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Methodology for investigating the behavior of reinforced concrete structures subjected to post earthquake fire

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Abstract. Post earthquake fire (PEF) can lead to the collapse of buildings that are partially damaged in a prior ground-motion that occurred immediately before the fire. The majority of standards and codes for the design of structures against earthquake ignore the possibility of PEF and thus buildings designed with 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 Life-Safety performance level of structures designed to the ACI 318-08 code after they are subjected to two different earthquake levels with PGA of 0.35 g and 0.25 g. This is followed by a four-hour fire analysis of the weakened structure, from which the time it takes for the weakened structure to collapse is calculated. As a benchmark, the fire analysis is also performed for undamaged structure and before occurrence of earthquake. The results show that the vulnerability of structures increases dramatically when a previously damaged structure is exposed to PEF. The results also show the damaging effects of post earthquake fire are exacerbated when initiated from second and third floor. Whilst the investigation is for a certain class of structures (regular building, intermediate reinforced structure, 3 stories), the results confirm the need for the incorporation of post earthquake fire in the process of analysis and design and provides some quantitative measures on the level of associated effects.

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

1. Introduction

It is a universally accepted fact that second to devastating wars; natural disasters are the largest enemy of human kind. For instance, in 2010, more than 230,000 Haitians were killed and more than one million people were injured or become homeless because of earthquakes. In addition to the damage caused by the earthquake itself, post-earthquake events resulting from the shock, such as fire, can potentially create even more significant damage than the earthquake itself. The effects of post-earthquake fire (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 can be caused due to excess structural loads on the building (Chen *et al*. 2004). These excess loads are

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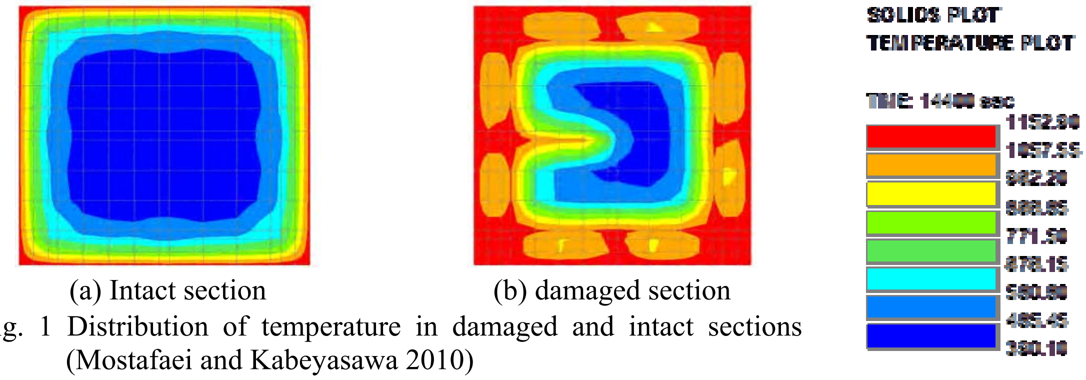
normally the loads that the structures are not designed for, and when combined with the earthquake damage (even if moderate) can lead to the collapse of buildings.

On the other hand, structural elements are normally designed to satisfy a specific level of safety or performance. Clearly, fire is signified as one of the most problematic conditions and therefore, the provision of proper fire safety measures is essential. In general, fire resistance rating is defined as a period of time for 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 that is being designed. Indeed, the purpose is not only to provide adequate time to evacuate trapped people under the fire, but to reduce the possibility of any conflagration (Taylor 2003). Although, typically fire-resistance rating is presented in national building codes such as NRCC 2005 (National Research Council Canada 2005) and IBC 2006 (International Building Code 2006); much of them are provided for fire conditions only and not for the post-earthquake fire. This is an important point because the vulnerability of earthquake-damaged structures exposed to post earthquake fire is much more 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, experience from past earthquakes has proven that both active and passive fireproofing systems such as sprinklers and fire control systems may become seriously damaged as a result of which, the fire resistance capability will decrease considerably. The fire resistance is defined as a time at which the element is unable to resist applied loads such as gravity loads (Kodur and Dwaikat 2007).

Thus, evaluation of buildings' performance under PEF condition is an important issue that has to be scrutinized carefully. PEF resistance of a building is dependent on various factors including the deformed geometry and degradation in stiffness resulted from earthquake (Zaharia and Pintea 2009). In reinforced concrete structures, in addition to the mentioned factors, the effects of level of damage including tensile cracking, removal of rebars' cover and compressive crushing on PEF resistance has to be considered as well. Assuming ductile behavior of RC elements, a typical moment-curvature relation can be idealized to separate stages (Kwak and Kim 2010). 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 reduce the PEF resistance (Ervine *et al*. 2012).

Investigating PEF resistance, Della Corte *et al*. (2003) investigated unprotected steel moment-resistant frames and their response subjected to fire following an earthquake. Assuming elastic perfectly plastic behavior for steel and considering P-Δ effect with P from gravity loads and Δ from the earthquake, the fire resistance was evaluated. Ignoring of the degradation of stiffness in Della Corte *et al*.'s work 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 various return periods of ground-motion; 2475 years return period and 475 years. The seismic response of the structures 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 475 years return period sustained notable inter storey drift. They then performed a fire analyses on both frames, which confirmed that the fire resistance of the structures considering their deformed state under earthquake is notably lower than 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 structure with shear wall. The model was first subjected to the Kobe 1995 earthquake on a shaking table. The damage sustained by the



(a) Intact section (b) damaged section

Fig. 1 Distribution of temperature in damaged and intact sections (Mostafaei and Kabeyasawa 2010)

structure was then quantified by the method of observation as well as a developed method called Axial-Shear-Flexure Interaction (ASFI) (Kabeyasawa and Mostafaei 2007) and then was used in a numerical thermal analysis to find the temperature rise in and around both the seriously damaged and the intact sections subjected to fire. Fire loading was then applied to the damaged structure considering the effect of concrete degraded compressive strength. The results showed that the ability of the structure to sustain gravity loads in the damaged components is considerably lower in comparison to the intact components. Although compressive strength of concrete plays an important role on the overall fire resistance, other factors such as P-Δ effect and the changes in the modulus of elasticity have to be considered as well in order to improve accuracy.

