Pre- and Post-earthquake Fire Response of Partitioned Steel Structures: A Performance-based Approach

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In addition to the damage caused by the earthquake itself, post-earthquake fires (PEF) can potentially create even more significant damage than the earthquake itself. Therefore, in the absence of established provisions for PEF, a series of investigations need to be performed in order to develop a better understanding of the issue. As these investigations should be in the direction of performance-based approach, where providing adequate safety for the buildings' inhabitants is aimed, compatible methods should thus be employed to simulate the issue. A sequential structural analysis is performed here on the Life Safety performance level of a five story steel moment resisting frame. The frame is first subjected to an earthquake load with the PGA of 0.35g. This is followed by a fire analysis, using the ISO834 fire curve and the iBMB fire curve. The time it takes for the structure weakened by the earthquake to collapse under fire is then calculated. As a point of reference, fire-only analyses are also performed for undamaged frame. The results show that while the fire resistance of the frame exposed to ISO curve and iBMB curve are almost similar to each other (about 27 minutes), the failure shape are completely different. By contrast, while the PEF resistance of the frame exposed to ISO curve is around 26 minutes, it is less than 17 minutes under iBMB curve. Nevertheless, the failure shapes in case of PEF analysis and under both curves are almost the same. It is then concluded that the fire and PEF analysis under iBMB curves are more compatible with the concept of performance-based design. Whilst the investigation is for a certain class of buildings, the results confirm the need for the incorporation of PEF in the process of analysis and design, and provide some quantitative measures on the level of associated effects.

Keywords: *Post earthquake fire, Fire resistance, Steel Structures, Performance based design, iBMB fire curve*

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1 SCOPE

Post-earthquake fire (PEF) is considered as a major concern in seismic areas. Past records have shown that the effects of PEF may even be worse than the earthquake itself (Farahani, Tahershamsi et al. 2020). For example, in the 1906 post-earthquake fire of San Francisco, most of residential and commercial buildings were completely ruined, resulted in thousands of fatalities. It is estimated that the fire itself accounted for more than 80% of the total damage (Behnam 2017). The effects of PEF on buildings are two-fold; one is the damage due to the burning of non-structural material, which possesses material value, and the other is the damage that is due to excessive structural loads on the building (Chen. S., Lee. George C. et al. 2004). As the post-earthquake fire loading has not been considered in the available codes, these excessive loads may thus lead to rapid collapse of the buildings. From a different perspective, there are usually two ways to suppress a fire; by provided facilities inside the buildings, such as sprinklers and vertical pipes and/or by rescue teams. After an earthquake, however, there is a high-probability of them not working properly. Meanwhile, there is a high-probability of damaged infrastructures such as bridges that result in the closure of roads. Accordingly, more time will be spent on extinguishing the fire by rescue teams than during a normal fire. This time may be greater as helping people trapped under rubble takes priority. On the other hand, according to the performance-based design approach, the "life safety" criterion is the criterion by which most (if not all) building structures are designed. Life safety guarantees that even under extreme loading situation, structures will remain gravity load-bearing allowing occupants to safely evacuate the building even though minor to major damage, depending on the level of importance of structure, is allowed during an extreme load level (Borden 1996). While different procedures are adopted for structural seismic design and structural fire design, because of difference between structural seismic behavior and structural fire behavior, they all point to the fact that adequate safety shall be provided for the building's inhabitants. However, providing adequate safety without appropriately quantifying the seismic or thermal loads may lead to a very conservative and non-economic design, e.g. over-dimensioning of structural elements for earthquake loads or over-establishing extinguishment facilities for fire loads. Adopting a performance-based approach is thus needed for the processes of design- for fire and earthquake-, which can then be used for evaluation of a building exposed to PEF.

The following sections present descriptions for structural seismic design and structural fire design based on performance, which are then used to investigate the risk of post-earthquake fire in an ordinary steel moment-resisting building.

1.1 Performance-based seismic design

Using the philosophy of earthquake design based on performance (ATC 2001), structural elements are designed to satisfy various levels of performance, e.g. Operational (O), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). According to the performance design criteria, the expected performance of structures shall be controlled by assignment of each structure to one of several "Seismic Use Groups" (FEMA450 2003). In buildings designed for the IO level, it is expected that after an earthquake, only minor damage is sustained by structural and non-structural elements. Buildings designed to meet the LS level of performance, would sustain notable damage with the values of total drift around 2.5%, if hit by the design level earthquake. Most earthquake codes strongly recommend that the exterior walls shall permanently be attached to the structural components to resist the effects of earthquake motions (ASCE 2006). Nevertheless, there is a high-possibility of the breaking down of internal walls and partitions. Overall, the structures at this level of performance can still carry all gravity loads and no failure will occur. Buildings designed for the CP performance level, would sustain more damage compared to the other mentioned performance. At this level, it is expected that the imposed drift would be more than 5%, which can lead to extensive damage to structural and non-structural components (FEMA356 2000).

1.2 Performance-based Fire design

According to the performance-based approaches with regards to fire, the main objective of design is confining the fire inside a compartment and not allowing to broaden out to other areas (Franssen J-M, Kodur V et al. 2009, Kodur, Garlock et al. 2012). This objective can be met by employing active measures such as sprinklers and vertical pipes and/or passive measures such as employing non-combustible materials to reduce the fire risk. However, if the fire cannot be controlled, for example due to active measures not working properly, the aim of the design is to assure the building stability for adequate time until either the fire are put under control by rescue teams, or the inhabitants of the building are safely evacuated. The time that a structural member exposed to fire can maintain its integrity against applied loads is called fire resistance, which is commonly expressed in minute (Kodur and Dwaikat 2007). In the performance-based approach, fire resistance is accounted for by involving various parameters such as the compartment size, the locations and the dimensions of the openings, the fire scenarios, the heat transfer mechanism, the load ratio, the material nonlinearity and the failure criteria (Meacham and Custer 2002, ACI216 2006). Nevertheless, fire safety provisions are mostly established for fire-only situation and no pre- or post-earthquake conditions are defined (Alderighi and Salvatore 2009). After an earthquake, depending on the level of intended seismic performance, buildings are sustained damage results in not working properly active facilities such as sprinklers and vertical pipes. Hence, a double effort has to be made to develop appropriate performance-based provisions for PEF. In doing so and as the first step, the fire resistance of buildings should be evaluated by consideration of the new geometry of structural components.

2 PREVIOUS STUDIES

The performance of buildings subjected to fire after earthquake has been investigated by researchers in the past, but has received more attention since the horrific event of '9/11' (Chen. S., Lee. George C. et al. 2004, Mousavi, Kodur et al. 2008). Tomecek and Milke (Tomecek and Milke 1993) in a two-dimensional study showed that steel columns protected by fire-proofing materials have much more fire resistance than un-protected columns. They also showed that even 4% loss of protection layer could significantly decrease the fire resistance of the columns up to 40%. A similar study was performed by Ryder et al. (Ryder, Wolin et al. 2002) but in a three-dimensional fashion to show the effect of loss of fire protection materials on the fire resistance of steel columns. Initially, only fully protected columns were exposed to a 90 minutes fire exposure to find the temperature rise at the exposed surfaces. Then various areas of the protection layers were removed from the columns. The partially unprotected columns were then subjected to fire. The results showed that even when a minor area of fire-protection materials is removed, significant decline is observed in the fire resistance. A similar study was also performed by Wang and Li (Wang and Li 2009) that showed the fire resistance of steel columns with partial damage to fire protection in much less than those of fully protected.

