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Post-earthquake fire performance-based behavior of reinforced concrete structures

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Abstract. Post-earthquake fire (PEF) can lead to a rapid collapse of buildings damaged partially as a result of prior earthquake. Almost all standards and codes for the design of structures against earthquake ignore the risk of PEF, and thus buildings designed using those codes could be too weak when subjected to a fire after an earthquake. An investigation based on sequential analysis inspired by FEMA356 is performed here on the Immediate Occupancy, Life Safety and Collapse Prevention performance levels of structures, designed to the ACI 318-08 code, after they are subjected to an earthquake level with PGA of 0.35g. This investigation is followed by a fire analysis of the damaged structures, examining the time taken for the damaged structures to collapse. As a point of reference, a fire analysis is also performed for undamaged structures and before the occurrence of earthquake. The results indicate that the vulnerability of structures increases dramatically when a previously damaged structure is exposed to PEF. The results also show that the damaging effects of post-earthquake fire are exacerbated when initiated from the second and third floor. Whilst the investigation is made for a certain class of structures (conventional buildings, intermediate reinforced structure, 3 stories), the results confirm the need for the incorporation of post-earthquake fire into the process of analysis and design, and provides some quantitative measures on the level of associated effects.

Keywords: post-earthquake fire; sequential analysis; fire resistance; reinforced concrete structures; performance-based design

1. Introduction

It is an accepted fact that providing adequate safety is a main objective of building construction in urban areas. Pursuing this objective depends mostly on the existence of a safe environment. In unforeseen natural disasters, the safety of the buildings can decline quickly. This lack of safety can be further worsened if a natural disaster such as an earthquake is followed by an urban disaster such as a fire. In this situation, providing adequate time for extinguishing the fire and/or evacuating people trapped in the fire must be a key aspect of a post-earthquake fire (PEF) safety strategy. Past statistics have proved that PEF can create even more damage compared with the earthquake itself. For example, the city of San Francisco became an inferno after the strong earthquake in 1906, with the fire continuing over three successive days and destroying the city. It is estimated that the fire accounted for 80% of the total damage.

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The effect of PEF on buildings can be categorized into two kinds; one is the damage owing to the burning of non-structural materials such as furniture and possessions; the other is the damage that caused by excess structural loads on the building (Chen *et al.* 2004). This is important because the majority of structural members are not designed for extreme conditions, combining gravity loads, lateral loads and aftershock loads. Consequently, the buildings which have been moderately damaged by an earthquake can rapidly be destroyed in a subsequent fire. From a different perspective, as earthquakes can cause serious damage to lifeline structures, arterial roads and bridges, fire brigades would have increased difficulty in controlling fires. Accordingly, they would have to spend considerably more time to control the post-earthquake fire than in usual situations. This time may further increase, as helping people trapped under the rubble takes priority, with untended fires therefore leading to a conflagration. In this case, it is difficult to estimate the size of the catastrophe (Scawthorn 2008).

1.1 Performance levels of structures

Using the philosophy of design based on performance, structural elements are normally designed to satisfy various levels of performance, some of which are 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”. In FEMA 450, for example, there are three “Seismic Use Groups”, which are categorized based on the occupancy of the structures within the group and on the relative consequences of earthquake-induced damage to the structures. Design codes specify progressively more conservative strength, serviceability, and detailing requirements for structures in order to attain minimum levels of earthquake performance suitable to the individual occupancies. Structures contained in these groups are not specific to a certain seismic zone; rather they are spread across all zones from high to low hazard and, as such, the groups do not really relate to hazard. Rather the groups, categorized by occupancy or use, are used to establish design criteria intended to produce specific types of performance in earthquake events, based on the importance of reducing structural damage and improving life safety (Fig. 1).

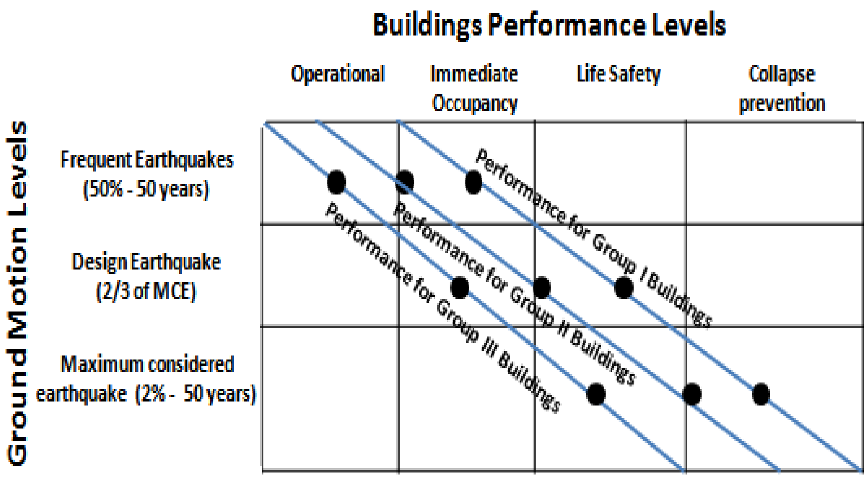


Fig. 1 Building performance levels versus earthquake severity (FEMA450 2003)

In terms of post-earthquake recovery and service to the public, certain types of occupancies, such as major medical facilities (named as Group III in FEMA450) are vital to public needs, as are those structures that contain substances that, if released into the environment, are deemed hazardous to the public. These special occupancies are supposed to remain in the "Immediate Occupancy" category when subjected to the design earthquake, and shall remain operational under all frequent earthquakes. One class lower in performance requirement are Group II structures, which contain a large number of occupants or those where the occupants' ability to exit is restrained. Schools, day care centers and suburban medical facilities are examples of this category. Group I, then, includes those ordinary structures, such as residential and commercial buildings, which shall remain in the life safety level if subjected to the design earthquake.