In the same year, Faggiano and Mazzolani (2011) investigated steel structures exposed to post earthquake fire. They performed coupled analyses consisting of both earthquake and fire. Based on the FEMA356 procedure, Faggiano et al developed a method for evaluating the performance of buildings subjected to earthquake, suggesting fire performance levels for various conditions of fire. Clearly, in the couple analyses, 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 on PEF resistance. Recently, in 2011, Ervine *et al*. (2012) conducted an experimental and a 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 changes of thermal propagation inside the section. The results showed that minor tensile cracking would not considerably change the heat penetration inside the section. They concluded that fire resistance of the intact specimen and the minor damaged specimen are roughly identical. However, exposing the rebar directly to fire, i.e., in case of crushing of cover, considerably changes both the thermal and the structural behavior of specimen (Ervine *et al*. 2012). Another study on this issue is currently being undertaken by Bhargava *et al*. (Bhargava *et al*. 2010) on the fire resistance of an earthquake damaged RC frame. A nearly full-scale portal frame was firstly loaded by the relevant gravity loads and then subjected to a cyclic lateral load based on the Indian standard in a quasi-static fashion. Moreover, load-controlled 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 (ABAQUS 2008) for making a comparison between the test and the analytical results. The results showed a good conformity with FEMA356 descriptive definition of damage levels at various performance levels as such Immediate Occupancy (IO) and Life Safety (LS). They suggested that the results of quasi-static cyclic test can be used for the subsequent fire analysis.

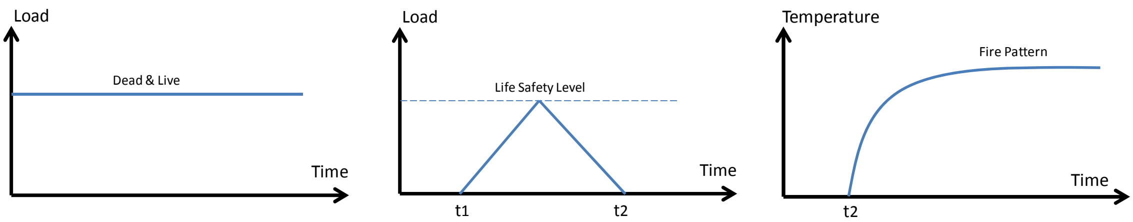
Inspired by the abovementioned studies and FEMA356 performance level definition, in this study, a series of more numerical investigations are carried out on the PEF resistance of conventional buildings designed for Life Safety performance level. The study here includes a sequential analysis comprising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

2. Methodology

2.1 Sequential analysis

In order to properly analyze a structure under post-earthquake fire, a sequential analysis is used. Fig. 2 schematically shows the stages of a nonlinear sequential analysis. The first stage of loading is the application of gravity loads, which are assumed to be static, uniform and constant throughout the analysis. A pseudo earthquake load then follows in a push over fashion reaching its maximum value and returning back to zero in a short time. Clearly, during this time, gravity loads are also applied. The pattern chosen for the pseudo earthquake load is similar to pushover analysis with a difference that the structure is unloaded after reaching certain level of load. Here, it is assumed that the maximum level of earthquake load corresponds to life safety level of FEMA356. This assumption is in line with the seismic design philosophy in which the performance level of structures shall not exceed life safety level when subjected to the “Design earthquake”. Therefore, the structure is pushed to this life safety level and then is unloaded. It should be noted that no dynamic effects are considered in this study. Finally, as can be seen in Fig. 2, the fire load is applied to the structure after the lateral load is unloaded. Prior to fire loading, properties of structures are set to the reference temperature, but during fire, mechanical properties change with temperature.

In this research, incorporating the pattern shown in Fig. 2 and using SAP2000 for the pushover analysis and SAFIR for the sequential analysis (Franssen 2011), sequential seismic and fire analyses are performed.



(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. 2 Stages of the sequential analysis

2.2 Material nonlinearity

Fiber element is a most promising element for the nonlinear analysis of reinforced concrete members. Many researchers have developed finite element formulations for this element. The model accounts for material nonlinearities in rebar steel and concrete. A fiber beam element is made up of a series of sections along the element length, whose number and locations depend on the integration scheme. The constitutive relation for the section is not specified explicitly, but it 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 axial force-deformation relationships. Concrete can be modeled into two regions. One is the core that is confined and the other, the cover that is unconfined. In SAFIR program, the stress-strain relationship for concrete and rebar steel are implemented according to Eurocodes. Using fiber elements of SAFIR, the spread of plasticity can be modeled easily. Unlike lumped plasticity, in the fiber element model, plasticity is spatially distributed in both the cross section and along the member.

