of the key aims of PSFE is to reduce the number of thermo-mechanical response analyses that have to be performed. Thus following guidance from PBEE, “Incremental Fire Analysis” (IFA) was proposed by Moss et al. [20]. IFA is an analysis method available in the literature for measuring the relationship between EDP and FSM. IFA is based on two earthquake engineering analysis methods Incremental Dynamic Analysis (IDA) [30] and Multi-Stripe Analysis (MSA) [31–33]. In IFA, a few fire profiles are generated by considering a limited range of fuel load and ventilation factor. Since IFA requires several fire profiles, the generated fire profiles are scaled to a wide range of FSM levels. However, the unrestrained scaling of the profiles produces unrealistic fire scenarios whose attributes are not representative of physical fires, such as area under the curve, fire duration, heat flux and time to reach peak temperature. Therefore, it is required to explore solutions to avoid intense scaling of fire profiles. For this reason, this paper describes a technique known as Fire Stripe Analysis (FSA) in Sect. 3.

Based on the above research gaps, the objectives of the paper are outlined below:

1. Investigate a wider set of potential FSMs and identify a suitable FSM amongst them which more suitably characterises fire severity.
2. Introduce another FSM selection criteria of performance-based earthquake engineering, i.e. sufficiency, for the first time in probabilistic structural fire engineering to identify suitable FSM.
3. Introduce and implement a new analysis method called “Fire Stripe Analysis” in PSFE to relate FSM and structural response, to avoid extensive scaling.
4. The paper further evaluates the mean annual rate of exceeding a given level of fire hazard and structural response for Christchurch city and New Zealand.

2. Fire Severity Measures (FSMs)

The severity of any hazardous event can be represented by many parameters. It is important to choose an ideal FSM. Four FSMs identified so far in literature are maximum fire temperature, fire duration, area under the time–temperature curve and cumulative radiant heat [20, 22]. In order to fully understand which FSM represents the fire severity most precisely, this paper investigates additional FSMs as discussed below. The illustration of FSMs is presented in Figs. 2 and 3 with the help of a temperature–time profile generated using the formulation of the Eurocode parametric fire [34].

Maximum Fire Temperature (MFT): this is the maximum temperature of the fire inside the compartment. **Time to maximum fire temperature (TMFT):** this is the time at which the temperature in a compartment reaches its maximum value. It represents the duration of the heating phase of a fire (Refer Fig. 2). **Fire Duration (FD):** the fire duration is the total time taken by the fire in the heating phase as well as to cool down to ambient temperature. It reflects the total exposure time of a structure to a fire event (Refer Fig. 2). **Area under the fire temperature–time curve (AUC):** this provides some information about the potential heat energy of a fire, both convective and radiative, which a structure is exposed to. It is calculated as the area under the time–temperature curve (refer Fig. 2), above a 20°C reference temperature. Unfortunately, this product does not have a physical meaning, but it is indicative of the effect the shape of the temperature–time profile has on the structural response. **Cumulative Incident Radiation (CIR):** Cumulative incident radiation is the total incident radiant heat flux to which a structure is exposed [34]. CIR is calculated as the area under the incident radiant heat flux–time curve. It represents the amount of radiant heat that the structure is exposed to during a particular fire. Heat transfer from a fire to the surface of a member occurs in two parts—convective and radiative. In general fire engineering terms the radiative and convective parts of the fire will be at two different temperatures. The cumulative incident radiation will only therefore consist of the radiative contribution of the fire. For spaces that are assumed to have uniform temperatures in the entire compartment the radiative and convective temperatures can be assumed

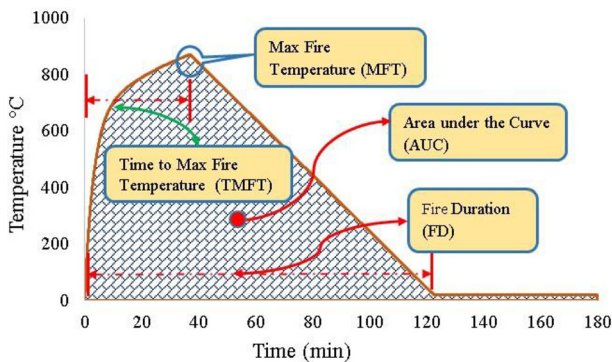


Figure 2. Time-temperature profiles.

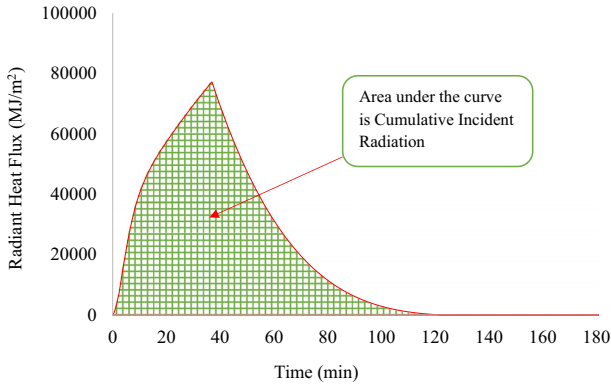


Figure 3. Radiant heat Flux versus time.

to be equal to the gas temperature. The radiation part of fire is calculated from Eq. 2.

$$h_{rad} = \phi \cdot \varepsilon \cdot \sigma \cdot (\theta_r + 273)^4 \quad (2)$$

h_{rad} (W/m^2) is the radiant heat flux at time t , ϕ is the configuration factor, θ_r is the radiation temperature in $^{\circ}\text{C}$ (here it is the gas temperature as the member is fully engulfed in fire) and ε and σ represent the total emissivity and the Stephan Boltzmann constant ($5.67 \times 10^{-8} \text{ W}/\text{m}^2\text{K}^4$) respectively.