The various performance levels required for buildings of different categories can implicitly be met by increasing the "design earthquake" by a factor called the "importance factor". The importance factor adjusts the intensity of earthquake in the design so that the required performance level under the "design earthquake" is met. Specifically, in important structures, it is expected that after an earthquake only minor damage will be sustained by the structural elements. Minor damage is quantified with a value of drift limited to 1% according to FEMA356. This is the boundary of Immediate Occupancy and Life Safety level of performance. At this level of drift, while some elements go beyond the yield point in the corresponding pushover curve, non-structural components may not operate properly owing to mechanical failure or lack of amenities such as disconnection of electricity. If designed well, important structures are expected to remain habitable after the shock. Structures such as schools fall into this category. Indeed, they are not only supposed to allow immediate occupancy for the purpose they have been designed for, but also to act as shelters for those who are in need after the earthquake. It is therefore crucial to investigate the performance of these structures under a PEF scenario. Most buildings in urban areas, however, are residential and commercial buildings, which are designed to meet the Life Safety level of performance. The main objective at this performance level is to limit both the amount of damage in buildings and to subsequently provide more safety for the inhabitants. To meet this objective, limiting the values of drift to around 2% is recommended by FEMA356 as a margin for Life Safety and Collapse Prevention levels of performance. At the LS level, it is expected that, along with some residual displacement in the building, there is considerable damage to both structural and non-structural elements. However, there should be adequate resistance left in the structure to carry the applied gravity loads and no failure should occur. Obviously, buildings designed for CP performance level, sometimes called Limited Safety, will sustain more damage compared to other levels of performance. At this level, it is expected that the imposed drift would be more than 4%, which can lead to extensive damage to structural components.

2. State of the art

Understanding structural behavior becomes more important when a fire after a seismic event occurs, because the fire adds to level of complexity. In general, "fire resistance rating" is defined as the period of time in which the integrity of a member subjected to fire is maintained to resist applied loads. This definition is correlated with various factors, one of which is the type of building being designed (McGhie 2007). Indeed, the purpose is not only to provide sufficient time to evacuate people trapped inside the burning building, but also to reduce the possibility of any conflagration. Although typically, fire resistance ratings are presented in national building codes, such as NRCC 2005 and IBC 2006, many of them provide only for fire condition and not for post-earthquake fire. This is an important point because the vulnerability of earthquake-damaged structures exposed to PEF is much greater than the fire itself. This is because earthquake excitation may produce residual lateral deformation as well as residual stresses on the members (Mousavi *et al.* 2008). Moreover, experiences from past earthquakes confirm that both active and passive fireproofing systems, such as sprinklers and fire control systems, may become seriously damaged, thereby considerably decreasing the fire resistance capability. As pointed out earlier, “Fire resistance” is defined as the time at which the element is unable to resist applied loads, such as gravity loads (Kodur and Dwaikat 2007). Therefore, evaluation of a building’s performance under PEF is essential, requiring careful scrutiny. The PEF resistance of a building is dependent on various factors, including the deformed geometry and degradation in stiffness resulting from earthquake (Zaharia and Pintea 2009). In reinforced concrete structures, in addition to the aforementioned factors, the effects of the level of damage, including tensile cracking, removal of rebars’ cover and compressive crushing on PEF resistance, have to be considered as well. Assuming ductile behavior of RC elements, a typical moment-curvature relation can be idealized to separate stages. While it seems that tensile cracking, as the first stage of cracking, has no significant effect on the PEF resistance, major cracking resulting in removal of rebars’ cover or crushing of concrete in compression drastically reduces the PEF resistance (Ervine *et al.* 2012).

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’. For example, Della Corte *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 P-Δ effect with P from gravity loads and Δ from the earthquake, the fire resistance rating was then taken into account. Ignoring the degradation of stiffness in Della Corte et al.'s study is an issue subject to discussion.

Further study on steel frames was carried out by Zaharia and Pintea (2009). They investigated two different steel frames, designed for two return periods of ground-motion; 2475 years return period and 475 years. The seismic response of each structure was then evaluated by a pushover analysis. 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 notably

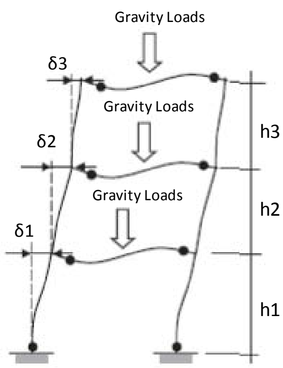
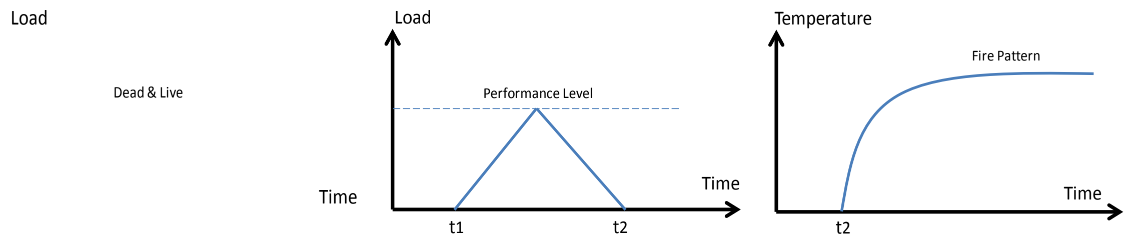