Della Corte et al. (Della Corte, Landolfo et al. 2003) investigated unprotected steel moment-resistant frames and their responses when subjected to fire following an earthquake. Assuming elastic perfectly plastic (EPP) behavior of steel and considering geometrical P-Δ effect with P from gravity loads and Δ from the earthquake, the fire-resistance rating was found using numerical methods. Nevertheless, no degradation of stiffness was considered in Della Corte et al.'s study, which can be an issue of discussion. In addition, no active fire protection systems were considered in the process of analyses. The fire analysis was then accounted for both situations of before and after earthquake. The results showed that the drift ratio is an important parameter, which affects the fire resistance. Another study which contained both numerical and experimental elements on single-storey, multi-bay steel frames when are exposed to large uncontrolled PEF, was carried out by Hosam et al. (Hosam, Senseny et al. 2004). They considered unprotected elements in their analyses; however, every bay was protected by firewalls to reduce losses in uncontrolled fire. The firewalls were indeed assigned to meet the first objective of the performance-based fire design, where controlling the fire inside the compartment is aimed. It was shown that the type of failure, whether inward or outward collapse, and the PEF resistance of the models are greatly dependent on both fire scenarios and the gravity loads.

Further study of steel frames was carried out by Zaharia and Pintea (Zaharia and Pintea 2009). They investigated two different steel frames, designed for two return periods of ground motion: 2475 years and 475 years. The seismic response of each structure was then evaluated by a pushover analysis developed by Fajfar (Fajfar 1996). While the frame designed for the 2475 years return period remained elastic in the pushover analysis, the weaker frame designed for the 475 years return period sustained notable inter-story drift. They then performed a fire analysis on both frames, which confirmed that the fire resistance of the structures, considering their deformed state under earthquake, is lower than that of structures that do not have any history of deformation prior to the application of the fire. Alderighi and Salvatore (Alderighi and Salvatore 2009) performed a numerical investigation for composite steel frames with circular concrete-filled columns. The frames were designed to meet high-ductility set by Eurocode 8. Then, performing a pushover analysis, the seismic behaviors of the frames were evaluated. A series of thermo-mechanical analyses under different fire exposure time were then performed in order to categorize the potential structural failure by the state of stress at the critical cross-sections. Consequently, the influence of various restraining conditions on the performed thermo-mechanical analyses was investigated.

Braxtan (Braxtan 2010, Braxtan and Pessiki 2011) investigated the efficacy of sprayed fire resistive material (SFRM) in steel moment frames buildings exposed to fire after earthquake; mainly focusing on the axial load behavior of the columns. Several scopes were considered in Braxtan study such as, the bond of SFRM to the members, the seismic behavior of the SFRM frames subjected to cyclic loading, temperature distribution in the cross-sections in case of damaged SFRM and then structural analyses of the strength of the columns exposed to fire. The intact SFRM frames were designed to meet the strong column-weak beam theorem (ASCE41-06 2007). Then under the design earthquake, the frames were pushed to arrive at 1% drift and 3-4 % drift. While at 1% drift only debonding of the SFRM occurred, at 3-4% drift tearing of the SFRM due to buckling of the beam flanges occured. Imperfection of insulation, i.e. SFRM, accordingly, caused increase of heat inside the elements, which in turn accelerated the rate of decrease in both strength and stiffness. Pucinotti et al. (Pucinotti, Bursi et al. 2011) investigated the performance of steel-concrete composite joints when seismic actions are followed by fire loads. Using numerical analysis and experimental tests, they showed that the composite joints designed for fire or earthquake separately, could not provide the required minimum of fire resistance if subjected to fire following earthquake. They then suggested that at least 15 minutes fire resistance should be provided in case of fire following earthquake and accordingly, proposed a design method.

Faggiano et al. (Faggiano and Gregorgio 2010, Faggiano and Mazzolani 2011) investigated steel structures exposed to post-earthquake fire. They performed a coupled analysis consisting of both earthquake and fire. They showed that the earthquake-induced structures are more vulnerable than the undamaged structures. Then, based on FEMA356 procedure, Faggiano et al. developed a method they called robustness assessment, for evaluating the performance of buildings subjected to earthquake, and for suggesting fire performance levels for various conditions of fire. The proposed procedure was then exemplified with regard to specific class of steel structures. The author has also investigated the response of different structural systems under fire and PEF (Behnam 2017, Behnam 2019, Behnam 2019, Behnam and Abolghasemi 2019)

Aligned with the above-mentioned studies, in this study, a numerical investigation is

carried out on the PEF resistance of partitioned steel moment resisting frames designed for the LS level of performance. The study here includes a sequential analysis of the frame designed based on the AISC (ASCE 2005), subjected to earthquake and the aftermath fire. In doing so, a performance-based fire analysis is involved in the study.

3 COMPUTATIONAL PROCEDURE

A nonlinear post-earthquake fire analysis comprises three steps, which are the application of gravity loads, earthquake loads and then fire loads. The gravity loads are static and uniform and are maintained constant during the analysis. Simulation of earthquake loads is then performed using a pseudo earthquake load, which can be in a pushover style. When the earthquake loads are unloaded, the fire loads are applied to the structure. The modeling of the fire is explained in the following sections. Figure 1 schematically shows the mentioned processes.

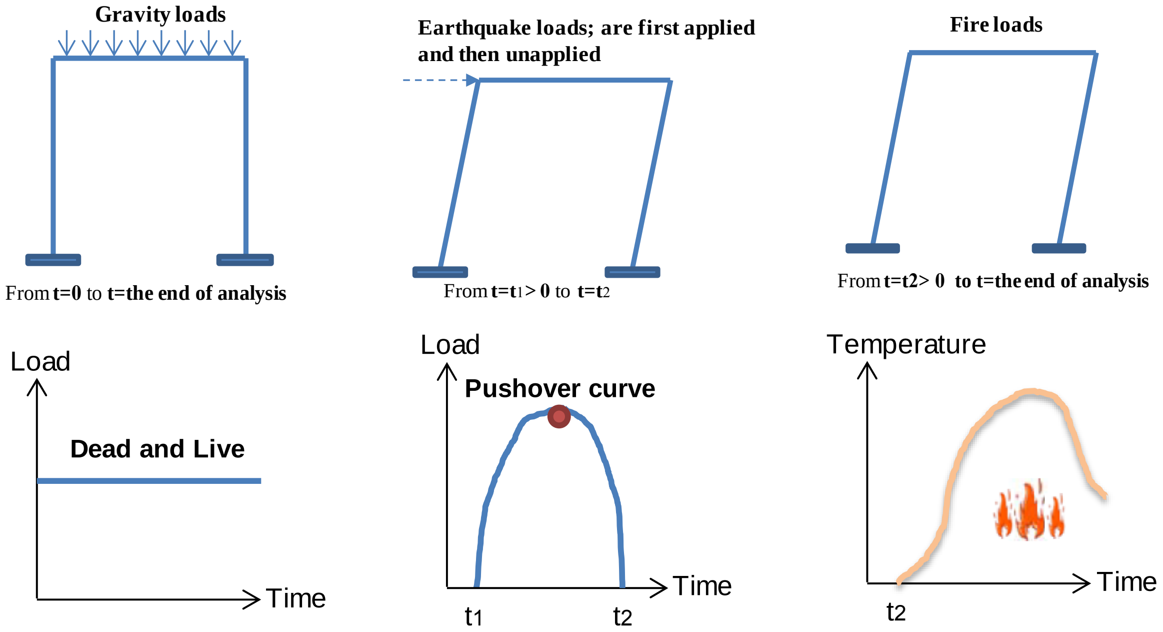
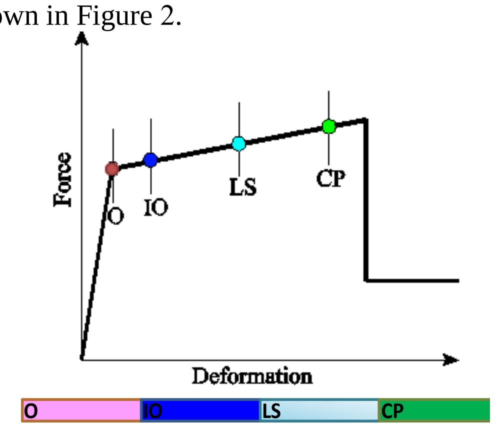