2.3 Pushover analysis

The static pushover analysis is one of the nonlinear static methods that are used for analyzing structures subjected to seismic loads. This method is becoming a popular tool for seismic performance evaluation of existing and new structures (Cardone *et al*. 2009). In this method, 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 structure. In this study, a vertical distribution of loads proportional to the shape of the fundamental mode in the direction under consideration is used. Fig. 3 shows a structural frame subjected to a lateral load pattern and a typical base shear versus top story displacement.

Building performance is a combination of the performance of both structural and nonstructural components. According to FEMA356, the structural performance level is divided into four main categories: Operational (O), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), which represent states of minor damage to notable damage as shown in Fig. 4.

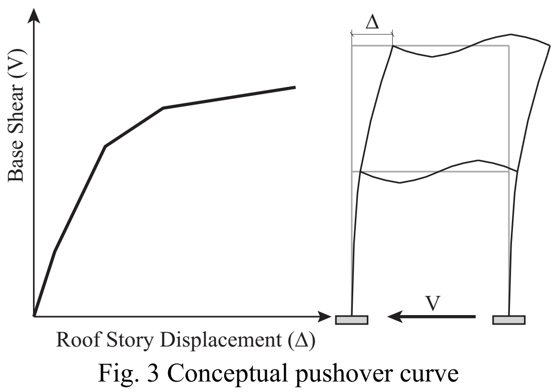
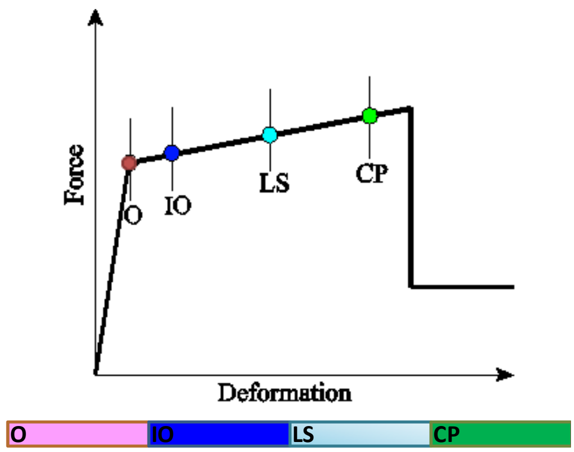


Fig. 3 Conceptual pushover curve

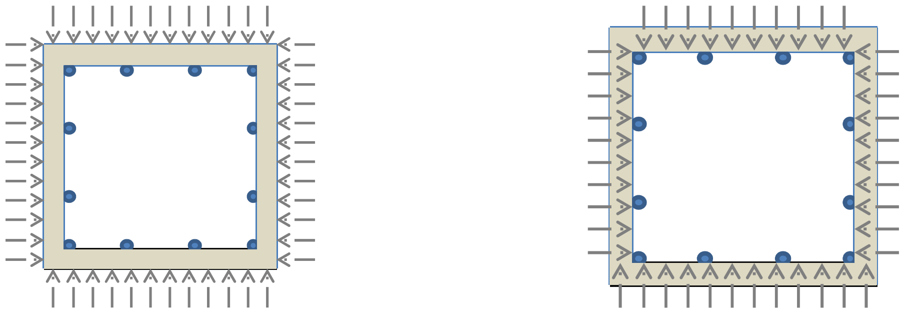


O : Operational ; **IO** : Immediate Occupancy ; **LS** : Life Safety ; **CP** : Collapse Prevention

Fig. 4 Conceptual plastic hinge states

2.4 Fire frontiers

As previously mentioned, in this study, the Life Safety performance level is considered for the seismic analysis prior to fire loading. Clearly, in structures that experience plastic deformations, residual deformation remains in the structure and thus, the structures do not return to their initial condition. Using the definition of lumped plasticity, the potential locations of high plasticity are introduced by plastic hinges in SAP2000 (SAP2000-V14 2002).The moment-rotation behavior of each plastic hinge follows FEMA definitions as shown in Fig. 4. These definitions 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 (Ervine *et al*. 2012). In FEMA356, it is assumed that in the Life Safety performance level, extensive damage is observed in beams, while ductile columns sustain spalling of their cover. While the interpretation of the “extensive damage” is left to the reader in FEMA, the assumption here has been the further penetration of the fire front to behind the outermost reinforcing bars unlike columns where the fire is assumed to penetrate to the edge of the concrete cover as it spalls according to FEMA. The dotted lines and the arrows in Fig. 5 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 FEMA356 code, the Japan Building Disaster Prevention Association (JBDPA) and an experimental study performed by Bhargava et al. While none of the aforementioned references differentiate in between the beam and the columns 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. While at the Life Safety level of performance, the cover of columns is spalled, it states that more damage is observed in the beams. In a similar manner but rather in terms of quantity than quality, the experimental results of Bhargava et al (Bhargava *et al*. 2010) on a nearly full scale RC frame that was performed to find the level of damage when the frame is pushed to a certain level of displacement, shows that while at a roof drift ratio of 1.37%, the flexural cracking was observed.