Fig. 2 Residual deformation resulting from the earthquake

lower than that of the structures that do not have any history of deformation prior to the application of the fire. Mostafaei and Kabeyasawa (2010) investigated PEF resistance of reinforced concrete structures with shear wall. The model was first subjected to an equivalent Kobe 1995 earthquake on a shaking table. The damage sustained by the structure was then quantified by observation through use of a method called Axial-Shear-Flexure Interaction (ASFI) (Kabeyasawa and Mostafaei 2007), which was then used in a numerical thermal analysis to find the temperature rise in and around both the cracked and the intact sections subjected to fire. Fire loading was then applied to the damaged structure to consider the effect of concrete's degraded compressive strength. The results showed that the ability of the structure to sustain gravity loads in the cracked components is considerably lower than in the intact components. Although the compressive strength of concrete plays an important role in the overall fire resistance, other factors such as P-Δ effect and changes in the modulus of elasticity have also to be considered in order to improve accuracy.

In the same year, Faggiano and Mazzolani (2011) investigated steel structures exposed to post-earthquake fire. They performed a coupled analyses consisting of both earthquake and fire. Based on FEMA356 procedure, Faggiano *et al.* developed a method for evaluating the performance of buildings subjected to earthquake, and suggesting fire performance levels for various conditions of fire. Clearly, in a coupled analysis, both residual deformation and degradation of mechanical characteristics are applied. However, the method can be more effective for steel structures because, as was previously mentioned, in reinforced concrete structures, seriously damaged sections play an important role in PEF resistance. Recently, Ervine *et al.* (2012) conducted an experimental and numerical study on a reinforced concrete element subjected to conventional loads followed by a fire load. Applying two concentrated vertical loads on the specimen and recording the subsequent deflection, the created cracks were observed through the member. The model was then subjected to fire load to find the effect of created cracks on the thermal propagation inside the section. The results showed that minor tensile cracking would not significantly change the heat penetration inside the section. They concluded that the fire resistance of the intact specimen and of the minor damaged specimen are roughly identical (Ervine *et al.* 2012). However, exposing the rebar directly to fire, e.g. in case of crushing of cover, considerably changes both the thermal and the structural behavior of the specimen. Another study on this issue is currently being undertaken by Bhargava *et al.* (2010) on the fire resistance of an earthquake-damaged RC frame. A nearly full-scale portal frame was first loaded by the relevant gravity loads and then subjected to a cyclic lateral load based on the Indian standard in a quasi-static fashion. The load-control mode was considered to meet 2% drift, corresponding to the Life Safety performance level as described in FEMA 356 code. The cracks' widths were then observed using optical tools, non-destructive tests and ultrasonic method. A computational analysis was also performed using the finite element method with ABAQUS (2008) for comparing the test and the analytical results. The results show a good conformity with FEMA356 descriptive definition of damage levels at various performance levels, such as Immediate Occupancy and Life Safety. They suggest that the results of quasi-static cyclic test can be used for the subsequent fire analysis.

Aligned with the abovementioned studies and FEMA356 performance level definition, in this study, a series of numerical investigations is carried out on the PEF resistance of conventional buildings designed for Life Safety performance level. The study here includes a sequential analysis comprising both earthquake and the aftermath fire and using FEMA356 descriptive performance levels, while consideration is given to effects such as the removal of cover on the PEF resistance.



(a) Gravity loading is constant during the time

(b) Earthquake loading is applied in a short time

(c) Fire loading is applied after the unloading process of earthquake loads

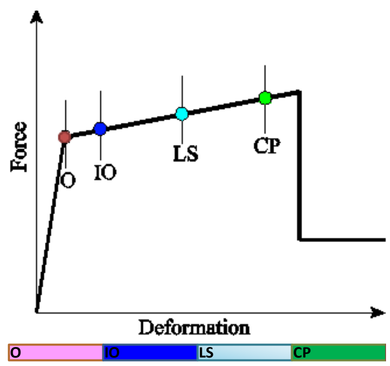
Fig. 3 Stages of the sequential analysis

3. Methodology

3.1 Sequential analysis

Performing a sequential analysis is a useful method to consider the effect of both earthquake and fire on a structure. Fig. 3 schematically shows the stages of the nonlinear sequential analysis. The first stage of the loading is the application of gravity loads, which are assumed to be static and uniform. A pseudo earthquake load then follows in a pushover style reaching its maximum value and returning to zero in a short time. Clearly, during this time, gravity loads are also applied. The pattern that is chosen for applying the earthquake load is similar to pushover analysis with the difference that the structure is unloaded after reaching a certain level of load. Here, it is assumed that the maximum level of earthquake load corresponds to the defined performance level, i.e. IO, LS or CP, according to FEMA356. This assumption is in line with the seismic design philosophy in which the performance level of structures shall not exceed the assumed level when subjected to the “Design Earthquake”. Therefore, the structures are pushed to these levels and then unloaded.

Load duration is not important for gravity and earthquake loads, because in this study long-term effects such as creep and shrinkage are not included in the analysis. Thus, any arbitrary load duration could be chosen for these loads. It should be noted that no dynamic effects are considered in this study. Finally, as can be seen in Fig. 3(c), the fire load is applied to the structure.