Figure 1: Stages of the PEF analysis

3.1 Seismic analysis

Structural seismic analysis can be performed through static or dynamic methods, which can also be linear or non-linear (Chopra 2001). Although most of static and linear methods are simple to use, they cannot entirely demonstrate the inelastic deformation or damage in structures, which is an important deficiency of these methods. On the other hand, earthquake-induced forces have a dynamic nature and therefore, only a dynamic analysis -as the most rigorous approach- can meet a realistic assessment (Humar and Mahgoub 2003, Mortezaei, Ronagh et al. 2011). Nevertheless, dynamic analyses are not employed in routine design, mostly because of significant complexities and uncertainties in the analysis (Saatcioglu and Humar 2003). In view of the mentioned difficulties, there is a more practical method, which is currently used to determine the design forces. The method known as static pushover analysis with the concept of performance-based design (Fardis 2007, Isaković, Lazaro et al. 2008). The method is based on the fundamental assumption that the structural response can be attributed to the response of an equivalent single degree-of-freedom (SDOF) system. Thus, the assumption implicitly demonstrates that the response is governed by a single mode with a constant shape throughout the analysis. There is a large consensus on the use of the method as a rational tool to estimate the maximum seismic response of multi degree-of-freedom (MDOF) structures, if the response of the structure is governed by a single mode (Fajfar 1996, Mwafy and Elnashai 2001, Bagchi 2004, Chopra and Goel 2004, Han and Chopra 2006, Azimi, Galal et al. 2009, Kalkan and Chopra 2011). For those structures not meeting the mentioned limitations, advanced pushover-based methods can be employed such as adaptive pushover analysis (Shakeri, Shayanfar et al. 2010, Shakeri, Tarbali et al. 2012) and modal pushover analysis (Chopra and Goel 2004, Chopra, Goel et al. 2004), in order to consider the effects of higher modes in the analysis. In conventional pushover analysis, using a specific load pattern, the structure is pushed to arrive at a displacement called the target displacement. The target displacement serves as an estimate of the global displacement that the structures is expected to experience in a design earthquake often represented by the roof displacement at the center of mass of the roof. In this study, a vertical distribution of loads proportional to the shape of the fundamental mode in the direction under consideration is used.

Here, the LS performance level is considered for the seismic analysis. In structures that experience plastic deformations, residual deformations remain and thus, the structures do not return to their initial condition. Using the definition of lumped plasticity (Roh, Reinhorn et al. 2012), the potential locations of high plasticity are introduced by plastic hinges (SAP2000-V14 2002). The moment-rotation behavior of each plastic hinge follows FEMA definitions as shown in Figure 2.



O : Operational ; IO : Immediate Occupancy ; LS : Life Safety ; CP : Collapse Prevention

Figure 2: Conceptual plastic hinge states

3.2 Definition of the fire models

The simulation of a fire in a compartment is a complex process that is commonly categorized into independent parts. In this view, the method by which the temperature produced by the fire is accounted for - termed as design fire - is of importance. Several methods have been developed to calculate the thermal actions produced by a fire on a compartment (Lundin 2005, Remesh and Tan 2007). These methods have been established either using parametric fires called “time-temperature curves”, or using “natural fires” which rely mainly on the volume of gas produced by the combustible materials in a covered space. Both models, are represented assuming a fully developed fire (Buchanan 2001). In the time-temperature curves based on the natural fire method, several factors are involved, such as the thermal characteristics of materials, type of occupancy, total area of compartment and size and shape of openings; the fire may behave as ventilation-controlled or fuel-controlled. In most small and medium compartments, fire is governed by a ventilation mode, which means that the growth of fire is limited by the availability of air. In large compartments, fires are mostly fuel-controlled, which means that the growth of fire is limited by the availability of combustible materials (Zehfuss and Hosser 2007). It is also worth mentioning that structural behavior in the cooling phase is of importance particularly in the joints of steel structures. During the decay phase, extensive tensile forces are created in axially restrained beams and a rapid collapse may occur. This is pertained to the fact that bolts, welds and structural members are not made with the same materials as the structure and this leads to different rate of temperature loss (Behnam 2017).

Among proposed time-temperature curves based on natural fires, one method has been proposed by Eurocode 1 Annex A, which is relatively simple to use. Nevertheless, the proposed method in some cases cannot demonstrate a realistic time-temperature curve, neither in the heating phase nor in the cooling phase. This is because; for example, the maximum temperature of the fuel-controlled compartments is set to an exact time, e.g. 20 minutes. In addition, while conducted fire tests have shown that it takes a few minutes from the start of the fire to reach a fully developed fire, the ignition phase is not considered in the proposed method. On the other hand, the heat release rate (HRR) is a function of burning rate, which is correlated with the dimension and location of the openings. In addition, for specific amounts of combustible materials inside a compartment, it is the openings that determine whether the heating phase is fuel-controlled or ventilation-controlled, which consequently results in different HRR (Denoël 2007). Although, the HRR is explained in Eurocode 1 Annex E, no sequential connection is made between the HRR and Eurocode 1 Annex A (Zehfuss and Hosser 2007). This, therefore, results in a non-realistic design fire, which is in contradiction with the concept of design based on performance (Lennon and Moore 2003).

In the view of the mentioned deficiency, there is a more realistic natural fire curve called iBMB that is proposed by Zehfuss and Hosser (Zehfuss and Hosser 2007), which considers both the HRR and the ignition phase. According to the method, using the consumed fire load density (q″, MJ/m^2), i.e. the total available combustible material in a compartment, the HRR is determined, which is consequently written as a function of time versus temperature (Zalok 2011). One of the most advantages of the method is about its applicability for the modeling of successive fires. Most residential and commercial buildings are subdivided into separate rooms with internals walls and partitions and almost without any fire classification. However, the partitions hamper any fast fire spread, which is nearer to the reality. Indeed, in reality a fire grows from one place and then spreads to the adjacent place till the entire storey is engulfed by the fire. Obviously, modeling of fire inside a building without consideration of partitions may result in a conservative estimation, which is not compatible with the concept of design based on performance. From a different view, after earthquake, depending on the seismic performance level, the fire modeling may change. As an example, at the LS level of performance, which most of residential and commercial buildings are designed for, it is expected that light internal partitions will be destroyed. Hence, a difference should be made between the fire models used for before and after an earthquake. In this respect, iBMB provides a suitable base to model a more realistic fire with consideration of different compartment boundary conditions.