(a) Intact sections (b) Life safety

*Note : the arrows show fire frontiers

Fig. 5 Applied fire frontiers on the damaged sections

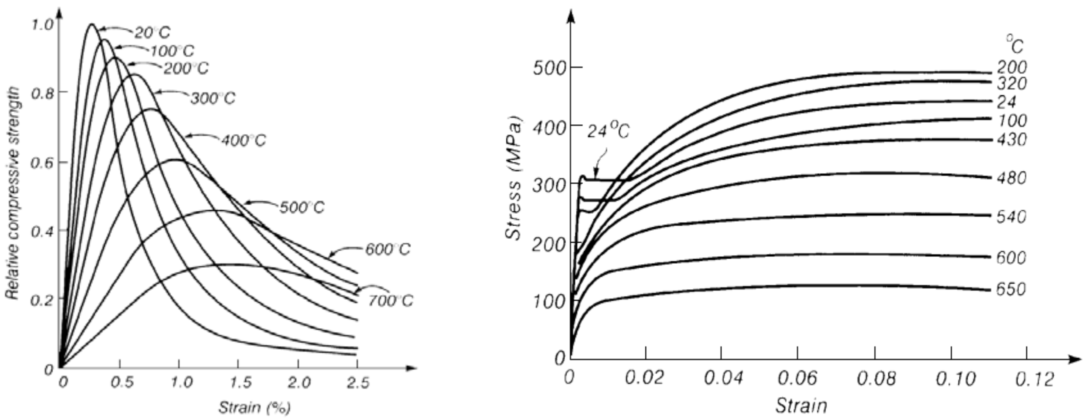
(which is corresponding to the drift ratio in Immediate Occupancy level of performance), at 2.11 % drift ratio- corresponding to the drift ratio of Life Safety level of performance- spalling and wide cracks in columns and beams were observed. The study does not reveal any differences in between the columns and the beams. JBDPA presents similar observations (Meada and Kang 2009).

2.5 Reinforced concrete behavior under the effect of fire

Materials' thermal and mechanical characteristics considerably change when they are exposed to fire as a result of high thermal stresses (Kwasniewski 2011). The thermal stresses are not necessarily uniform across the material and the differential 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 concrete cover, however, causes more thermal propagation penetrating at a quicker rate with serious negative outcomes. It is apparent that this can be worse if a previously damaged member, as a result of earthquake loading for example, experiences high-temperature, because the fire resistance of the seriously damaged members is much less than that of the intact ones. In other words, the more members are damaged, the shorter is the time to collapse during the PEF. Compared to reinforcement steel for which the critical temperature is around 5000C at which the steel ultimate strength decreases of 50%, for the concrete, the critical temperature is about 3000C (Youssef and Moftah 2007) .The critical temperature is defined as a degree of temperature beyond which the values of strength are considerably reduced. Fig. 6 shows stress-strain relationship in hot roll bars and concrete at high-temperatures as developed by Eurocodes 2 and 3.

2.6 Fire patterns

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 are established either using parametric fires called time-temperature curves such as those mentioned in ISO 834, and ASTM E119 (based on experiment and tests) or using natural fire which rely mainly on the volume of the



(a) Concrete (b) Hot-rolled bar

Fig. 6 Stress-strain relationship at different temperature (Minson 2006)

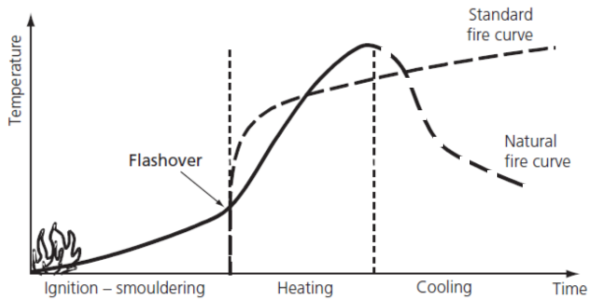


Fig. 7 Temperature-time curve for fully developed fire (Lennon 2011)

produced gas by the combustible materials in a covered space such as those stated in SEI and ASCE. Both models, i.e., the time-temperature curve and the natural fire, are represented assuming a fully developed fire as schematically shown in Fig. 7. Moreover, the produced temperature resulted from burning combustible materials are lumped into one single parameter (Drysdale 2011).

The cooling phase in Fig. 7 is based on the assumption that after a while, either air or combustible materials become less and less available and thus, temperature or fire load decreases.

To calculate the fire resistance of the selected models on this study, a computer program written based on Finite Element Method (FEM) called SAFIR is employed. The program performs non-linear analyses on one, two or three dimensional structures in which both geometrical and material non-linearity 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. 8.