Nyman's approach of using cumulative radiant heat energy is based on equivalent areas under heat flux vs time curves of a real fire and a standard fire to predict the failure time of dry wall systems [35, 36]. Whereas in the paper the authors are using cumulative radiant heat as a representation of a post-flashover fire which is independent of the type of the structure, construction material and its use. Therefore, the application of the cumulative radiant heat concept is not limited to non-load bearing functions of dry wall systems only [36].

Fire load density per floor area (q_{fd}): this is the fire load density related to the floor area of the compartment. The fuel loads of the compartment show the amount of potential heat which can be released during the fire event [34]. Fire load density per internal surface area (q_{td}): this is related to the total internal surface area of the compartment [34].

Precise quantification of FSM is necessary to accurately predict the structural response. The detailed explanation of sufficiency and efficiency criteria for the identification of a suitable FSM is presented here. Implementation of other selection criteria into PSFE is part of ongoing work by the authors.

2.1. Efficiency and Sufficiency

An efficient FSM reduces the number of analyses required for confidence in the predicted structural response and produces less variation of the EDP for a given FSM value. The efficiency of an FSM can be evaluated by computing the disper-

sion of the structural response. An efficient FSM would result in a lower dispersion.

For the aforementioned range of FSMs to be sufficient to represent the fire severity no additional information is required to fully quantify the FSM. An FSM is sufficient if the correlation of structural fire response (EDP) shows no trend with the parameters which defines the fire such as fuel load and opening factor. If the estimated structural fire response is based on the response to a suite of fire profiles, then the distribution of key elements of post-flashover fire of the selected fire profiles may not represent the distribution of fire scenarios which may occur in the compartment at a later date in the future. Therefore, it is desirable to have no trend in the correlation of EDPs with key elements of post-flashover fire. This selection criteria is commonly adopted in PBEE and requires the seismic response EDP to show no trend with earthquake magnitude (M) and source-to-site distance (R). A sufficient FSM ensures that any set of fire profiles selected for analysis of a structure will produce a similar probability of exceeding a specified EDP subjected to a defined FSM, i.e. $G(\text{EDP}|\text{FSM})$. In case of an insufficient FSM, the estimation of $G(\text{EDP}|\text{FSM})$ will depend on the selection of fire profiles to some extent, which then alters the estimation of the performance of the structure for the fire condition.

After the identification of suitable FSM, the next step in the process is the response analysis. The response analysis involves the estimation of a structure's response to a given fire hazard. Traditionally, deflection has been the most commonly accepted indicator of the damage [37–40]. Therefore, the authors chose to use maximum deflection as the response parameter in this study. Also, as steel temperature governs the load bearing capacity of the steel member, it is also important to select a suitable parameter to track the failure of steel elements. In this study, maximum vertical displacement (structural response of a structure) and maximum steel temperature (thermal response of a structure) are considered as EDPs, which closely define the significant behaviour of the structure and help to quantify the damage in the structure. The relationship between structural response and fire hazard is established by the analysis method. The authors proposed a new method called Fire Stripe Analysis (FSA) which overcomes the limitations of IFA.

3. Fire stripe Analysis (FSA) Overview

The general procedure demonstrating the methodology and application of FSA to establish a relationship between FSM and EDP is illustrated through a flowchart as presented in Fig. 4. Here, FSA is proposed without extensive scaling of a handful of temperature–time profiles. Instead, a suite of fires is generated based on the probabilistic distribution of key elements of post-flashover fire such as fuel load, ventilation, room geometry and surface lining.

The probabilistic distribution of these key elements generates several input values which can be used directly in a fire model (e.g. Eurocode parametric fire model) to produce temperature–time profiles, from which the various FSM values are derived. For example, maximum fire temperature (MFT) is calculated from

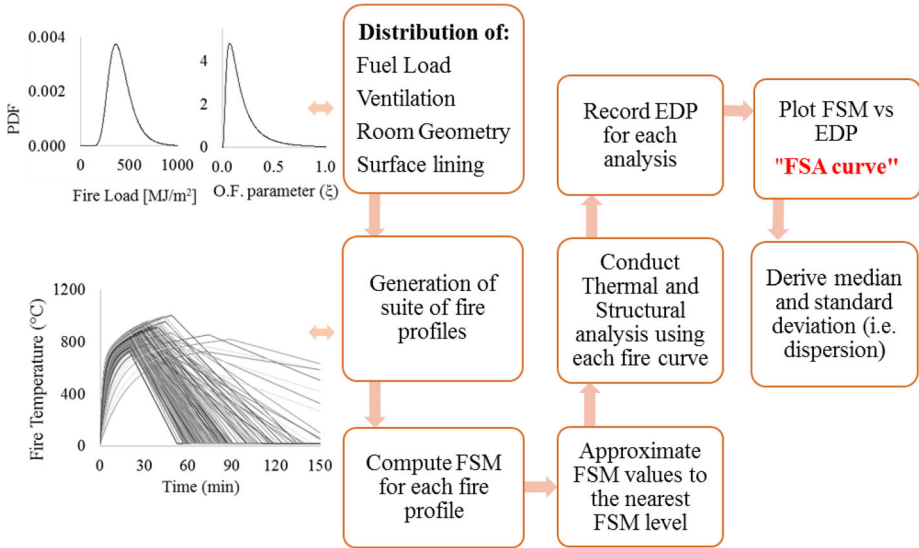


Figure 4. Fire stripe analysis (FSA) in probabilistic structural fire engineering.

each profile and recorded separately. This collection of maximum fire temperatures from several fire profiles provides an estimation of the probability of exceedance of a given maximum temperature in a compartment $[P(T_{max})]$. The mean annual rate of exceedance (MARE), i.e. Φ , of the maximum temperature in a compartment is found by the product of the probability of exceedance i.e. $P(T_{max})$ and the probability of a structure fire per year (r_{fi}). This paper evaluates MARE of the suitable FSM for Christchurch and New Zealand in Sect. 5.

$$\Phi_{(\text{maximum temperature})} = P(T_{max}) \cdot r_{fi} \tag{3}$$

Figure 5 shows fire profiles to illustrate the concept of FSA considering MFT as FSM. In FSA, FSM levels are carefully chosen to cover a wide range of severity of fire which apprehends the significant response of the structure, hence signifies damage. In the absence of factual fire severity levels, here fires are categorised in bands having equal interval of severity measures. A band of 50°C with a FSM level as 725°C is considered in Fig. 5. Fire profiles having maximum temperature ranging between 700°C to 750°C are scaled to FSM level of 725°C. This restricted scaling of fires within the narrow bands is considered reasonable, as the amount of scaling is very small, and therefore it does not affect the shape and properties of the fire curve. Fire-1 has MFT = 719°C and Fire-2 has MFT = 728°C, which are approximated to reach MFT of 725°C as illustrated in Fig. 5.