In general, an iBMB fire curve is divided into three distinctive parts; ignition, heating and decay, which are concurrently linked with the HRR of a compartment as schematically shown in Figure 3. As is seen in the figure, from the beginning to t_1, the curve has a quadratic function of time. The HRR then remains constant until more than 70% of the fire load is consumed, which corresponds to t_2. Finally, t_3 corresponds to the end of cooling phase, is where the total fire load is consumed and the HRR declines to 0. As the fire curve is correlated with the HRR, the times (t_1, t_2, t_3) can then be accounted for based on the consumed fire load. In addition, in terms of being fuel-controlled or ventilation-controlled, the value of temperature at each step can separately be determined. The application of the method is explained hereafter.

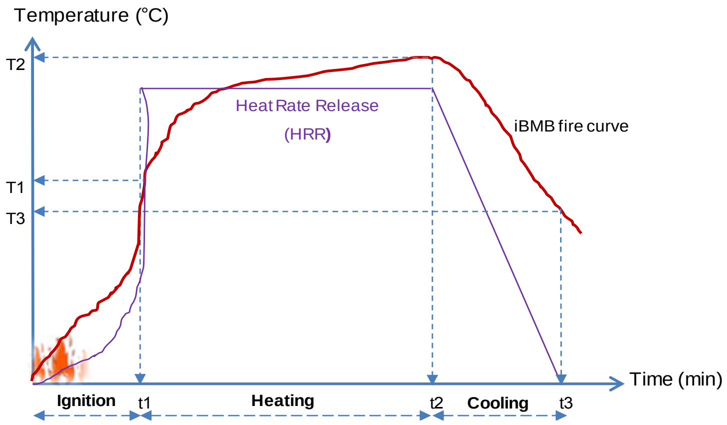


Figure 3: The HRR and the corresponding iBMB fire curve (Zehfuss and Hosser 2007)

The development of a fire is a successive process, which means it initiates from one cell and then spreads to other cells. Therefore, the HRR is firstly calculated for the cell itself, where the fire initiates. After the breaking down of the partitions of the cell, a new HRR is determined for the extended cell, while considerations of new geometry as well as larger openings are also taken into account. The partitions failure time can be extracted from the codes such as ASTM E119. Figure 4 schematically shows the rate of heat release in a partitioned compartment.

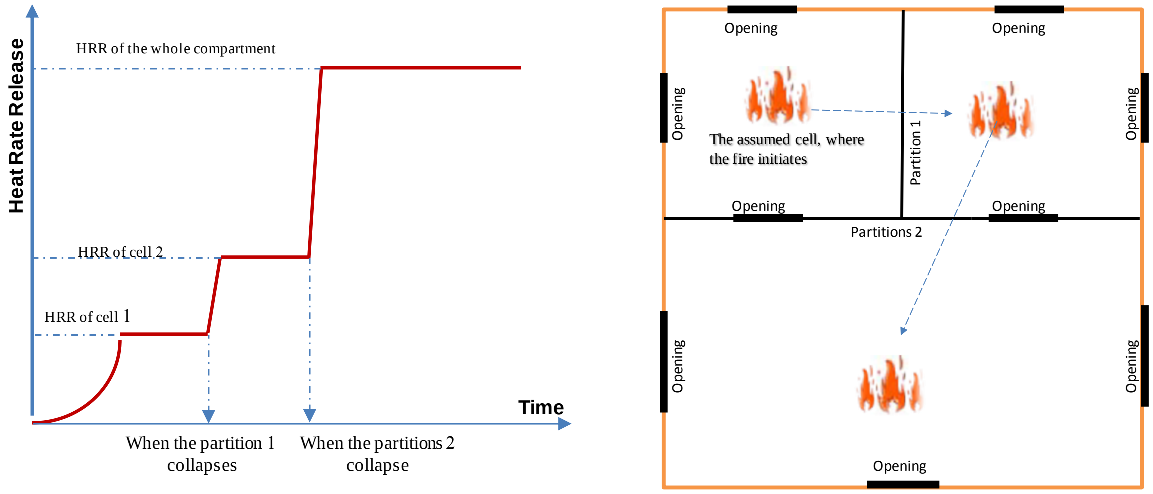


Figure 4: An example of the HRR development in case of successive fire in a partitioned compartment

3.2.1 Influence of active measures

The fire load of a compartment, $q_{t,d}$ (MJ/m^2), can be determined using Equation 1, where $q_{f,d}$ is the occupancy of the fire compartment and A_{roof} (m^2) and A_t (m^2) are the areas of the roof and the total area consisting of the area of walls, roof and floor, respectively.

$$q_{t,d} = q_{f,d} \times (A_{roof} / A_t) \quad (1)$$

Nevertheless, the determined value in Equation 1 may change in the presence or absence of active measures such as detection and suppression systems (Hietaniemi and Mikkola 2010). In that case, using the description stated in Annex E of Eurocode 1, the occupancy of fire competent is calculated using Equation 2.

$$q_{f,d} = \delta_{q1}\, \delta_{q2}\, \delta_n\, m\, q_{f,k} \quad (2)$$

Where, δ_{q1} is the risk of fire activation, which increases while the floor area increases. For this study, δ_{q1} is 1.54. δ_{q2} is the type of occupancy, which for the purpose of this study is 1.0. Combustion factor, shown by m in Equation 2, is between 0 and 1, and is derived from experimental results. It is assumed to be 0.8 for most cellulosic materials. $q_{f,k}$, or the so called characteristic fire load density can be calculated using the mentioned values in Annex E of Eurocode 1(BSI 1991). In order to take into account the effect of active firefighting, the factors δ_n are applied in Equation 2. The factors are mainly based on either available fire-extinguishing systems established in the building such as detection systems, and sprinklers or those used by fire brigades and rescue teams in a professional manner. The proposed values are based on a normal condition and as such cannot be utilized in a PEF scenario where the whole response teams are under stress. Consequently, the mentioned factors should be modified. Table 1 shows the values of δ_n for two situations of before and after earthquake, which are consequently used to modify Equation 2 where the occupancy of the fire compartment ($q_{f,d}$) is accounted for.

Table 1: Firefighting measures

Fire situation	Automatic water extinguishing systems	Water supply	Automatic fire detection	Automatic fire alarm	Automatic transmission	An on-site fire brigade	An off-site fire brigade	Safe access routes	Normal firefighting devices	Smoke exhaust systems	Sum
	δ_1	δ_2	δ_3	δ_4	δ_5	δ_6	δ_7	δ_8	δ_9	δ_{10}	$\Pi\, \delta_n$
Fire alone	1.0	1.0	1.0	0.73	0.87	0.61	0.78	1.0	1.3	1.0	**0.39**
After EQ	1.0	1.0	1.0	0.73	0.87	0.71	1.0	1.5	1.5	1.5	**1.64**

3.2.2 Application of iBMB fire curve

The heat release rate (HRR) is determined using Equation 3, where $\dot{Q}$, $\dot{m}$, γ and H_{net} stand for, the HRR, the mass flow rate, the combustion coefficient and the net calorific value, respectively. The combustion coefficient and the net calorific value are often obtained from wooden fire loads, which are assumed to be 0.7 and 17.3 MJ/kg, respectively.

$$\dot{Q}(t) = \dot{m}\ (t)\gamma\, H_{net} \tag{3}$$

As mentioned earlier, the ventilation conditions can change the HRR, resulting in being fuel-controlled or ventilation-controlled. If the fire is ventilation-controlled, then the burning rate ($\dot{m}$) is determined through Equation 4, in which A_w and h_w are the area of the opening in the compartment and the averaged height of openings, respectively. The equation is based on wooden fire loads, which are commonly referenced in accounting for the design fire.