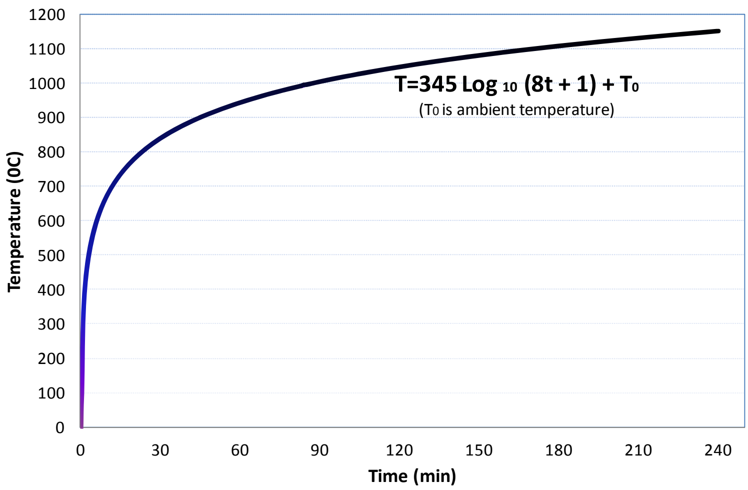


Fig. 8 Fire pattern according to ISO834

3. Case studies

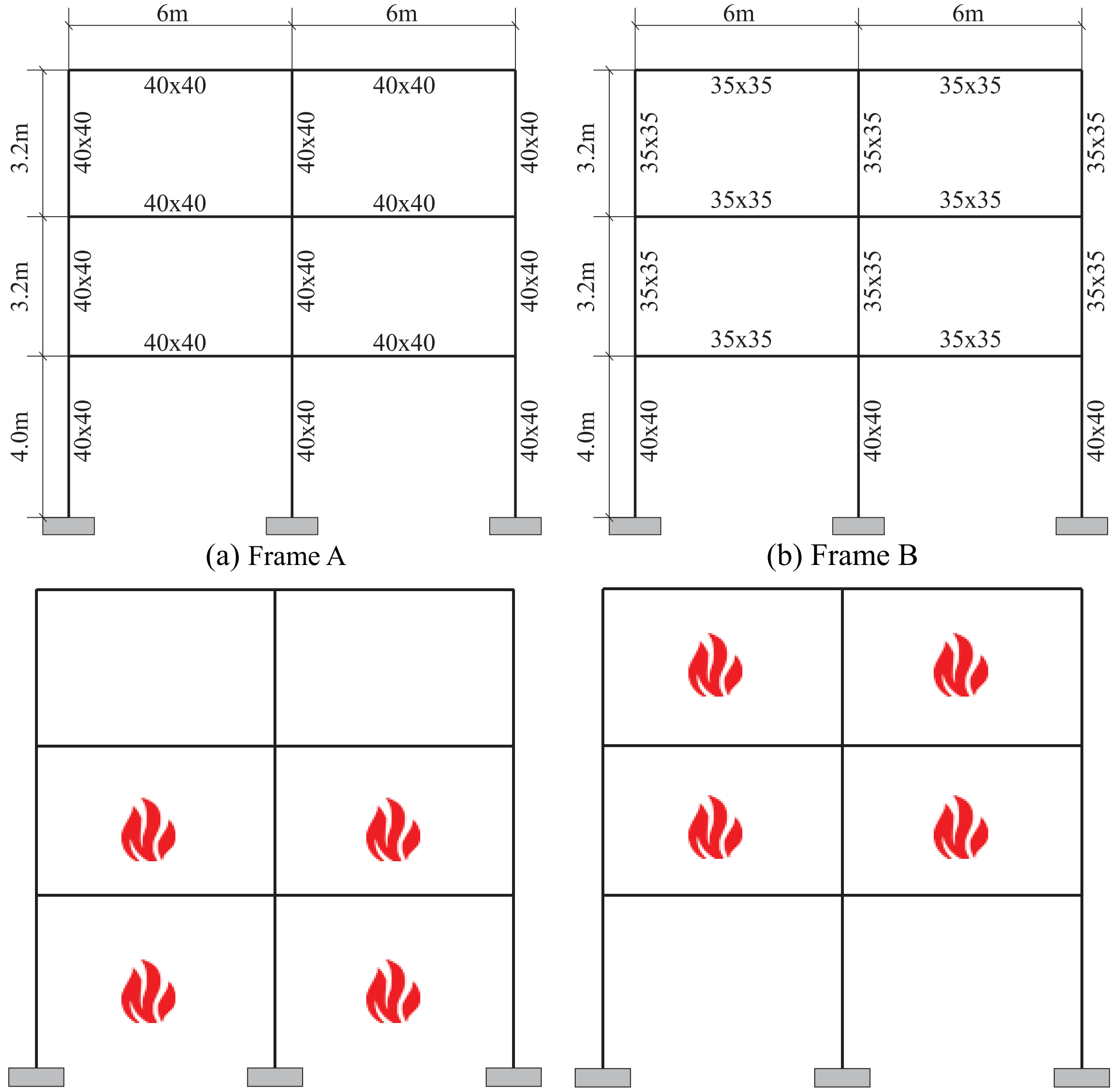
3.1 Materials

Two reinforced concrete frames (A and B) with identical geometry but designed for two different earthquake levels are selected. The selected cases are loaded and designed based on ACI 318-08 code and the properties of the designed frames are presented in Figs. 9(a) and (b). The structures are made of normal strength concrete with compressive strength of 25 MPa and longitudinal and transverse reinforcing bars with a yield stress of 400 MPa. Both frames are dimensioned for the same load combinations of 8.0 kPa for dead load and 2.5 kPa for live load. Combination of total dead load and 20% of live load is used to find the 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's (Park and Paulay 1975) formula is used as a simple but accurate method, where LP = 0.5 H and H is the section height.

The models are exposed to standard fire (ISO834 curve, without decay) and two different fire scenarios for 4 hours as shown in Figs. 9(c) and (d). For the thermal analysis, it is assumed that the concrete moisture content is 2%. Moreover, the thermal expansion coefficient of rebar and concrete are assumed as 12 × 10-6/°C and 10×10-6/°C, respectively. The 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 undamaged frames, i.e., before occurrence the ground-motion. It must be noted that while exterior side of external columns is not exposed to fire, all sides of interior columns are subjected to high-temperature. Meanwhile, only three sides of beams are exposed to fire, because it is assumed that the top side of the beams is protected by the concrete slab.