Thermal and structural analyses are then performed for these modified fire profiles and a structural response parameter i.e. EDP is recorded for each analysis. Each fire profile (or each FSM value) produces one EDP value, which is plotted

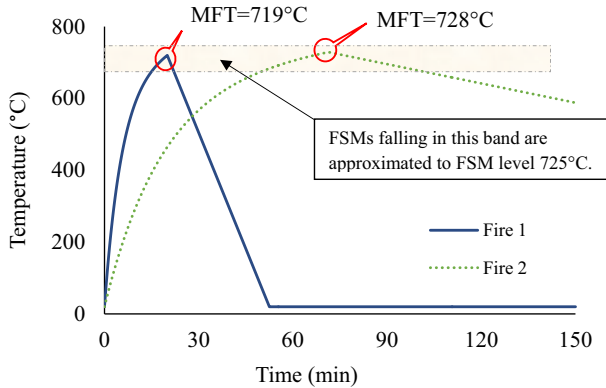


Figure 5. Fire profiles indicating FSM values and FSM levels.

as a point on the FSM-EDP graph. A group of EDP values is collected at each FSM level as shown in Fig. 6a; median and dispersion (assuming lognormal distribution) of the EDPs are estimated at each FSM level. A line joining the medians of EDPs at each FSM level is called the median FSA curve. The dispersion at every FSM level is recorded and plotted separately (Fig. 6b). This process is classified as FSA. The implementation of FSA is demonstrated with the help of an example in Sects. 5 and 6.

The fire hazard analysis should be able to describe the spatial variability of the fire scenario, considering incoherent key elements of post-flashover fire. There are various elements that govern the development of fire. These include fuel load, ventilation, geometry and surface lining of the compartment. For a given fire scenario the numerical interpretation of these elements involves large uncertainty. An efficient way to account for this uncertainty is to use a probabilistic approach. Hence, a probabilistic approach is used herein to generate several fire profiles using random values of fuel load and ventilation factor.

4. Key Elements of Post-Flashover Fire

The collapse of structures occurs when fires are severe. Various elements play an important role in the development of severe fires. These include fire-fighting measures, the effect of active control measures, fuel load, ventilation, the occupancy type and size of the compartment [41].

Fires vary depending on room sizes, their layout, lining materials and the amount of combustible material in the room. The architectural drawings of a building may help to estimate the values of the room geometry, ventilation and surface lining. Therefore, most researchers treat them as deterministic [8, 9, 42]. Although construction materials are fixed there is some variation in fuel load and ventilation conditions. The Eurocode parametric fire model offers an avenue where variations in fuel load and ventilation conditions could be modelled. The Eurocode suggests a Gumbel distribution for fuel load in an office building with

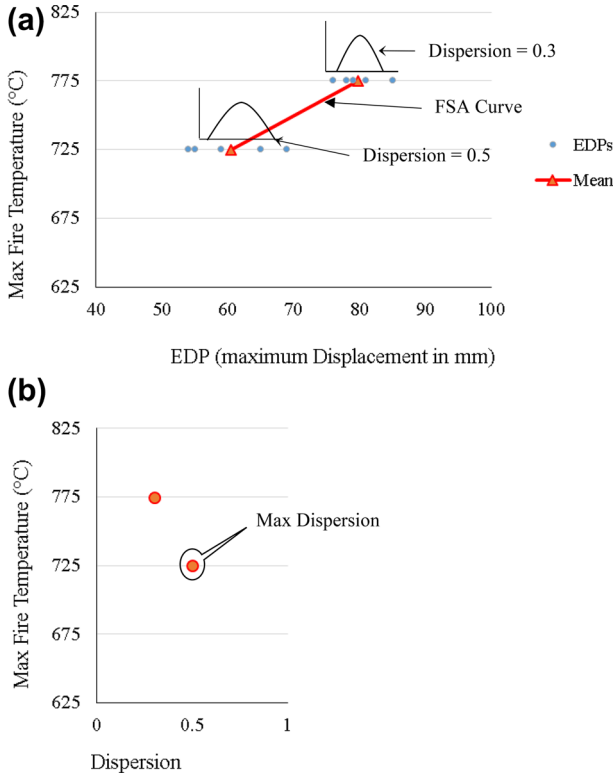


Figure 6. FSA curve and dispersion curve showing the FSM-EDP relation with dispersion at each FSM level. (a) Fire stripe analysis plot representing median and dispersion at each FSM level. (b) Dispersion plot indicating dispersion at each FSM level.

an average value of 420 MJ/m^2 as shown in Fig. 4. This value and associated distribution can be altered. The average value is based on surveys mentioned in Thomas [43], which might not reflect present-day conditions. The maximum ventilation of the compartment can be easily calculated from building architectural information but the amount of ventilation available during the fire is uncertain and it depends on glazing failure during the fire. Equation 4 has been suggested by the Joint Committee on Structural Safety (JCSS) [44] code to account for the variation in ventilation opening during the fire. This equation suggests a lognormal distribution function for the opening factor. Here, F_{vmax} is the maximum ventilation factor of the compartment, F_v is the opening factor which varies with a factor ζ which has a mean value of 0.2 and a standard deviation of 0.2. The distribution is truncated at 1 in order to keep F_v positive. The probability density functions of the fuel load and opening factor are shown in Fig. 4.

$$F_v = F_{vmax} \cdot (1 - \zeta) \quad (4)$$

With the help of the mean values and distribution functions of fuel load and opening factor, a range of parametric fires are generated. The generation of time–temperature profiles requires the knowledge of the geometry of the compartment which is discussed in the next section.