$$\dot{m} = 0.1\, A_w\, h_w^{\,½} \tag{4}$$

Substituting Equation (4) in Equation (3) with the mentioned values for γ and H_{net}, the maximum HRR ($\dot{Q}_{max,v}$) can therefore be re-written as Equation (5).

$$\dot{Q}_{max,v} = 1.21\, A_w\, h_w^{\,½} \tag{5}$$

On the other hand, if the fire is fuel-controlled, then the maximum HRR ($\dot{Q}_{max,f}$) is determined by Equation (6), where A_f is the floor area of the compartment.

$$\dot{Q}_{max,f} = 0.25\, A_f \tag{6}$$

As shown in Figure 6, the HRR $\dot{Q}(t)$ from 0 to t_1 changes quadratically, which is generally determined using Equation (7), where t_g is called the time of fire growth and for ordinary buildings is often assumed to be 300 sec.

$$\dot{Q}(t) = (t / t_g)^2 \tag{7}$$

Therefore, based on Equation (5) and (6), the maximum $\dot{Q}(t)$ in a compartment with a defined fire load density, is determined using Equation 8.

$$\dot{Q}_{max} = Minimum\ (\dot{Q}_{max,v}\ \ \&\ \ \dot{Q}_{max,f}) \tag{8}$$

For the adoption of the iBMB fire curve, a maximum fire load density needs to be selected as a reference, e.g. q''=1300 MJ/m^2 for ordinary buildings. The HRR diagram for various areas is then plotted based on the selected reference. Depending on being fuel-controlled or ventilation-controlled, the temperature ($T_{i=1,2,3}$) corresponds to time ($t_{i=1,2,3}$) – see Figure 6- is thus determined. If the fire is governed by ventilation, then Equations (9) to (13) are employed, otherwise, i.e. in case of fuel-controlled, Equation (14) to (17) are employed, as shown in Table 2. The shown equations are represented based on the reference fire load density of q''=1300 MJ/m^2.

Table 2: Temperature-time equations for fuel or ventilation-controlled compartment

Ventilation-controlled fire		Fuel-controlled fire	
$T_1 = -8.75 \times 1/O - 0.1\,b + 1175$ °C	**(9)**	T_1=24000k + 20 °C for k≤0.04 and 980°C for k>0.04	**(14)**
$T_2 = (0.004\,b - 17) \times 1/O - 0.4b + 2175$°C ≤1340°C	**(10)**	T_2=33000k+ 20 °C for k≤0.04 and 1340°C for k>0.04	**(15)**
$T_3 = -5.0 \times 1/O - 0.16\,b + 1060$ °C	**(11)**	T_3=16000k + 20 °C for k≤0.04 and 660°C for k>0.04	**(16)**
$O = A_w\ \ h_w^{½} / A_t$; A_t = The total area with openings	**(12)**	$k = [\,(\dot{Q}^2 / A_w\ \ h_w^{½} / A_T\, b)\,]^{⅓}$; A_T = The net area	**(17)**
$b = \sqrt{c\rho\lambda}$; see Table 3	**(13)**	**Note:** O, called opening factor and b, called averaged thermal property of enclosure	

Using the above-mentioned equations, the HRR for the values lower than the reference value at each time is determined for three stages of Figure 6. For sections of $0 \le t \le t_1$, $t_1 \le t \le t_2$ and $t > t_2$, the temperature is accounted for using Equation 18 to 20, respectively.

$T = \dfrac{T_1 - T_0}{t_1^2} t^2 + T_0$, where T_0 is the ambient temperature and assumed to be 20 °C (18)

$$T = (T_2 - T_1)\sqrt{\frac{t - t_1}{t_2 - t_1}} + T_1 \tag{19}$$

$$T = (T_3 - T_2)\sqrt{\frac{t - t_2}{t_3 - t_2}} + T_2 \qquad (20)$$

The HRR, for each section is accordingly accounted for. For the first section, the HRR, Q_1 is determined using Equation 21.

$$Q_1 = \int_0^{t_1} \left(\frac{t}{t_g}\right)^2 dt \qquad (21)$$

As it is well-accepted that the maximum HRR occurs when more than 70% of the combustible materials is consumed, the HRR in the second section $Q_{2,x}$ is thus accounted for using Equation 22, in which Q_x is the fire load and is ascertained by Equation 23, where q_x is the fire load density and is lower than the reference fire load density (q″) of 1300 MJ/m^2.

$$Q_{2,x} = 0.7Q_x - Q_1 \qquad (22)$$

$$Q_x = q_x A_f \qquad (23)$$

Finally, the temperature at the third section changes logarithmically, which is accounted for using Equation 24.

$$T_{3,x} = (T_3 / \log_{10}(t_3 + 1))\log_{10}(t_{3,x} + 1) \qquad (24)$$

By inserting Equation 24 into 20, the temperature at the third branch of Figure 6 is ascertained using Equation 25.

$$T = (T_{3x} - T_{2,x})\sqrt{(t - t_{2,x}) / (t_{3,x} - t_{2,x})} + T_{2,x} \qquad 25)$$

Meanwhile, in iBMB fire curve mentioned earlier, the flashover process is also considered. The HRR at the ignition phase is accounted for using Equation 26, where $\dot{Q}_{fo}$ stands for the rate of heat release at flashover process.

$$Q_{f0} = 0.0078A_T + 0.378A_W\sqrt{h_w} \qquad (26)$$

Accordingly, the time of the development the fire is determined using Equation 27.

$$t_{1,fo} = \sqrt{t^2{}_g Q_{fo}} \qquad (27)$$

3.2.3 Heat transfer

After deciding on the method of accounting for the temperature produced by the source of fire (T_{gas}), the problem of heat transfer reduces to differential equations with certain boundary conditions that allow the calculation of T_s at the surface of the structural elements and the transfer of heat through the element to the other side. The heat is mostly transferred through one or more of the basic laws, i.e. convection, radiation and conduction (Denoël 2007, Drysdale 2011). While in order to transfer the heat via convection, existence of a medium is required, radiative heat transfer needs no material between the fire source and the solid (Torero 2012). Although the radiation has a predominant role in heat transfer from the source of fire to the surface of the solid, the role of convection cannot entirely be ignored (Welch, Jowsey et al. 2007). In addition, a fully engulfed compartment assumption, i.e. when it is assumed that $T_{gas} = T_s$, will not necessarily result in a conservative fire resistance (Denoël 2007, Welch, Jowsey et al. 2007). In order to illustrate this in a simple manner, a one-dimensional problem through a cubic compartment is assumed with insulated walls and floor having a non-insulated ceiling named here as the “solid”. The transfer of fire is therefore only in the vertical direction from T_{gas} at the fire source (floor level) to T_s (bottom of the ceiling) to T_i (the other side of the ceiling) although there is some radiation from wall to wall radiative exchange which is ignored here for simplicity.

Accounting for the fire resistance of the selected case in this study, the SAFIR program (Franssen 2011) is employed. The program is capable of performing structural analysis under ambient or elevated temperature. Structures exposed to fire, are analyzed in two stages, thermal analysis and structural analysis. In the thermal analysis, the history of temperature inside the cross-sections is stored to be used either independently or for the subsequent structural step. For the purpose of this study, the time-temperature curves according to iBMB and ISO834 are used, which are assumed to be the furnace temperature (T_{gas}). Then, the surface temperature (T_s), is accounted for based on the work performed by Welch et al.(Welch, Jowsey et al. 2007) and is applied to the surface of the solid (steel material here).

