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The effect of earthquake damage on fire resistance of reinforced concrete portal frames

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ABSTRACT: 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. An investigation based on sequential analysis inspired by FEMA356 is performed here on the Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance levels of a portal frame, after they is pushed to arrive at a certain level of displacement corresponding to the mentioned performance level. This investigation is followed by a fire analysis of the damaged frames and when only beam or columns are exposed to fire, controlling the time taken for the damaged frame to failure. As a benchmark, a fire-alone analysis is also performed. The results show that while there is minor difference between the fire resistances of the fire-alone analysis and the frame pushed to the IO level of performance, a significant variation is seen between the fire-alone analysis and the fire resistance of the other performance levels, i.e. LS and CP. The results also show that exposing only the beam to fire results in a higher decline of the fire resistance, compared to exposing only the columns to fire.

1 INTRODUCTION

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 (Borden 1996; Taylor 2003). As the post-earthquake fire loading has not been considered in the available codes, theses excessive loads may thus lead to rapid collapse of the buildings. On the other hand, using the philosophy of design based on performance (California Seismic Safety Commission 1996), structural elements are designed to satisfy various levels of performance, such as 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 the assignment of each structure to one of several "Seismic Use Groups" (Figure 1). 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 (FEMA356,2000). Most buildings in urban areas, however, are designed to meet the Life Safety level of performance. To meet this objective, limiting the value of drift to around 2% is recommended. 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 of the structural components.
Understanding the structural behavior of buildings becomes more important when a fire occurs after a seismic event. 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 (König 2005; Kodur and Dwaikat 2007). Although typically, fire-resistance ratings are presented in national building codes, such as NRCC 2005(National Research Council Canada 2005) and IBC 2006(International Building Code 2006), many of them provide only for fire condition and not for post-earthquake fire. This is important as the vulnerability of earthquake-damaged structures exposed to PEF is much more than those exposed to fire alone. This is because earthquake excitation may produce residual lateral deformations as well as residual stresses on the members(Mousavi, Kodur et al. 2008). Therefore, evaluation of a building's performance under PEF is essential, requiring careful scrutiny.

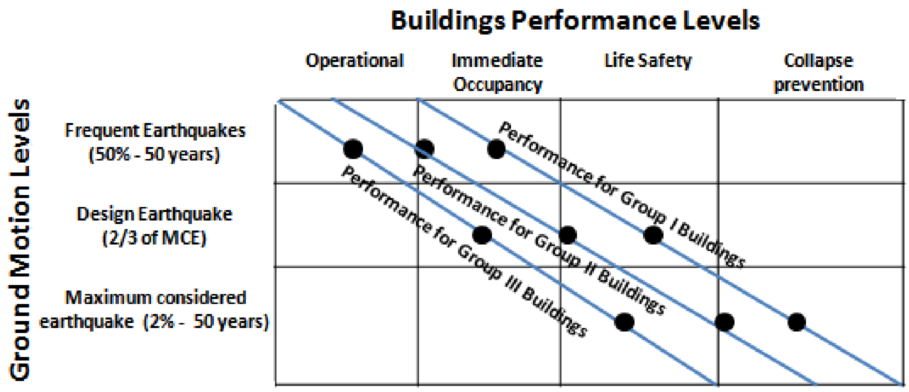


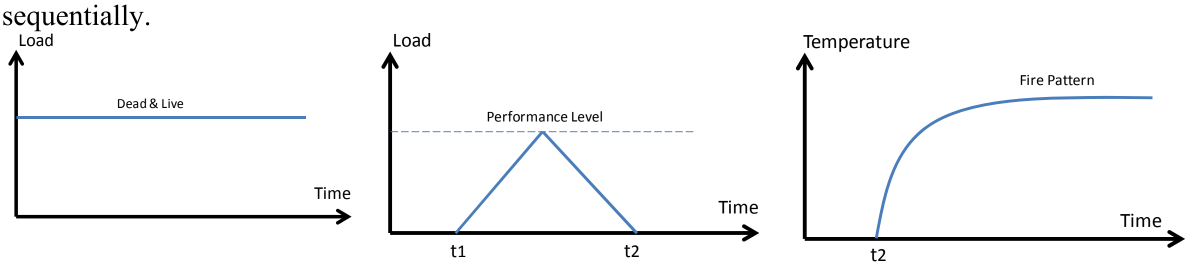
Figure 1: Building performance levels versus earthquake severity (FEMA450 2003)

2 PAST STUDIES

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 P-Δ effect with P from gravity loads and Δ from the earthquake, the fire-resistance rating was found using numerical methods. Further study of steel frames was carried out by Zaharia and Pintea [15]. They investigated two different steel frames, designed for two return periods of ground motion. 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 lower than that of structures that do not have any history of deformation prior to the application of the fire. In 2010, Mostafaei and Kabeyasawa (Mostafaei and Kabeyasawa 2010) investigated the PEF resistance of reinforced concrete structures with shear wall. Their 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) 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 in order to consider the effect of concrete's degraded compressive strength. In 2011, Ervine et al. (Ervine A., Gillie M. et al. 2011) conducted an experimental and numerical study of a reinforced concrete element subjected to conventional loads followed by a fire load. After 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 loading in order to find the effect of the 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 were roughly identical (Ervine, Gillie et al. 2012). However, exposing the rebar directly to fire, e.g. in the case of crushing of the cover, changes both the thermal and the structural behavior of the specimen considerably.