Here randomness in only two elements of post-flashover fire are considered which can also be termed as aleatory uncertainty. There are other sources of uncertainty, such as randomness in surface lining, room geometry, fire models, structural models, thermal analysis approach, material uncertainty and many more. All these uncertainties have an impact on the evaluation of the probable structural response. But for the purpose of enabling the probabilistic analysis and to focus on the response of the structure variation in only two parameters is considered in this paper.

5. Structure Considered

An office building constructed in New Zealand in 1988 is considered for the purpose of this research. Typical floor plan details are shown in Fig. 7. It is a composite structure with steel beams and reinforced concrete slabs. The beam size is 610UB101 having full composite action with a 120 mm thick concrete slab (65 mm continuous depth and 55 mm decking height). The beam is located at the centre of the plan. Similar work can be performed for edge beams in future. The reinforcing steel used in the slab is A193 mesh which is located at the centre of the continuous portion of the slab. The beam and slab are exposed to various time–temperature profiles from three sides [45–47]. The gravity beam is modeled in the analysis using the finite element software Vulcan [48, 49].

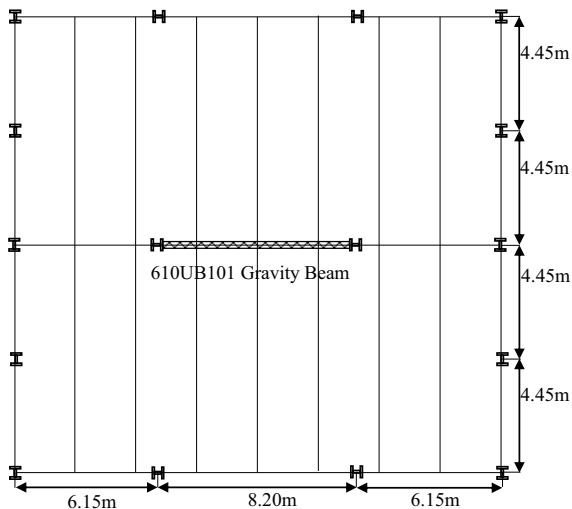


Figure 7. Floor plan of the building.

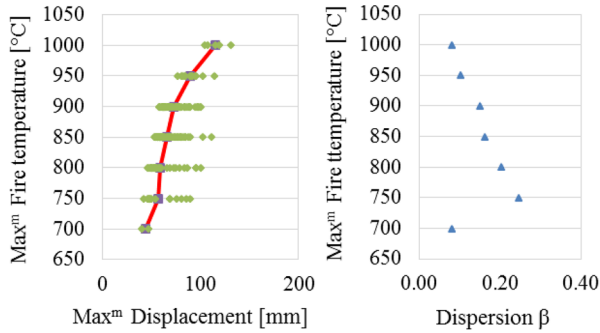
6. Modelling Details

The compartment dimensions are 18 m by 20.5 m and the height is 3 m. Maximum ventilation area is taken as 15% (to cover a range of ventilation and fuel controlled fires) of the floor area (54.74 m²) with average window height of 2 m. The maximum ventilation area is based on covering the range of fire profiles from short hot fires to long cool fires. The walls and roof of the building are assumed to be made of normal weight concrete. The gravity load, a combination of dead and live loads, is used in the analysis as a uniformly distributed load along the length of the beam. Similar to the loading conditions of Stevenson [45], the most adverse fire design load combination of 50 kN/m is considered in this study. A pin–pin support condition was used for modeling the composite beam. This was chosen based on a preliminary study by the authors which revealed that the pin–pin condition produces the worst structural response. Both protected and unprotected beam cases are investigated for the effect of fire protection on the selection of the efficient FSM. Fire protection thickness of 16 mm is assumed based on the design of the beam for 60-min fire rating using spray protection of 300 kg/m³ density, specific heat capacity of 1050 J/kgK and thermal conductivity of 0.15 W/mK. As mentioned in Sect. 4, randomness in fuel load and ventilation factor is considered. 200 fire records (i.e. temperature–time profiles) are produced with the help of Monte Carlo simulation, which uses distributions of fuel load and ventilation as shown in Fig. 4. The records cover a wide range of fire from short-hot fires to long cool fires.

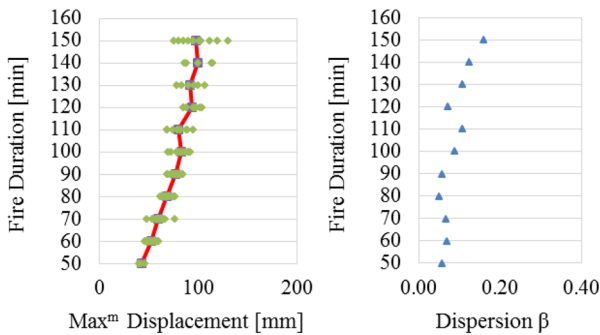
As can be seen in the time–temperature fire profiles, the maximum temperature of the fire ranges between 600°C and 1050°C. This is because of the low average value of fuel load provided by the Eurocode. Also, very limited short-hot fires and long-cool fires are observed in Fig. 4 (Temperature–time profiles) because of the use of the distribution of available ventilation which generates a limited range of opening factors together with a low average fuel load. Suites of fire profiles have been generated by many researchers considering similar variation in fuel load and the ventilation but there are differences in geometry of the compartment.