4. Results

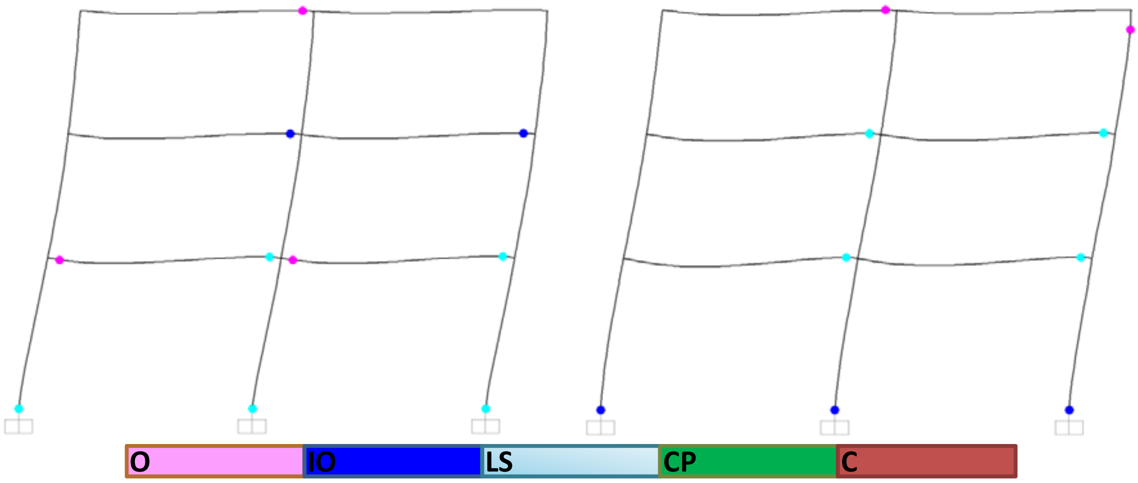
Sectional dimensions of beams and columns for the considered structures were given



(c) The fire initiated from 1st and 2nd floors (d) The fire initiated from 2nd and 3rd floors

Fig. 9 Geometric properties of selected cases and fire scenarios

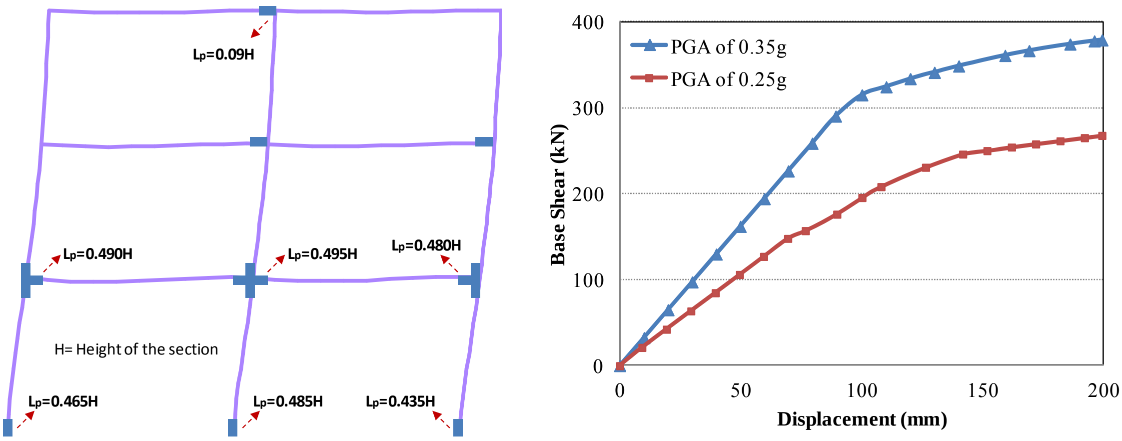
previously in Fig. 9. All material properties were also introduced in previous sections. The sequential analysis comprises of three main stages that are gravity loading, followed by seismic pushover analysis, and finally post earthquake fire. In seismic analysis, the structure is subjected to a monotonically increasing lateral load to attain a certain level of drift. According to seismic codes philosophy, it is expected that ordinary structures like residential ones remain at life safety level of performance. According to ASCE 7-10, the drift limit of 2% is used to control the lateral displacement of structures with the first period greater than 0.70 second. FEMA assumes that controlling this limit adequately provides for the life safety performance of a structure. The first modal period of the selected cases is greater than 0.70 seconds, thus, the selected structures are



(a) PGA of 0.35g (b) PGA of 0.25g

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

Fig. 10 Plastic hinges states of the structures resulted from pushover analysis



(a) Frame A (b) Pushover results

Fig. 11 Extend of plastic hinges in Frame A and pushover curves in Frame A and B

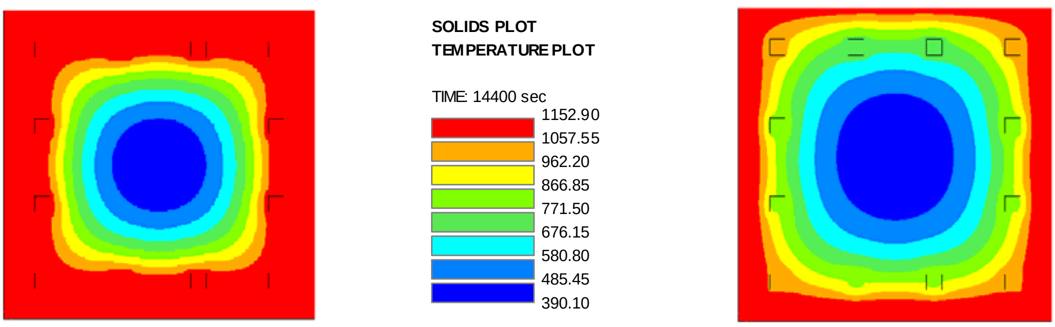
pushed to 0.02 drift. SAP2000 program is used for utilizing nonlinear pushover analysis.