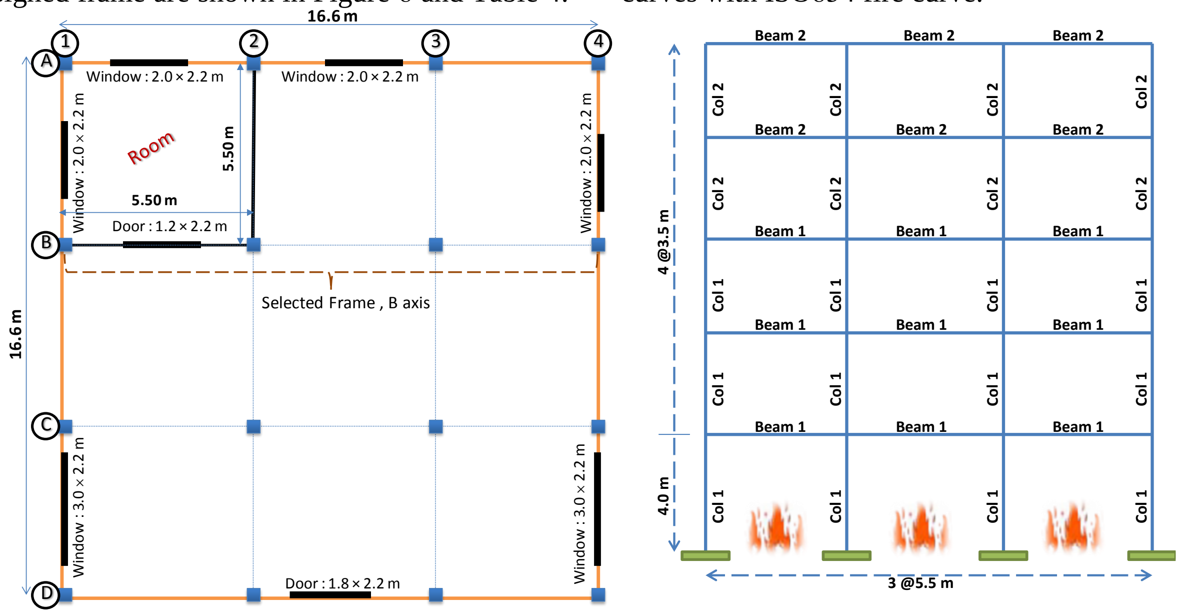
4 CASE STUDY

An unprotected steel resisting frame is selected from a five-story building, designed for LS levels of performance as shown in Figure 5. The floor and the ceiling are made of normal weight concrete. The external walls and the internal walls-partitions are made of standard bricks and gypsum board, respectively. For the partitions, a fire resistance of 20 minutes is assumed (Manzello, Park et al. 2008). Table 3 shows the thermal characteristics of the materials in the building. The characteristic fire load density of 511 MJ/m2 is assumed for accounting for the thermal actions, which is modified for both situations of normal and after earthquake using Equation 2. It is also worth noting that two-dimensional model is considered here. Nevertheless, as slab component can increase the overall fire resistance of the frame (Moss and Charles Clifton 2004), an effective length is considered. The effective length of 1000 mm is adopted from ACI 318-08 code (ACI318 2008) and used in this study.

Table 3: Thermal characteristics of the considered materials

	Specific heat (J/kg K) (c)	Material's density (kg/m^3) (ρ)	Thermal conductivity (W/mK) (λ)
Floor and Roof	1000	2400	1.6
External walls	840	1600	0.7
Partitions	852	743	0.276

The selected frame is loaded and then is designed using steel plates (welded compact sections) with the yield stress of 240MPa and for peak ground acceleration (PGA) of 0.35g. The frame is dimensioned for the load combinations of 7.0kPa for dead load and 2.0kPa for live load. Combination of 100% dead load and 20% of live load for the designed frame is used to find the required mass for calculating the earthquake load (ASCE 2006). The resulting sections for the designed frame are shown in Figure 8 and Table 4. For the fire scenario, first floor is exposed to fire as shown in Figure 8b. In addition, it is assumed that the fire initiates from the partitioned room and then spread to the whole floor. The fire curve is determined using iBMB method as explained and for two cases: 1) normal situation, i.e. before earthquake, 2) after the earthquake, while partitions have been destroyed. In order to improve our understanding of the behavior, a comparison is also made between the results of iBMB fire curves with ISO834 fire curve.

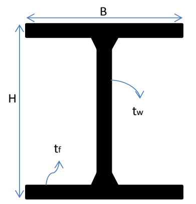


a) Plan view of the building

b) Geometric properties of Frame axis B

Figure 5: The case study and the position of the openings

Table 4: Sections dimensions

Col 1	Col 2	Beam 1	Beam 2	 Dimensions are in mm
H=400 B=300 t_f=20 t_w=15	H=350 B=300 t_f=15 t_w=12	H=300 B=300 t_f=20 t_w=15	H=250 B=250 t_f=15 t_w=12	

5 RESULTS AND DISCUSSION

Sectional dimensions of the beams and columns for the considered frame were given previously in Figure 5 and Table 3. All material properties were also introduced in previous sections. The sequential analysis comprises three main stages that are the gravity loading, followed by the seismic pushover analysis, and finally the post-earthquake fire. In seismic analysis, the structure is subjected to a monotonically increasing lateral load to arrive at a certain level of displacement. According to the philosophy of design based on performance, it is expected that structures designed for a specific level of performance remain at the assumed level when subjected to the “Design earthquake”. Using the FEMA356 procedure, a target displacement shall be calculated for controlling the situation of the structure for the assumed performance level. In this respect, SAP2000 program is used for the pushover analysis. Moreover, the FEMA procedure is used to define hinge positions and properties. The lateral forces corresponding to the target displacement extracted from the SAP2000 program are then inserted into the SAFIR program for performing sequential analysis. Figure 6 shows the pushover curve for the mentioned performance level.

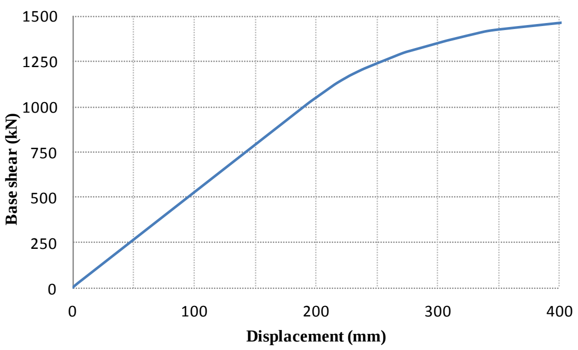


Figure 6: The pushover curve at LS level

he final stage is to apply the fire loads on the undamaged and damaged frame and according to the two cases as mentioned earlier. Clearly, in an undamaged frame, the fire follows the application of gravity loads, but in damaged frame the fire follows the application of the gravity loads and then the earthquake loads. Tables 5 and 6 represent summarized calculations of the thermal actions according to iBMB method and Figure 7 shows the time-temperature curves according to iBMB method compared with the ISO834 standards fire. In the figure, the sharp increase in the partitioned floor graph is where the partitions fail (20 minutes), which results in the released energy swiftly increases owing to the fast ignition of the fire loads in the rest of the floor. In addition, the plateau part of the time-temperature curve in the partitioned floor graph is pertained to the constant HRR in the room, before breaking down the partitions. As is seen, there are notable differences between the results of iBMB curves with those of ISO834 curve, which can lead to a different design of structural members. Based on the heat transfer laws, the accounted for temperatures are then applied to cross-sections. The results are consequently used for the structural analysis. On the other hand, applying the temperature to the cross-sections is a very important point and requires adequate considerations. For the case of partitioned floor, it is clear that some of structural members are located inside the room, while other members are located outside the room. This means that structural members inside the room are exposed to the fire for a longer time than those outside of the room. As an example, columns B1 and B2 and beam B1-2 from the selected frame are subjected to the fire for a longer time. After the earthquake however, as partitions would have been destroyed, no difference needs to be made between the exposure times for the members. As an example, Figure 8 shows the temperature distribution in the cross-sections of column C1 when exposed to pre- and post-earthquake fire and according to iBMB curves.