6.1. Fire Stripe Analysis

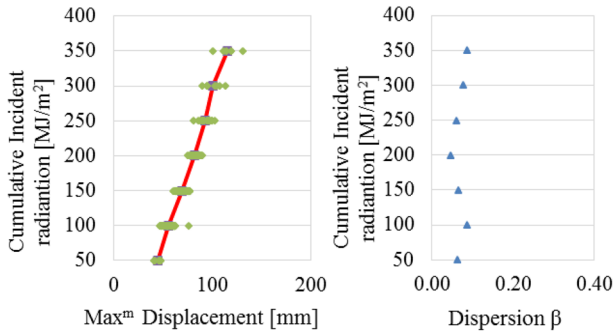
The temperature–time curves are used as input for the thermal analysis of the composite beam exposed on three sides. Subsequent to the thermal analysis, structural analysis was performed and maximum values of both vertical displacement and steel temperature from each analysis were recorded as EDPs. Figure 8 shows the graphs of three FSMs against the maximum displacement of the protected composite beam and Fig. 9 shows the graphs of three FSMs against the maximum steel temperature. In Fig. 8a, maximum fire temperatures are recorded for corresponding EDPs. As observed in the graph FSM values are also grouped in bands and approximated to the nearest FSM level, such as 700°C or 750°C. The fire profiles are then used in the fire analysis and maximum displacement is recorded during the analysis. The process is repeated with each fire profile, noting EDP values. The EDP values are then plotted for each FSM value. Since many fire profiles are approximated within restricted bands to any one FSM value, many EDP



(a)



(b)



(c)

Figure 8. FSA curve (left) and dispersion curve (right) showing the FSM-EDP (maximum displacement) relation with dispersion at each FSM level to compare the efficiency of FSMs. (a) Maximum fire temperature—Maximum displacement FSA curve (left) and dispersion curve (right). (b) Fire duration—Maximum displacement FSA curve (left) and dispersion curve (right). (c) Cumulative incident radiation—Maximum displacement FSA curve (left) and dispersion curve (right).

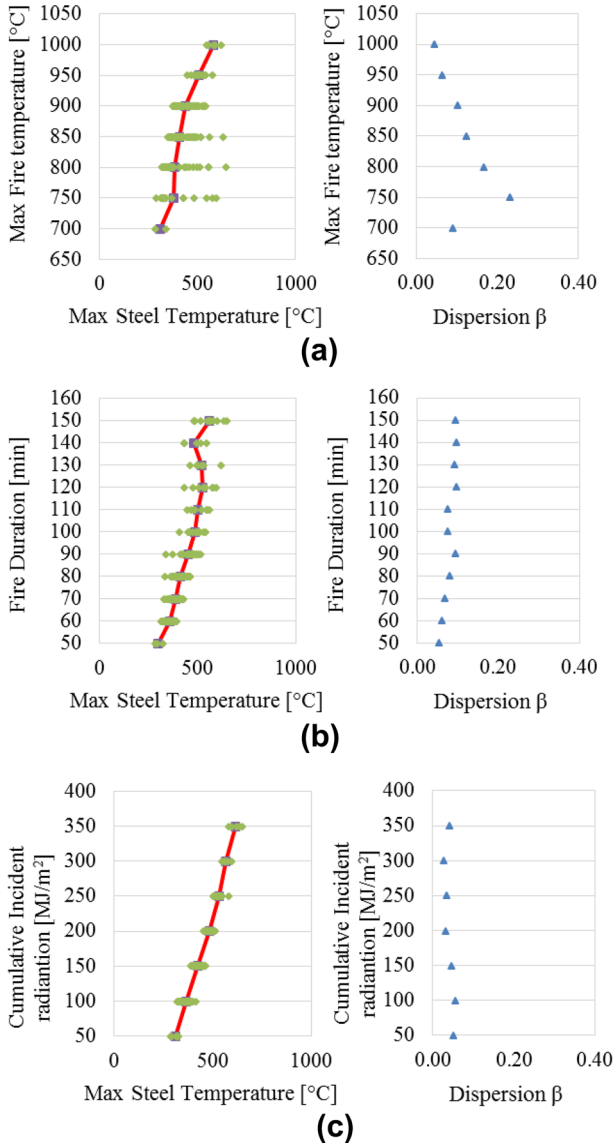


Figure 9. FSA curve (left) and dispersion curve (right) showing the FSM-EDP relation with Maximum steel temperature at each FSM level to compare the efficiency of FSMs. (a) Maximum fire temperature—Maximum steel temperature FSA curve (left) and dispersion curve (right). (b) Fire duration—Maximum steel temperature FSA curve (left) and dispersion curve (right). (c) Cumulative incident radiation—Maximum steel temperature FSA curve (left) and dispersion curve (right).

values get collected at each level. The median and dispersion are calculated at every level of FSM. Dispersion of each level is recorded separately in the dispersion curve (Fig. 8a [right]) and maximum dispersion of each FSM-EDP combination is recorded. The process is repeated for each FSM-EDP combination.

7. Results and Discussion of FSA

In Figs. 8 and 9, the FSA curve (on the left of each set of results) and the dispersion curve show the variation of EDP at each FSM level (on the right of each set of results). The dark line shows a general trend in the variation of EDP as the FSM changes by joining the median of EDPs at each FSM level.

7.1. Efficiency

Firstly, the efficiency of FSM candidates is evaluated by comparing the maximum dispersions from each FSM-EDP dispersion curve, as recorded and summarized in Fig. 10. The results of these analyses show that for a protected composite beam CIR is the most efficient FSM for both maximum vertical displacement and MST. This study is also performed on an unprotected composite beam and for that q_{td} and q_{fd} are found to be the efficient FSMs for maximum vertical displacement; MFT is found to be an efficient FSM for maximum steel temperature (MST). It is appropriate to find MFT as an efficient FSM for MST for an unprotected beam because the member temperature profile closely follows the fire temperature. FSA and dispersion curves for the unprotected beam analysis are not shown due to space limitations. Figure 10 demonstrates that efficient FSMs produce considerably lower dispersion for the EDP than the other potential FSMs.