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

Fig. 4 Conceptual plastic hinge states

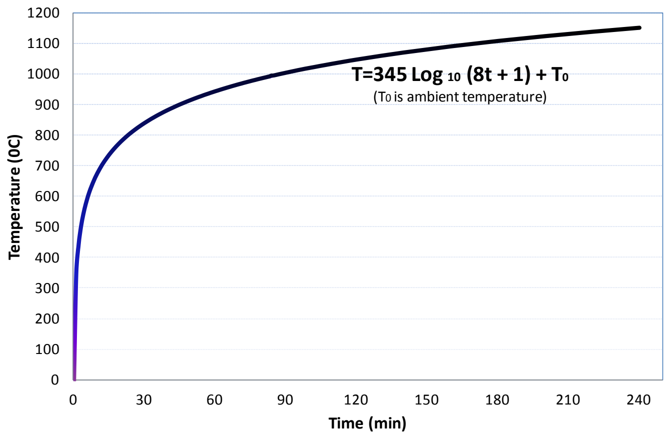


Fig. 5 Fire curve according to ISO834 (ISO 834 International Standard 1999)

Prior to fire loading, the properties of the structures are set to the reference temperature, but during fire, mechanical properties change with temperature. In this study, incorporating the explained pattern and using SAP2000 (SAP2000-V14 2002) for the pushover analysis and SAFIR for the fire analysis (Franssen 2011), sequential seismic and fire analyses are performed. In SAP2000, the lumped plasticity is considered while the moment-rotation behavior of each plastic hinge follows FEMA356 definition.

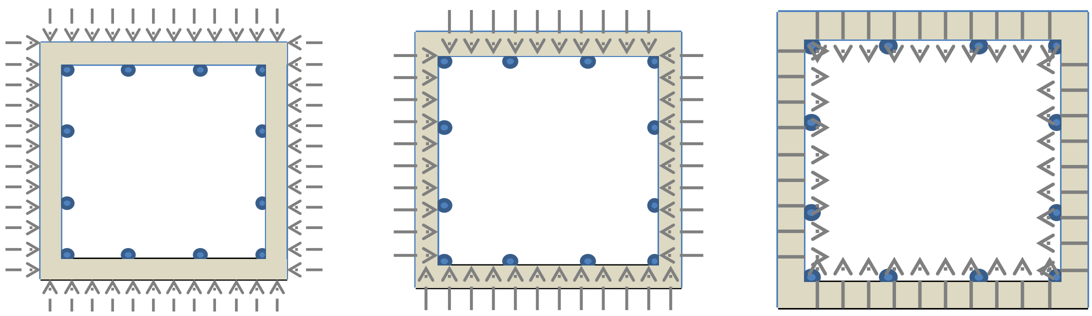
Fig. 4 shows a typical force-deformation curve for an assumed hinge. This figure also shows the performance levels as mentioned earlier. To calculate the fire resistance of the selected cases in this study, SAFIR that is a computer program written based on Finite Element Method (FEM) is employed. The program performs nonlinear analyses on one, two or three dimensional structures in which both geometrical and material nonlinearity are taken into account. The analyses can also be performed under ambient or elevated temperature. The stress-strain relationships for various materials, as well as their thermal characteristics, are embedded in the software, according to Eurocodes. Meanwhile, accounting for thermal action in a structure, both ‘time-temperature curves’ and ‘natural fire’ can be used. Structures that are exposed to fire are analyzed in two stages, thermal analysis and structural analysis. In the thermal analysis, the temperature inside the cross sections at every thermal step is stored to be used for the subsequent structural step. For the purpose of this study, the time-temperature curve according to ISO 834 without cooling phase is used, as shown in Fig. 5.

3.2 Material nonlinearity

Fiber element is the most capable model for nonlinear analysis of reinforced concrete members. Many researchers have developed the finite element formulation for this element. The model accounts for material nonlinearities in rebar steel and concrete (Zhao and Sritharan 2007, Godat 2008, Lin *et al.* 2009). A fiber beam element is made up of a series of sections along the element length, whose number and location depend on the integration scheme. The constitutive relation of the section is not specified explicitly, but is derived by integration of the response of the fibers, which follow the uniaxial stress-strain relation of the particular material. The consecutive material stress-strain curves are used to generate the moment-curvature and the axial force-deformation relationships. Concrete can be modeled depending on the region: the core that is confined; and the cover that is unconfined.

3.3 Fire frontiers

The previously mentioned definitions of performance levels in a concrete cross section are required for the post-earthquake fire analysis, because variation of temperature across the section is highly dependent on the state of damage. In FEMA356, it is stated that in the Immediate Occupancy performance level, minor damage in the structural elements is observed, which has no significant effect on PEF resistance (Ervine *et al.* 2012). On the other hand, in the Life Safety performance level, extensive damage is observed in beams and ductile columns, resulting in spalling of their cover. The dotted lines and the arrows in Fig. 6 show the assumed pattern of applied fire frontier for damaged beams and columns after the pushover analysis. This assumption is based on the authors' interpretation of the information available in the FEMA356 code, the Japan Building Disaster Prevention Association (JBDPA) and an experimental study performed by Bhargava *et al*. While none of the aforementioned references differentiates between the beam and the column responses as to the extent of cracking or concrete spalling, they all point to the fact that the concrete cover is no longer part of the section. In FEMA356, "Table C1-3 Structural Performance Levels and Damage", the different levels of damage in columns and beams are explained. Relating to quantity rather than quality, Bhargava *et al.* (2010) conducted an experimental study on a nearly full-scale RC frame, performed to find the level of damage when the frame is pushed to a certain level of displacement. Their results show that while at a roof drift ratio of 1.37%, flexural cracking was observed (corresponding to the drift ratio in Immediate Occupancy level of performance), at 2.11 % drift ratio (corresponding to the drift ratio of Life Safety level of performance) spalling and wide cracks in columns and beams were observed. The study does not reveal any differences between the columns and the beams. On the other hand, for structures designed for CP level of performance, it is expected that the structure would sustain considerable damage in beams and columns, much more than those mentioned in IO and LS levels of performance. Based on JBDPA, Meada *et al*. (2009) and Nakano *et al.* (2004) showed in several studies that when a structure sustains severe damage (corresponding to the Collapse Prevention performance level in FEMA356) crushing and spalling of concrete cover with exposed reinforcement is observed.