Moreover, FEMA 356 procedure is used to define the hinges for the nonlinear pushover analysis. Fig. 10 Mshows the state of hinges resulting from the pushover analysis. While the colored dots in Fig. 10 represent the state of hinges, the results show that none of the hinges have gone beyond the LS level of performance. This was expected, as the design aim of seismic design codes is to maintain the Life Safety level for regular buildings. The figure also shows that the frame subjected to the PGA of 0.35 sustain more damage than that subjected to the PGA of 0.25. This was also an expected result, as the severity of earthquake loads directly affects the level of damage.

The lateral forces correspond to 2% lateral drift are extracted from the SAP2000 program, and are then cast in the SAFIR program to perform sequential analysis. The damage level defined by FEMA356 can be observed in damaged members with color coded plastic hinges. The mentioned results are then used for the fire analysis, which is performed by the SAFIR program. 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 lumped plasticity. Fig. 11(a) schematically shows the hinges length in some of selected joints in Frame A (as an example) resulted from SAFIR. Fig. 11(b) shows the pushover curves used in SAFIR program.

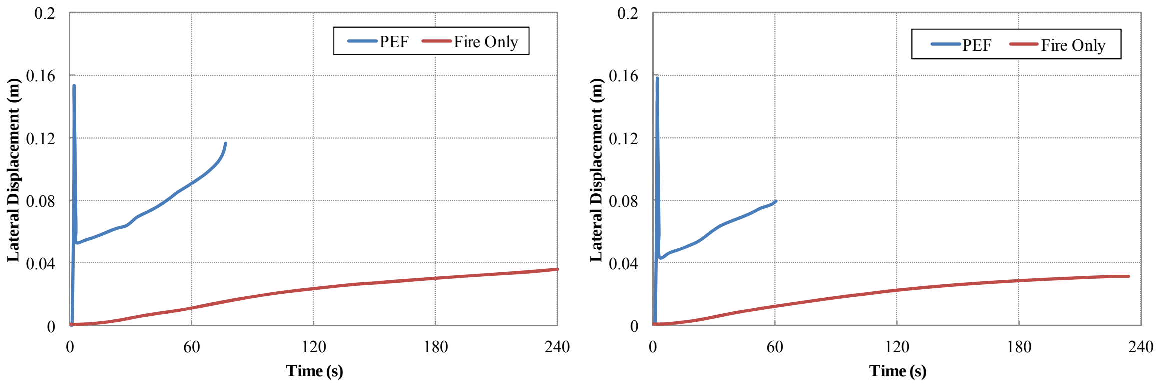
The final stage of sequential analysis is to apply a post earthquake fire on the structure. The two different mentioned scenarios are used for the fire analysis, for both damaged and undamaged frames. In one case, the undamaged structure is subjected to fire loading, while in the second one, the damaged structure is exposed to fire load. In other word, in the first case, the fire load follows the gravity loads, but in the second one, the fire load follows gravity and earthquake loads. Fig. 12 shows the temperature distribution after four hours of fire exposure on a seriously damaged and intact section. In addition, Figs. 13 and 14 show the fire resistance of the structures for both before and after the earthquake. The sharp increase and then decrease in PEF analysis is due to, 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.

Fig.13 shows 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 first and second floors. The fire resistance is defined as a 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 word, a member is considered as failed when it unable to resist the initially applied gravity loads (Kodur and Dwaikat 2007). Using this definition, vulnerability of the designed frame for the PGA of 0.25 g is more than that PGA of 0.35 g. While the PEF resistance of the weaker frame is around one hour, the stronger frame response to PEF is more than 90 minutes. However, both of the undamaged frames have relatively identical fire resistance after four hours exposure to high-temperatures. The graphs also show that the fire resistance rating for stronger frame in regular situation is even more than the weaker one.



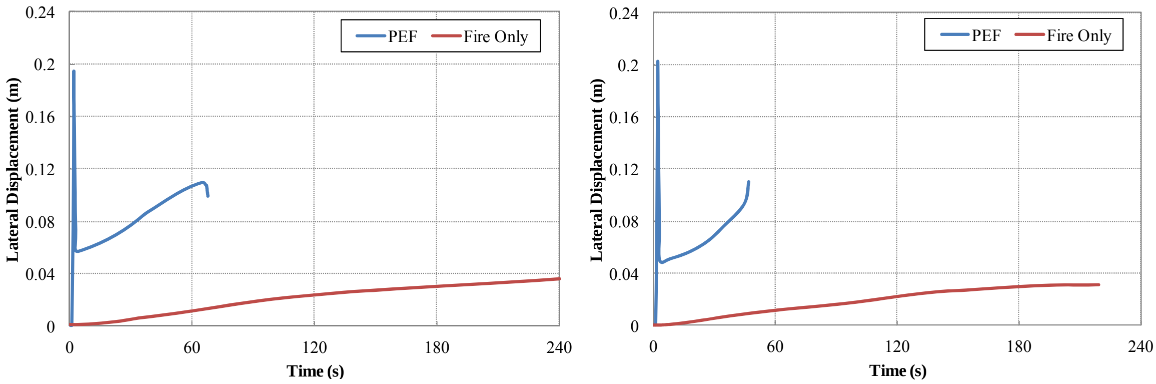
a) damaged section b) intact section

Fig. 12 Distribution of temperature in a column according to ISO 834 after four hours



(a) 2nd story lateral displacement (PGA of 0.35 g) (b) 2nd story lateral displacement (PGA of 0.25 g)