7.2. Sufficiency

As previously mentioned, FSM should be sufficient with respect to fuel load and opening factor for a finite number of fire profiles. A sufficient FSM should not

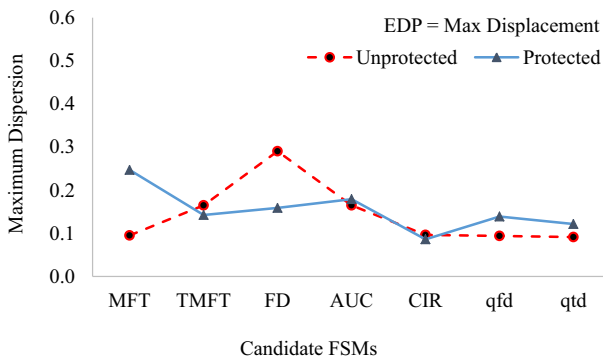


Figure 10. Efficiency comparison of FSMs for maximum displacement.

distinguish between a short duration high-temperature fire and a long duration low-temperature fire, if they produced the same structural response. The sufficiency of an FSM is evaluated by the extent to which the residuals of EDP, the difference in the predicted value and the actual value of EDP, shows no trend in the correlation with fuel load and opening factor. Here, residuals are obtained from the FSA curve, as the difference of mean FSA value at any FSM level to the actual EDP value at that level. Regression analysis is performed between the attained residuals of EDP and the corresponding fuel load or opening factor. No observed trend in the regression line of the residuals with respect to fire development parameters indicates the sufficiency of an FSM. Numerically, sufficiency is quantified by determining the p value, which is defined as the probability that the slope of the regression line is equal to zero. If the p value is less than 0.05, it provides enough evidence to reject the null hypothesis i.e. that the slope of the regression line is zero.

For illustration, the regressions of residuals of maximum displacement, of the protected composite beam, for the MFT, CIR and TMFT, with respect to FLED and OF are shown in Fig. 11 and recorded in Table 1. It can be seen from the figures that for CIR and FD, the p value is greater than 0.05. Though all FSMs produce p values less than 0.05 with respect to FLED, FD and CIR visibly show no trend in the regression line and have comparatively higher p value than others. Similarly, for MST as an EDP, TMFT produced a p value of 0.2 for opening factor, and CIR produced 0.13 for FLED as shown in Table 1.

The above calculation process indicates that CIR is the most efficient FSM, and CIR and FD are the most sufficient FSM with respect to opening factor for the evaluation of maximum displacement. CIR is also sufficient with respect to FLED for estimating MST. The prediction of efficient and sufficient FSM leads to the better estimation of maximum vertical displacement and maximum steel temperature. The identified suitable FSMs in the study are applicable to a composite structure, though results from the other research indicate that these are applicable for general structures too.

8. Probability of Occurrence of Fire

For the purpose of the present paper information is sought on the probability of structural fires in Christchurch and New Zealand per year. Statistics of fires in New Zealand is obtained from a report “Emergency incident statistics” by the New Zealand Fire Service [50]. It covers a range of fires which required the intervention of the fire service to extinguish. All small fires which could self-extinguish or were extinguished by the building occupants were not included in the statistical data, as they were no threat to the structure.

The report by NZFS classifies fires as structure fires (which occur in buildings), vegetation, mobile properties, and so on. Structure fires have damage classification in terms of property saved, such as flame damage, smoke damage and water damage, roughly from 1% to 100%. There is no damage threshold mentioned in the report to classify the damage level of the structure but there is some information

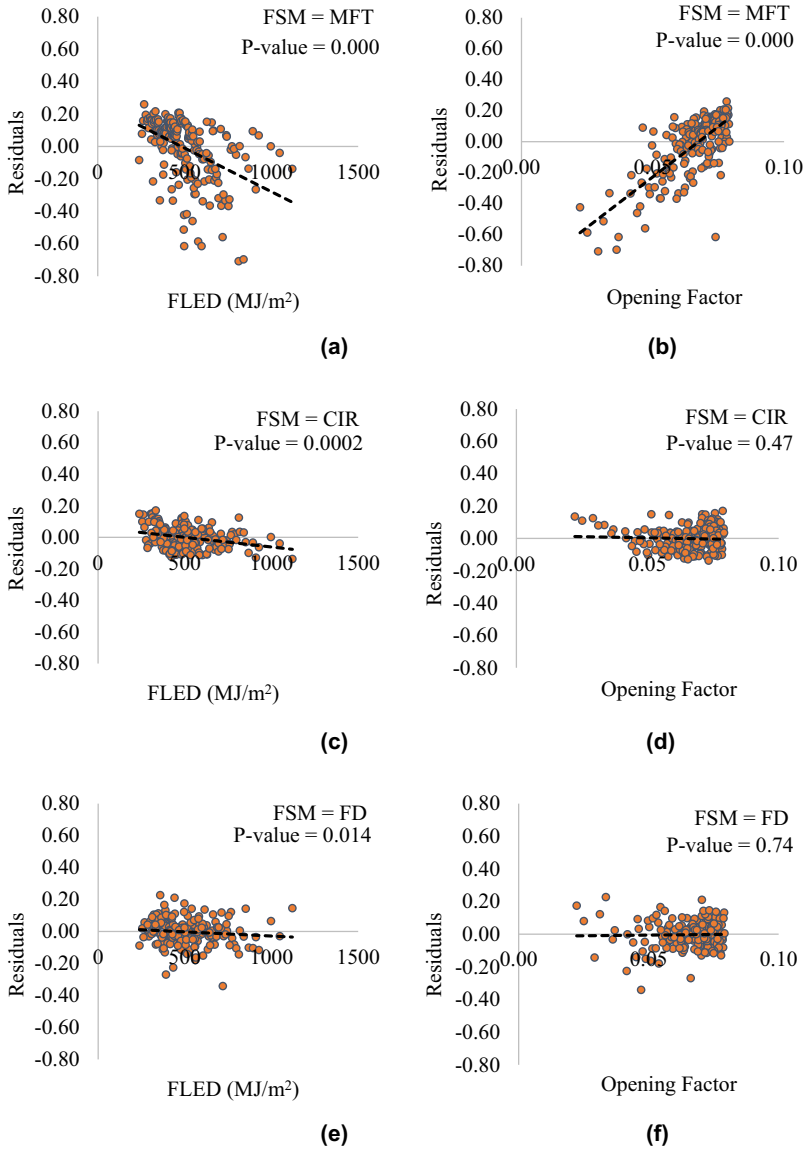


Figure 11. MFT sufficiency with respect to (a) Fuel load and (b) Opening factor in predicting maximum displacement of protected composite beam; CIR sufficiency with respect to (c) Fuel load and (d) Opening Factor in predicting maximum displacement of protected composite beam; FD sufficiency with respect to (e) Fuel load and (f) Opening factor in predicting maximum displacement of protected composite beam.