Table 5: Summarized calculations for case 1 (partitioned floor)

$A_{f(room)}$ = 30.25 m^2 ; $A_{f(floor)}$ = 275.6 m^2	
A_w = 11.44 m^2	
A_t= 130.9 m^2	
A_T= 119.5 m^2	
h_w= 2.2 m	
Ventilation $factor_{(room)}$ = 16.97 $m^{3/2}$; Ventilation $factor_{(floor)}$ = 51.6 $m^{3/2}$	
Opening $factor_{(room)}$= 0.130 $m^{1/2}$; Opening $factor_{(floor)}$= 0.068 $m^{1/2}$	
q″ (decreased by active measures) = 189 MJ/m^2	
Total fire load room Q_{189}= 5719 MJ ; Total fire load floor Q_{189}= 52088 MJ	
Averaged thermal properties $b_{(floor)}$= 1721; Averaged thermal properties $b_{(room)}$= 1320.6	
Floor: $\dot{Q}_{max}$ = Min ($\dot{Q}_{max,v}$ & $\dot{Q}_{max,f}$) = Min (62.39 & 68.9) = 62.39: Thus **Ventilation-controlled** **Room**: $\dot{Q}_{max}$ = Min ($\dot{Q}_{max,v}$ & $\dot{Q}_{max,f}$) = Min (20.50 & 7.56) = 7.59: Thus **Fuel-controlled**	
$Q_{1300\ (reference)}$= 35325 MJ	$Q_{1300\ (reference\text{-}remained)}$= 329180 MJ
$Q_{189\ (reference)}$= 5717 MJ	$Q_{189\ (remained)}$= 49659 MJ
t_1= 7 min; Q_1=1573 MJ	$Q_{1(remained)}$= 13167 MJ
Q_2= 25955 MJ; t_2= 67 min	$Q_{2(remained)}$= 217259 MJ
Q_3= 107484 MJ; t_3= 115 min	$Q_{3(remained)}$= 98754 MJ
Room: $Q_{2,189}$ =2429 MJ; $t_{2,189}$= 900 sec ≈ 15 min	Floor: $Q_{2,189}$ =30887 MJ; $t_{2,189}$= 1866 sec ≈ 31 min
Room: $Q_{3,189}$ = 1715 MJ; $t_{3,838}$=2289 sec ≈ 38 min	Room: $Q_{3,189}$ = 18772 MJ; $t_{3,838}$=3348 sec ≈ 55 min
$T_{1,f}$= 686 °C	$T_{1,v}$= 874 °C
$T_{2,f}$= 935 °C	$T_{2,v}$= 1338 °C
$T_{3,f}$= 464 °C	$T_{3,v}$= 711 °C
$T_{2,189}$=746 °C	$T_{2,189}$= 932 °C
$T_{3,189}$=338 °C	$T_{3,189}$=419 °C

Table 6: Summarized calculations for case 2 (broken down internal partitions)

A_f = 275.6 m^2
A_w = 34.8 m^2
A_t= 763.6 m^2
A_T= 728.8 m^2
h_w= 2.2 m
Ventilation factor = 51.6 $m^{3/2}$
Opening factor= 0.068 $m^{1/2}$
q″ (increased by active measures) = 838 MJ/m^2
Total fire load Q_{838}= 230953 MJ
Averaged thermal properties b= 1721
$\dot{Q}_{max}$ = Min ($\dot{Q}_{max,v}$ & $\dot{Q}_{max,f}$) = Min (62.39 & 68.9) = 62.39 : Thus **Ventilation-controlled**
$Q_{1300\ (reference)}$= q A_f = 1300 × 275.6= 358280 MJ
t_1= 10 min; Q_1=14331 MJ
Q_2= 236465 MJ; t_2= 77 min
Q_3= 107484 MJ; t_3= 139 min
$Q_{2,838}$ =160234 MJ; $t_{2,838}$= 3178 sec ≈ 53 min
$Q_{3,838}$ = 69286 MJ; $t_{3,838}$=5720 sec ≈ 95 min
$T_{1,v}$= 874 °C
$T_{2,v}$= 1338 °C
$T_{3,v}$= 711 °C
$T_{2,838}$= 1172 °C
$T_{3,838}$=656 °C

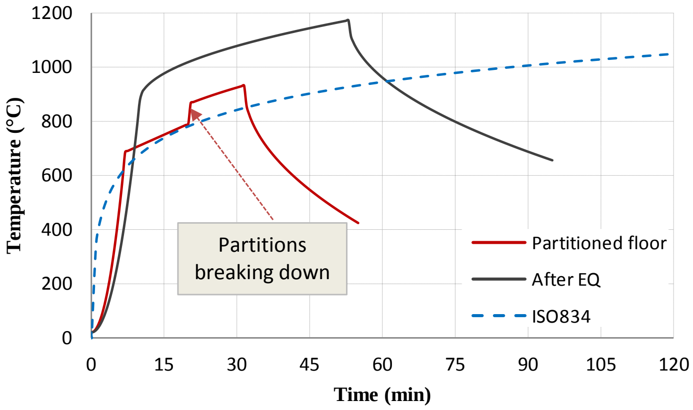
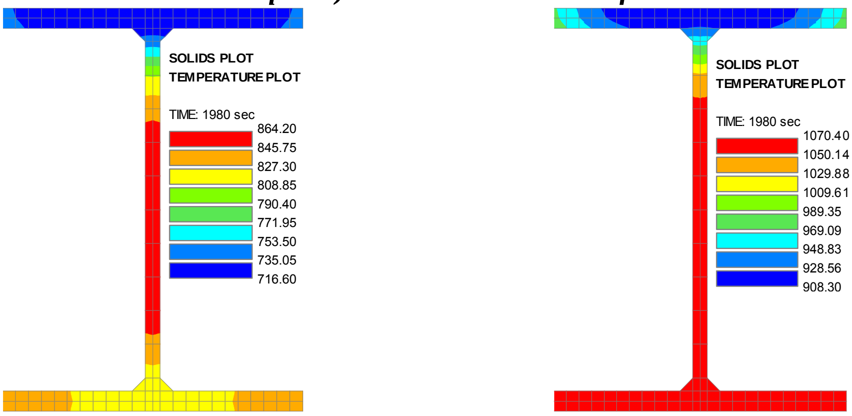