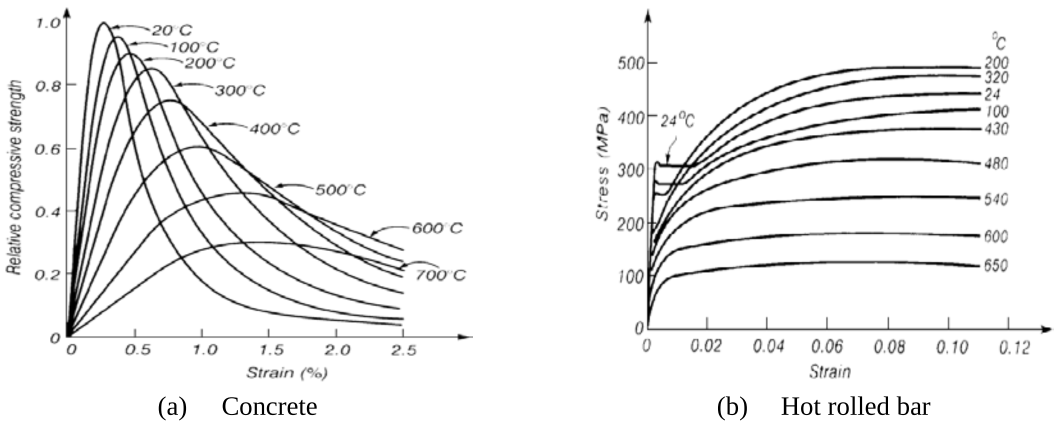
(a) IO level of performance (b) LS level of performance (c) CP level of performance

Fig. 6 Schematically applied fire frontiers on the sections in various performance levels. The arrows show fire frontiers

Overall, the PEF analysis in structures designed for IO level of performance is only followed by a minor residual displacement, while at LS level of performance, along with some residual deformation and degradation in strength and stiffness, the removal of cover in a region around the plastic hinges should be considered. At CP level of performance, however, the structures not only sustain severe damage and considerable degradation in strength and stiffness, even the totally exposed rebars need to be considered for the PEF analysis.

3.4 Reinforced concrete behavior under the effect of fire

Materials' thermal and mechanical characteristics change considerably when they are exposed to fire, which in many cases produces high levels of thermal stress in structures (Bamonte 2008). In addition, when a heterogeneous composite material with different thermal characteristics is subjected to elevated temperature, differential thermal stresses speed up the degradation. Concrete has low thermal conductivity, which creates slow transmission of heat inside the cross section. On the other hand, although the reinforcement bars have high thermal conductivity, they are generally protected by the concrete cover. Cracking and crushing of this concrete cover, however, causes more thermal propagation to penetrate at a quicker rate with serious negative outcomes. It is apparent that this can be worse if a previously damaged member (for example, as a result of earthquake loading) experiences high temperature, because the fire resistance of seriously damaged members is much less than that of intact members. In other words, the more the members are damaged, the shorter is the time to collapse during the PEF. Overall, along with increasing in temperature, the yield strength decreases in both concrete and rebar (Youssef and Moftah 2007). Fig. 7 shows the stress-strain relationship in hot rolled bars and concrete at high temperatures, as given in Eurocodes 2 and 3.

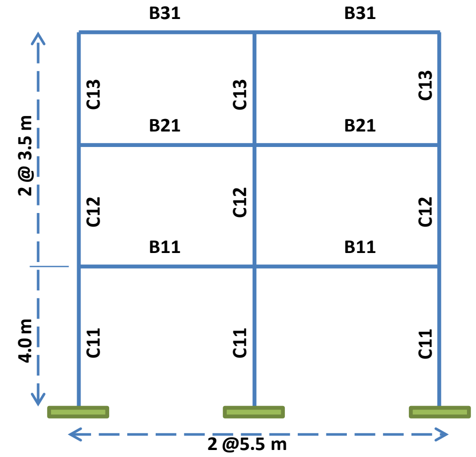


(a) Concrete (b) Hot rolled bar

Fig. 7 Stress-strain relationship at different temperatures

4. Case studies

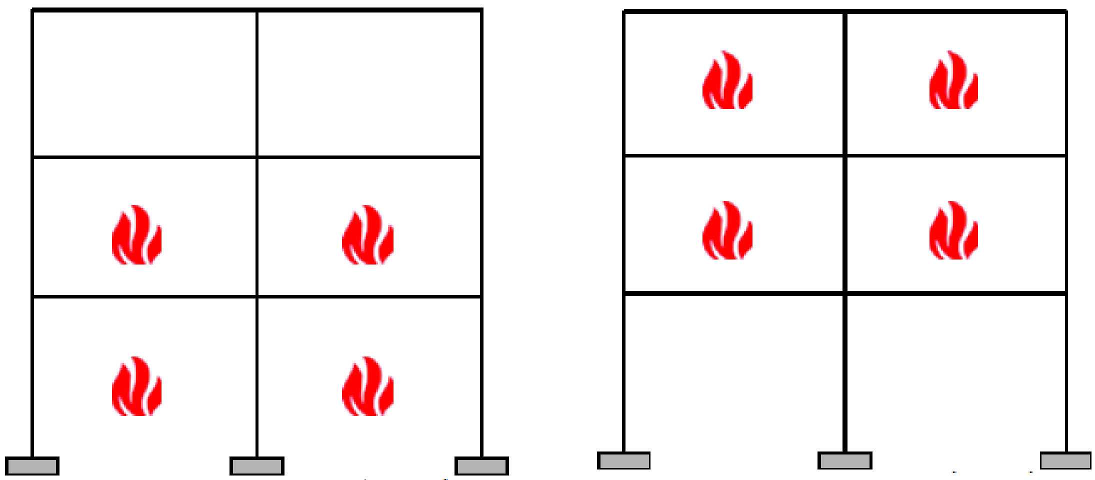
A reinforced concrete frame designed for IO, LS and CP levels of performance and for PGA of 0.35g is selected. The selected case is loaded and designed based on ACI 318-08 code and the properties of the designed frame are presented in Figure 8. The structure is made using normal