Table 1
Sufficiency Comparison Using p value for FSMs

	EDP = Max displacement		EDP = MST	
	FLED	Opening factor	FLED	Opening factor
MFT	0.00	0.00	0.00	0.00
TMFT	0.00	0.00	0.00	0.20
FD	0.01	0.74	0.00	0.00
AUC	0.00	0.00	0.00	0.00
CIR	0.00	0.47	0.13	0.00

about the percentage of property saved. Therefore, any fire which produces damage to a structure, flame damage or smoke damage, is considered under the “Structure Fire” classification. Fires which did not reach flashover and died out before affecting the structure are considered under the classification of “structure fires with no damage”. The structure may also be of any type i.e. residential or commercial. Fire statistics from the NZFS report is shown in Figs. 12 and 13. Looking at Table 2, the total number of fire calls recorded in 2012 were 21,946 in NZ and 4524 in Christchurch, out of which the total number of structure fires (i.e. fires in structures/buildings) in New Zealand are 5434, 116 in Christchurch CBD (Central Business District) and 359 in the wider Christchurch area. The underlined values were unknown and calculated based on the population ratio as outlined below.

As mentioned previously, the focus of this research is on office buildings. According to the NZFS report, the number of office fires in New Zealand was 180 in 2012/13 but was unknown for Christchurch city. Therefore, the percentage of an office structure fire in New Zealand is estimated to be 3.31% ($180/5434 \times 100 = 3.31\%$) per year. Since the NZFS report does not provide the number of structure fires for various occupancies for different cities, the same percentage of office structure fire for Christchurch city as of New Zealand could be assumed. This provides the number of office structure fires for Christchurch as 12 (3.31% of 359). Also, the number of commercial premises in Christchurch city area is 1740 [51, 52] assuming them linked to the full Christchurch population. Based on the above calculation the probability of office structure fire per year (r_{fi}) is given by:

$$r_{fi} = \frac{\text{No. of office structure fire in a year}}{\text{Total number of office buildings}} \quad (5)$$

The above equation gives the r_{fi} for Christchurch as 0.007 (= 12/1740).

In order to have a similar calculation for the New Zealand as a whole, Eq. 5 parameters for New Zealand needs to be calculated. The number of office structure fires in New Zealand is 180. Since the total number of office building data is not available for New Zealand, therefore extrapolation of numbers for an office

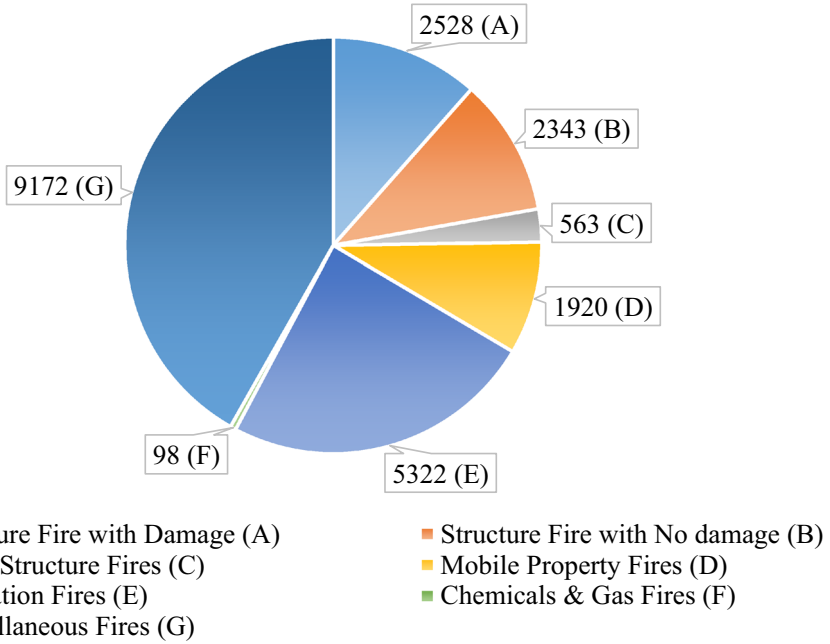


Figure 12. Fire statistics of New Zealand of year 2012/13.

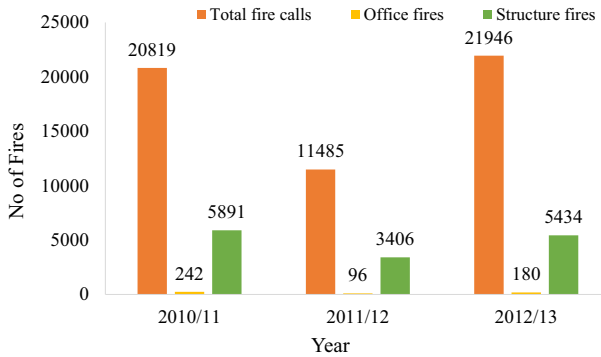


Figure 13. Fire statistics of NZ for three consecutive years.

building in Christchurch to New Zealand based on the population is performed. Christchurch had 1740 office buildings with a population of 341,469. Considering the same ratio of office building per person ($1740/341,500 = 0.005$), New Zealand is expected to have 21,616 ($= 0.005 \times 4,242,000 = 21,616$) office buildings for 4,242,048 people. r_{fi} for New Zealand is calculated using Eq. 5 as 0.0083 ($= 180/21,616$).