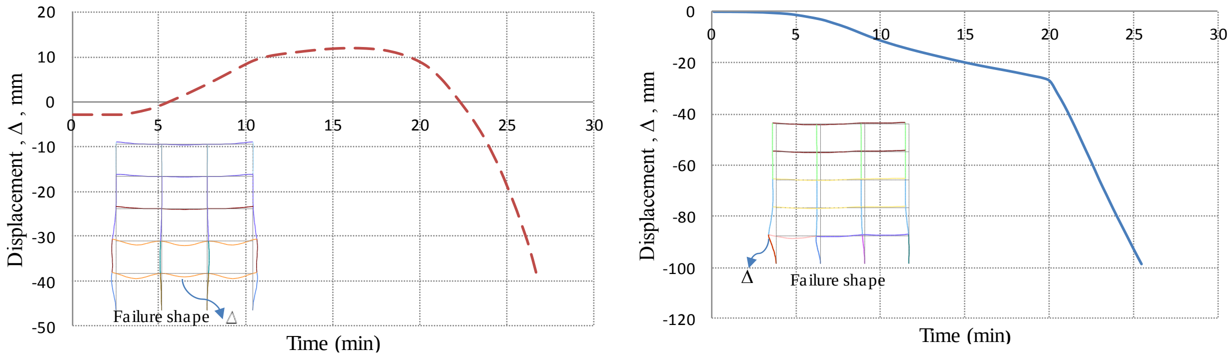
Figure 7: Comparison of time-temperature curve based on iBMB method (before and after earthquake) and ISO834 standard fire



a) before earthquake b) after earthquake

Figure 8: Distribution of temperature inside the cross-section of Col. C1 using iBMB curve

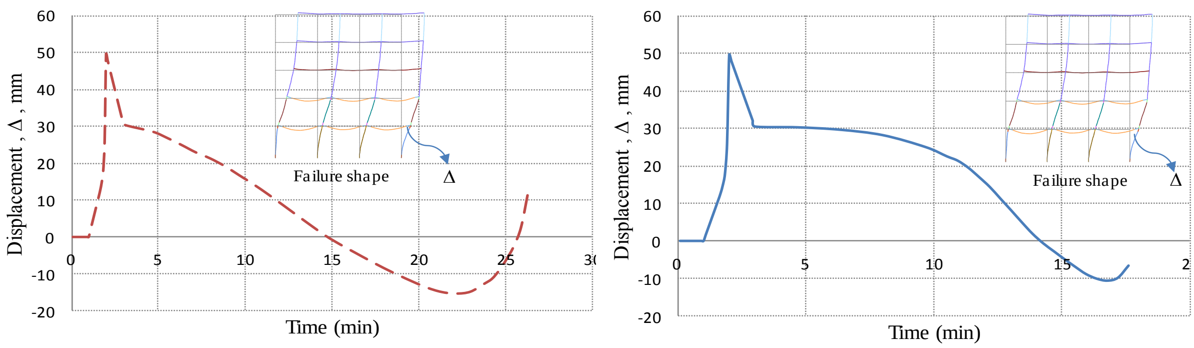
Figure 9 shows the fire resistance based on the iBMB and ISO curves before an earthquake. The fire resistance is defined as a time at which the displacements, either globally (i.e. the drift of a certain point) or locally (i.e. the deformations at the middle of a beam) go beyond chosen thresholds. The thresholds have been identified by the curve for displacements versus time step merging towards the vertical asymptote by a 1% error. These thresholds implicitly represent the definition of fire resistance of a member as described earlier, where the member is not able to resist the initially applied gravity loads (Kathryn. and Buchanan. 2000, Almand., Phan et al. 2004). As is seen in the figure, the fire resistance based on ISO and iBMB are roughly identical. Nevertheless, no similarity is observed in terms of the failure shape. While the frame subjected to ISO curve fails locally, the frame subjected to iBMB curve fails globally. The local collapse depends largely on the collapse of beams, while global collapse is mainly governed by considerable lateral displacement of the columns. The difference between the failure shapes in the frame is mainly because a uniform fire load is applied to the frame when ISO curve is employed, resulting in a symmetrical change. However, the iBMB fire load is not applied uniformly as explained earlier, which leads to a lateral collapse. In addition, Figure 12b shows a gentle slope between the time of 7 and 20 minutes, corresponding to the starting of heating phase and breaking down of the partitions, respectively. This is pertained to the fact that during these time period, the fire is only limited to the partitioned room. Nevertheless, after the destruction of the partitions, the displacement swiftly increases, mostly because of the fast application of the fire loads to the rest of the members. It is worth mentioning, that the frame does not enter the cooling phase and fails while it is in the heating phase. This can be considered as an important outcome showing that although the cooling phase is an important phase (as mentioned earlier), there is a possibility of the structure not entering that phase.



a) ISO834 curve b) iBMB curve

Figure 9: Fire resistance based on iBMB and ISO834 curves – before earthquake

Figure 10 shows the fire resistance based on the iBMB and ISO curves after earthquake. The sharp increase and then decrease is due to, the frame being pushed to a certain level of displacement and then unloaded. Some degree of damage would now exist in the structure. The damaged structure is then loaded with fire as a sequential load, which arrives at the structure in its residually deformed state. To do this, SAFIR allows a function to be written inside its computing environment that allows the importing of pushover loads extracted from SAP2000 at the target displacement. These loads are then applied inside the SAFIR environment to the structural model to bring the structure to the target displacement first, after which unloading takes place to be continued by re-loading with fire. As is seen in the figure, there is a difference between the results of ISO curve with iBMB curve. The frame fails at around 26 and 17 minutes when using the ISO and the iBMB curves, respectively. Nevertheless, in terms of failure shape, the frame collapses globally for both curves as shown in the figure.



a) ISO834 curve b) iBMB curve

Figure 10: Fire resistance based on iBMB and ISO834 curves – after earthquake

On the other hand, apart from the failure shape, no difference is observed between the results of fire-alone analysis and PEF analysis when ISO curve is employed. However, the results of pre- and post-earthquake fire analysis are not similar when iBMB curve is used. This difference is mainly attributed to the fact that in the iBMB method the boundary conditions of the compartment are considered. It therefore can be concluded that the fire resistance based on iBMB curves is more compatible with the concept of performance-based design.

6 CONCLUSION

Post-earthquake fire (PEF) is a reality that has not received adequate attention in the past, neither in the earthquake nor in the fire codes. Investigating the effects of PEF on urban structures is thus important, as these buildings comprise a major part of urban areas. The investigation should be in the direction of performance-based codes, where adequate safety shall be provided for the buildings' inhabitants, even under extreme conditions. To do so, appropriately simulation of the applied loads (seismic loads and fire loads) to the structures is a vital step of the investigation. In this research, sequential non-linear analysis is proposed for the PEF analysis of the case studied. An unprotected steel moment resisting frame from a five-story building is selected and then is designed based on ASCE code. The frame is then pushed to the maximum allowable inter-story drift, which was assumed to satisfy the Life Safety performance level. Pushover curve is then extracted for use in a subsequent analysis. Sequential loading consisting of gravity and lateral loads that is followed by fire. The P-Δ effect and the residual lateral deformation as well as degradation in stiffness are considered. For the fire analysis, a performance-based fire curve is used. As most residential and commercial buildings are subdivided into separate rooms, a growing fire successively extends from one room to the following one until the whole floor is surrounded by the fire. Assuming that after the earthquake, the partitions are destroyed, new boundary conditions for the fire analysis are adopted. Accordingly, the following remarks can be made:

- There is great similarity between the results of fire analysis under ISO curve with iBMB curve, in which the frame collapses at around 27 minutes. Nevertheless, different failure shapes are observed. While the frame under ISO curve collapses locally, global collapse is observed under iBMB curve. The local collapse occurs in beams and the global collapse is due to considerable lateral movement in columns.
- While the PEF resistance of the frame exposed to ISO curve is around 26 minutes, it is about 17 minutes under iBMB curve. The frame, however, collapses globally under both curves.
- When using the iBMB curve, the boundary conditions of the compartment are directly involved in the analysis and it can therefore be concluded that fire and PEF resistance based on iBMB curves are closer to the reality, which in turn is more compatible with the concept of design based on performance.

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