Dimensions are in cm

Type / Designed for	C11	C12	C13	B11	B21	B31
IO	45x45	40x40	40x40	45x45	40x40	40x40
LS	40x40	40x40	40x40	40x40	40x40	40x40
CP	40x40	35x35	35x35	40x40	40x40	35x35

Fig. 8 Geometric properties of concrete frame

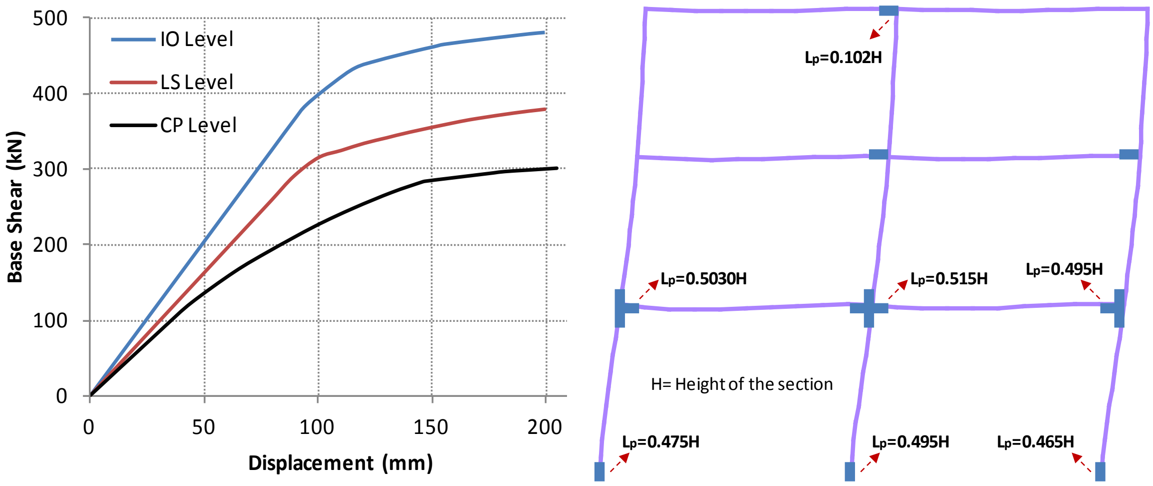


(a) The fire initiated from 1st and 2nd floors (b) The fire initiated from 2nd and 3rd floors

Fig. 9 Fire scenarios in the selected model

strength concrete with compressive strength of 25MPa and longitudinal and transverse reinforcing bars with yield stress of 400MPa. The frame is dimensioned for the load combinations of 8.0kPa for dead load and 2.5kPa for live load. The combination of 100% dead load and 20% of live load is used to find the required mass for calculating the earthquake load (ACI318 2008). In addition, the plastic hinge length (LP) can be found via several equations. Here Park and Paulay (1975)'s formula is used as a simple but accurate method, where LP= 0.5H and H is the section height.

Furthermore, the model is exposed to standard fire (ISO834 curve, without decay) and two different situations of fire for 5 hours, as shown in Fig. 9. For the thermal analysis, it is assumed that the concrete moisture content is 10 kg/m^3. Moreover, the thermal expansion coefficient of rebar and concrete are assumed to be 12×10-6 /$^\circ$C and 10×10-6 /$^\circ$C, respectively. Poisson's ratio of 0.2 is considered for the concrete. In addition, to improve our understanding of the behavior, the fire analysis is performed for the undeformed frame, i.e. before occurrence of the earthquake. It must be noted that while the exterior side of the external columns is not exposed to fire, all sides of



(a) Pushover curves (b) Extend of plastic hinges at LS level

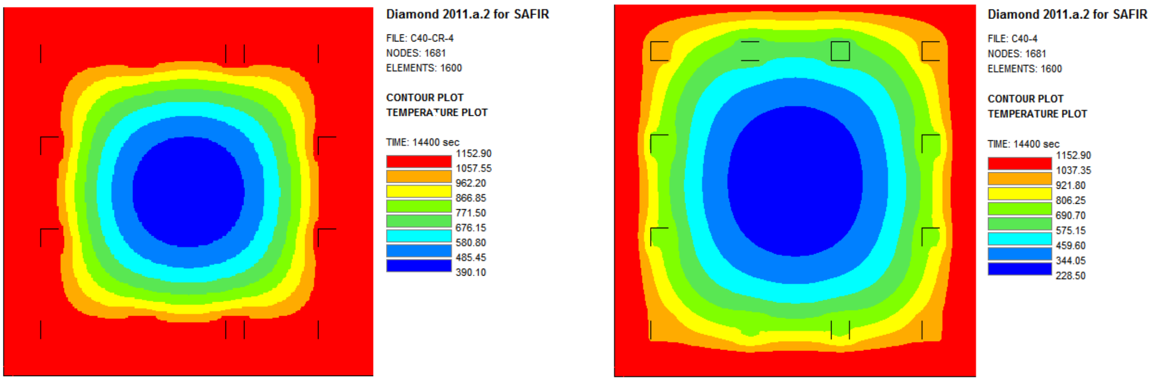
Fig. 10 Pushover curves at three levels IO, LS and CP and extend of plastic hinges at LS level

the interior columns are subjected to high temperature. Meanwhile, only three sides of the beams are exposed to fire, because it is assumed that the top side of the beams is protected by the concrete slab.