This indicates that Christchurch has a lower probability of occurrence of a structure fire in office buildings as compared to the entire country. This helps to

Table 2
Fire and Structure statistics of NZ and Christchurch

Factors	New Zealand	Christchurch
Population	4,242,000	341,500
Total fire calls	21,946	4524
Structure fire	5434	359
Office fire	180	12
No of office building	21,616	1740

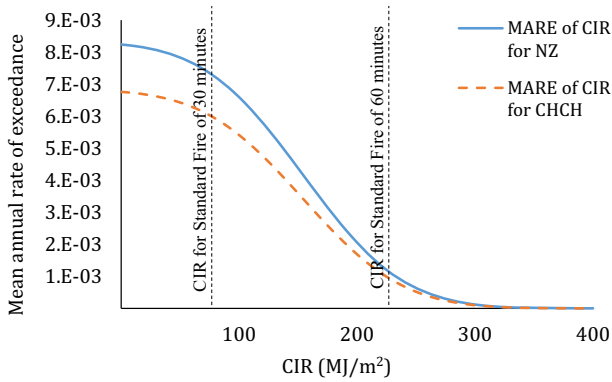


Figure 14. Hazard curve for CIR.

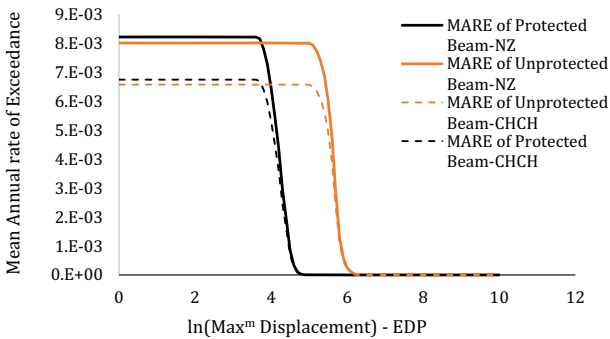


Figure 15. Mean Annual rate of exceedance of maximum vertical displacement.

calculate the annual rate of occurrence of fire and the structural response for the office building type. The above discussed probability calculation with certain assumptions can be improved with more accurate information of number of offices and number of office fires at the location (Christchurch city or in New Zealand as a whole).

The corresponding hazard curve (assuming a normal distribution) showing the mean annual rate of exceeding (MARE) a given value of CIR (the most suitable FSM) for both a New Zealand and a Christchurch office building is calculated with the help of Eq. 3. $P(T_{max})$ is evaluated based on the data of CIR collected for each fire profile and r_{fi} is calculated here for both New Zealand and Christchurch city. Using Eq. 3, MARE of CIR is shown in Fig. 14.

The data is very useful for design purposes since it indicates the probability of exceeding a CIR value in a fire compartment. Therefore, the designer need not perform fire analysis since this graph can be used to observe the fire scenario in a compartment of an office building. A similar interpretation can be drawn for other FSMs such as maximum fire temperature since it is a useful parameter for the response calculation of the member.

From Fig. 14 it is also possible to state that the probability of the cumulative incident radiation in a compartment to exceed 10 MJ/m^2 is 0.8% whereas to exceed 300 MJ/m^2 is very unlikely for both regions. The value of CIR up to 30 min and 60 min for the standard fire time–temperature curve is 80 MJ/m^2 and 234 MJ/m^2 respectively. For comparison, the standard fire requires 8.5 min to reach 10 MJ/m^2 of CIR and 70.5 min for 300 MJ/m^2 . This also indicates that the calculation of the intensity measures for the standard fire leads to very uneconomical results. The same interpretation can be drawn for the other FSMs based on the illustrated curves.

Similar to the hazard curves for efficient FSMs, the exceedance curve for the considered EDPs is calculated. The probability density function for EDP is assumed to be a normal distribution. This assumption is based on an ad hoc study which showed that EDP values fit best to a normal distribution at most FSM levels. $G(\text{EDP}/\text{FSM})$ is calculated and integrated with respect to $\lambda(\text{FSM})$. Figure 15 show the MARE curve for EDP (max displacement) for both protected and unprotected conditions. Similarly, MARE curve can be drawn for maximum steel temperature as an EDP.

It is clear from the exceedance curve for the EDPs that the protected beams reduce the probability of exceedance of larger deflections. This project may be extended to calculate the annual expected loss which may be useful in planning fire protection strategies.

9. Conclusion

This paper presented a modification in the adoption of the PBEE framework for PSFE. The study also investigated various possible FSMs and demonstrated a process to identify the most efficient and sufficient one by introducing a new selection criteria i.e. sufficiency. A new method called Fire Stripe Analysis (FSA) was

introduced, to avoid extensive scaling of fire profiles as is associated with IFA, in establishing relationships between FSMs and EDPs. A composite beam from a typical NZ office building was used to demonstrate the process of selection of an efficient FSM and the application of FSA. The beam, in both protected and unprotected conditions, was exposed to a family of fires. It was found that for the protected composite beam CIR was the most efficient FSM for both maximum vertical displacement and maximum steel temperature. The comparison of results for unprotected and protected beams revealed information about the advantage of fire protection in limiting structural failure. The annual rate of exceedance of hazard intensity and structural response was also calculated for New Zealand and Christchurch city with the help of the probability of occurrence of a structure fire and the probability of exceedance of a given level of FSM given an EDP. It is observed that Christchurch city has 15% less probability of exceedance of the specified fire severity level than the whole of New Zealand. These results can be extended to calculate annual loss or damage of a structure in NZ or Christchurch. Further investigations are underway to explore other FSMs to represent the severity of a fire more effectively. This work can be executed on framed structures (in both steel and concrete) with further extension to damage and loss estimation.

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