Fig. 13 Fire resistance based on scenario 1 (fire initiated from first and second floors)

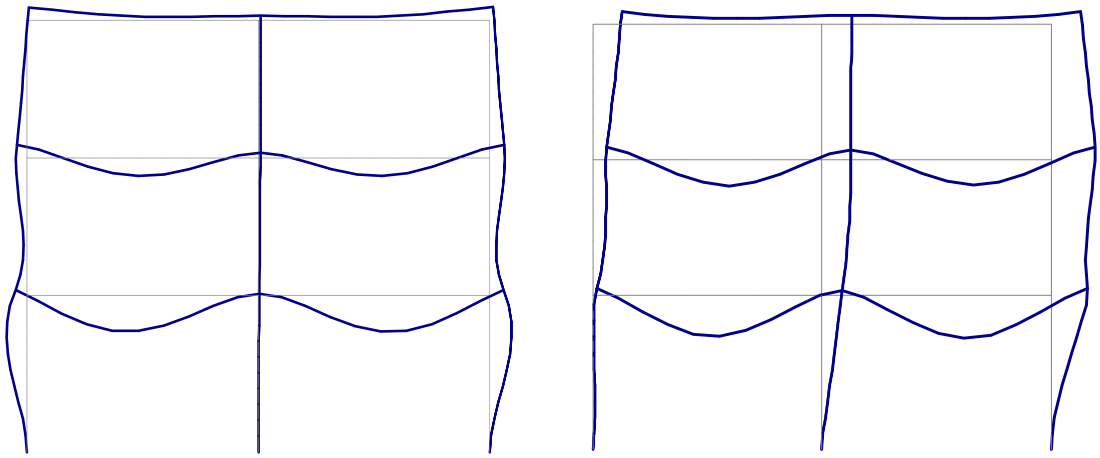


(a) 3rd story lateral displacement (PGA of 0.35 g) (b) 3rd story lateral displacement (PGA of 0.25 g)

Fig. 14 Fire resistance based on scenario 2 (fire initiated from second and third floors)

On the other hand, the fire resistance of PEF scenario, for both designed frames when the fire is initiated from second and third floors reduces compared to the fire initiated from first and second floors. Fig. 14 shows collapse mechanism of the frames for both situations before and after earthquake. The figure confirms that the fire resistance of the weaker frame is lower than the stronger one, roughly less than 60 minutes for the former and around 75 minutes for the latter. The figure also shows that the fire resistances of the undamaged frames are considerably more than the damaged frames.

In addition, for both structures and under both fire scenario, the designed frame for PGA of 0.35 g (stronger earthquake) shows higher fire resistant rating than the corresponding frame designed for the lower earthquake level, i.e., PGA of 0.25 g. Thus, it can be concluded that the earthquake design level has a direct consequence on the fire resistant rating of a frame. It is also worth mentioning that while the earthquake-induced frames represent a global failure mode, which means the frame fails because of considerable lateral movement in columns, the undamaged frames show the local collapse mechanism, mostly because of collapse of beams. Both types of collapse have schematically been shown in Fig. 15.



(a) Fire only (b) Post earthquake fire

Fig. 15 Collapse mechanism for the models, before and after earthquake

5. Conclusions

Post-earthquake fire is one of the most problematic situations in seismic regions. In this research, sequential non-linear analysis is proposed for post earthquake fire. Two three story RC frames were selected and designed at two different earthquake levels, PGA of 0.25 g and 0.35 g. The structures were then pushed to the maximum allowable inter-story drift, which is assumed to satisfy the Life Safety performance level. Pushover curve was then extracted for use in subsequent analysis. In addition, two different fire scenarios were assumed. While in the first scenario 1st and 2nd story are exposed to fire, in the second scenario 2nd and 3rd story are exposed to fire. Sequential loading consisting of gravity and lateral loads that was followed by fire was a key aspect of the study 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 - in the thermal analysis, in addition, was another remark considered in the fire analysis. The patterns of the damaged members were inspired from the descriptive definition of FEMA356 for building designed for the Life-Safety level of performance and the results of previous studies. Accordingly, the following remarks can be made:

- The sequential analysis is the potential tool to consider the effects of residual deformations from an earthquake as well as degradation in stiffness and strength.
- While the fire resistance of the frame designed for PGA of 0.35 g in the fire only situation was around 240 minutes, it considerably declined to around 90 minutes in post-earthquake fire. Similarly, the fire resistance of the frame designed for PGA of 0.25 g was about 210 minutes. However, it significantly decreased to around 60 minutes for post-earthquake fire. Therefore, and as expected, it can be concluded that structures suffered damage from earthquake loads have lower fire resistance than the undamaged structures. This can be the result of residual lateral displacements, the degradation in strength and stiffness, the direct heating of the steel reinforcement as a result of removal of cover that exacerbate the effects of fire.
- The buildings designed for stronger earthquake have more resistance against fire even in

case of fire alone. In other word, the stiffness of structures has an important role in the fire resistance rating.

• Two types of collapse mechanisms were observed during the fire analyses. While the global collapse occurred in the frames subjected to fire after an earthquake, the local collapse happened for the fire only case. The global collapse is mostly because of considerable lateral movement of the columns, while the local collapse occurs because of collapse of beams.

• Whilst the investigation performed here was for a certain class of structures, further studies need to be performed either numerically or experimentally on different levels of performance and structures subjected to post-earthquake fire in order to develop a better understanding of the issue.

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