5. Results

After performing the sequential analysis, the PEF resistances of the cases are accounted for. As mentioned earlier, the structures are pushed to arrive at different levels of performance as such IO, LS and CP. Using FEMA356 procedure, the accounted-for target displacement is used for performing pushover analysis in the mentioned performance levels. The lateral forces corresponding to the target displacement at every performance level are extracted from SAP2000 program, and are then input to the SAFIR program for performing the sequential analysis. Fig. 10(a) shows the pushover curves for the mentioned performance levels resulting from SAP2000 and used for the sequential analysis in SAFIR. The pushover forces are then used for the sequential analysis. To do this, SAFIR allows a function to be written inside its computing environment that allows the importing of pushover loads extracted from SAP at the target displacement. As plasticity of materials is included in the SAFIR program, degradation of strength and stiffness as a result of lateral loads are therefore automatically considered. Meanwhile, the results of SAFIR analysis showed that extend of the plastic hinges are similar to the assumption made with the lamped plasticity. Fig. 10(b) schematically shows the hinges length in some of selected joints of the frame and at Life Safety level of performance (as an example) resulted from SAFIR.

The final stage of the sequential analysis is to apply a post-earthquake fire to the structure. Two different scenarios are used for the fire analysis – in one case, the undamaged structure is subjected to fire loading, while in the second one, the earthquake-induced structure is exposed to fire load. In this way, in the first case, the fire load follows the gravity loads, but in the second case, the fire load follows gravity and earthquake loads. Fig. 11 shows the temperature distribution after four



(a) Damaged section (b) Intact section

Fig. 11 Distribution of temperature in a column according to ISO 834 after four hours

hours of fire exposure on a damaged and intact section. Figs. 15 and 16 show the fire resistance of the structures before and after the earthquake. The sharp increase and then decrease in PEF analysis is due to the fact that the structures are firstly pushed to a certain level of displacement and then unloaded. The deformed structures are then exposed to fire, as mentioned earlier.

Figs. 12 and 13 show the displacement plotted versus time, which implies the fire resistance of the models in seconds for both scenarios of fire and PEF when the fire is initiated from the first and second floors. Fire resistance is defined as the time at which the displacements, either globally (i.e. is 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 its horizontal asymptote by a 1% error. In other words, a member is considered as failed when it is unable to resist the initially applied gravity loads (Kodur and Dwaikat 2007). As is seen in the figures, regardless of subjecting a structure to fire alone or fire after an earthquake, there is a correlation between the fire resistance rating and the performance levels. Indeed, the fire resistance of the designed frame for IO level of performance is much greater than that of those frames designed for LS or CP performance levels. In addition, the PEF resistance of the designed frames in the second scenario is reduced compared to the first scenario.

Fig. 12 shows that, in the first scenario, the collapse time for PEF-IO level is more than four hours, while it reduces to around 80 minutes for PEF-LS level and around 45 minutes for PEF-CP level. By contrast, when the models are exposed to fire alone, the fire resistance rises considerably compared with the PEF resistance, at more than 5 hours in IO level and approximately 4 hours for LS and CP level of performance. Fig. 13 implies that the fire resistance rating for PEF-IO level is about 4 hours, which is comparable with that accounted for based on the first scenario. It also shows that the PEF resistances for LS and CP levels of performance are around 70 minutes and 40 minutes, respectively, which are significantly lower than the fire resistance in case of fire alone. Overall, the fire resistance of damaged and deformed frames is significantly lower than the fire resistance of undamaged and undeformed frames.

It is worth noting that two types of collapse mechanism were observed during the fire analyses: a global collapse that is mainly governed by considerable lateral displacement of columns; and a local mechanism that depends largely on collapse of beams. While only local mechanism was

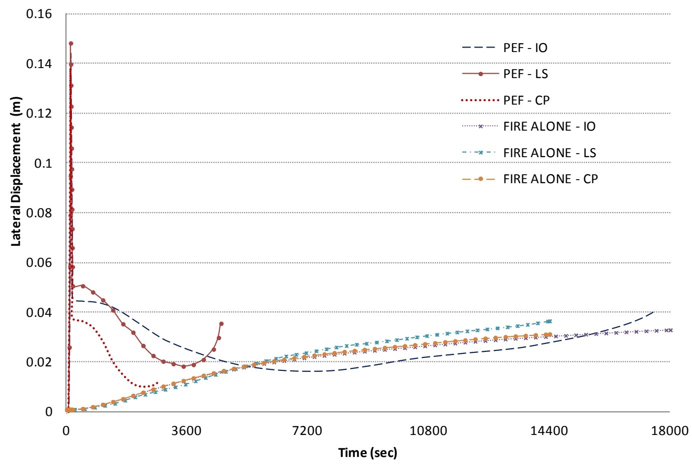


Fig. 12 Fire resistance based on scenario a (second story lateral displacement)

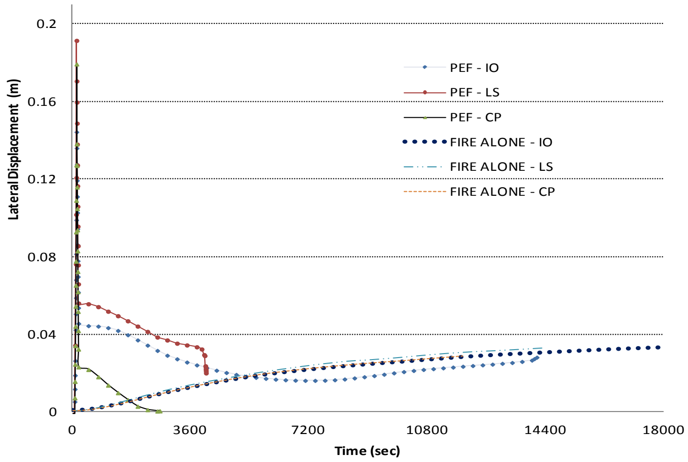
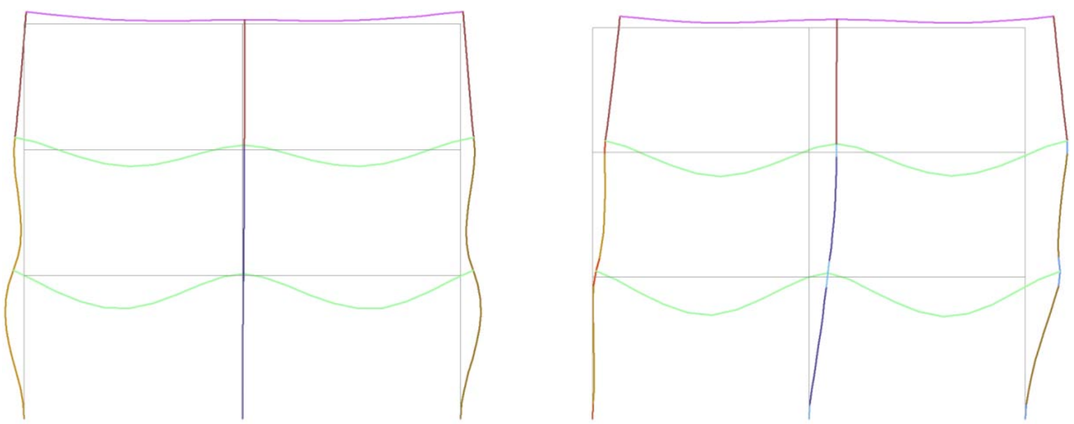


Fig. 13 Fire resistance based on scenario b (third story lateral displacement)



(a) Local collapse (b) Global collapse

Fig. 14 Collapse mechanism for the models observed in case of fire alone, both global and local failures were observed in PEF analysis. Fig. 14 schematically represents the two types of collapse failure as discussed.

6. Conclusions

Post-Earthquake fire is one of the most problematic situations in seismic regions. In this research, sequential nonlinear analysis was proposed for post-earthquake fire. Three RC frames designed for PGA of 0.35g and for three different performance levels, i.e. Immediate Occupancy, Life Safety and Collapse Prevention were selected. The frames were then pushed to the maximum allowable inter-story drift, which was assumed to satisfy the mentioned performance levels. Pushover curves were then extracted for use in subsequent analysis. In addition, two different fire scenarios were assumed. While in the first scenario first and second story are exposed to fire, in the second scenario second and third story are exposed to fire. Sequential loading, consisting of gravity and lateral loads followed by fire loads, was a key aspect of the studies conducted using the SAFIR software. In SAFIR, the P-Δ effect and the residual lateral deformation as well as degradation in stiffness were considered. Defining damaged sections (in terms of spalling in cover and such) in the thermal analysis was an additional factor considered in the fire analysis. The patterns of damage were drawn from the descriptive definition of FEMA356 and other numerical and experimental studies as mentioned earlier, and for buildings designed for different performance levels. Accordingly, the following remarks can be made:

- Sequential analysis is the rational tool for considering the effects of residual deformations from an earthquake, as well as degradation in stiffness and strength.
- In the first scenario, while the fire resistance of the frame designed for PGA of 0.35g in the fire only situation was more than 240 minutes, it considerably declined to around 80 minutes in post-earthquake fire at the Life Safety level and 45 minutes at the Collapse Prevention level. Similarly, the fire resistance of the frame based on the second scenario was about 300 minutes. However, it significantly decreased to around 70 minutes in post-earthquake fire at the Life Safety level and 40 minutes at the Collapse Prevention. Therefore, it can be concluded that structures that have suffered damage from earthquake loads have lower fire resistance than undamaged structures. This can be the result of residual lateral displacements, the degradation in strength and stiffness, or the direct heating of the steel reinforcement as a result of removal of cover, exacerbating the effects of fire.
- Buildings designed for a higher level of performance have more resistance against fire, particularly in case of fire alone. In other words, the stiffness of structures has an important role in the fire resistance rating. This can be seen from Figures 15 and 16, in which frames designed for higher levels of performance have greater fire resistance in a usual fire condition.
- Compared to the first scenario of PEF, i.e. when the fire is located in the first and second floor, the fire resistance of the second scenario, i.e. when the fire is located in the second and third floor, was lower.
- Two types of collapse mechanisms were observed during the fire analyses. While global collapse occurred in the frames subjected to fire after an earthquake, local collapse happened in the fire-only case. The global collapse occurred mostly because of considerable lateral movement of the columns, while the local collapse occurred because of collapse of beams.

- The study performed here was for a certain class of structures. Hence, further studies need to be performed, either numerically or experimentally, on different stories and different fire positions particularly, in order to develop a better understanding of this issue.

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