3 METHODOLOGY

Sequential analysis is a method for considering the effect of both earthquake and fire on a structure. Figure 2 schematically shows stages of the nonlinear sequential analysis. The first stage of 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. 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. Therefore, the structure is pushed to these levels and then unloaded. In this study, SAFIR software (Franssen 2011) is used to perform the seismic and subsequent fire analyses sequentially.

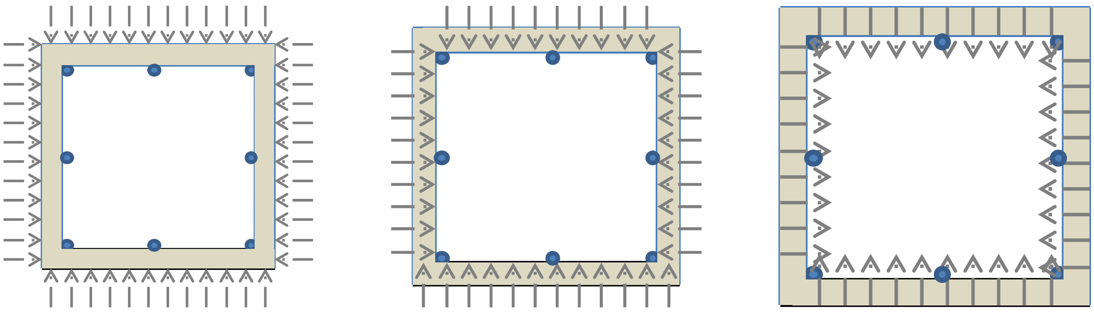


a) Gravity loading b) Earthquake loading c) Fire loading

Figure 2: Stages of the sequential analysis

3.1 Pushover analysis

In the pushover method, using a specific load pattern, the structure is pushed to a value of displacement called the target displacement. The target displacement serves as an estimate of the global displacement that the structure is expected to experience in a "Design Earthquake". Using the definition of lumped plasticity, the potential locations of plasticity are introduced by plastic hinges.The moment-rotation behavior of each plastic hinge follows FEMA definition. 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.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, but rebars also need to be considered totally exposed in the PEF analysis. (Figure 3)



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

Note: the arrows show fire frontiers

Figure 3: Schematically applied fire frontiers on the sections in various performance levels

3.2 Reinforced concrete behavior under the effect of fire

Materials thermal and mechanical characteristics change considerably when exposed to fire, which in many cases produce high levels of thermal stress (Kwasniewski 2011). The reinforcement bars have high thermal conductivity, but they are generally protected by the concrete cover. Cracking or crushing of the concrete cover, however, causes more thermal propagation to penetrate at a quicker rate with serious negative outcomes. It is apparent that this penetration can be worse if a member that has been previously damaged (for example, as a result of earthquake loading), experiences high temperature. It is worth mentioning that spalling of concrete cover under fire exposure is an important issue, which occurs suddenly, violently, is brittle and may lead to a significant decrease in the load-bearing of the structure (Debicki, Haniche et al. 2012). The thermal spalling, nevertheless, is more important in the elements with more than 4-5 cm cover (Majorana, Salomoni et al. 2010) or made of high-strength concrete (HSC) (Kodur 2005) with particles smaller than the cement grains (micro silica, for example) and moisture content of more than 3-4% (Hertz 2003; Hertz and Sørensen 2005). As for the elements of this study, which are made from normal-strength concrete (NSC) with the cover of 4 cm and moisture of 2%, thermal spalling is not considered.

3.3 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 have been established either using parametric fires called “time-temperature curves”, such as those mentioned in ISO 834 (ISO 834 International Standard 1999) and ASTM E119 (ASTM 2006) (based on experiment and tests), or using “natural fires” which rely mainly on the volume of gas produced by the combustible materials in a covered space, such as those stated in SEI and ASCE (ASCE 2006). To calculate the fire resistance of the selected model in this study, a computer program called SAFIR is employed. This program performs nonlinear analyses on one, two or three dimensional structures in which both geometrical and material nonlinearity are taken into account. Structures that have been 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 Figure 4.

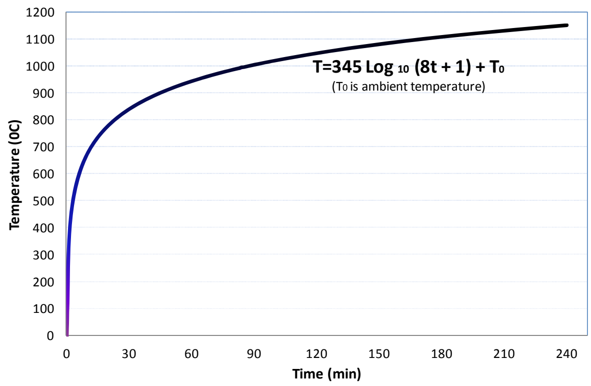


Figure 4: Fire pattern according to ISO834

4 CASE STUDY

A portal reinforced concrete frame designed based on ACI 318-08 code is pushed to arrive at different lateral drifts, corresponding to IO, LS and CP levels of performance as shown in Figure 5. The frame is made using normal-strength concrete with compressive strength of 25MPa and longitudinal and transverse reinforcing bars with yield stress of 400MPa. The frame is dimensioned for a height of 3.0 m and load combinations of 8.0kPa for dead load and 2.5kPa for live load. The combination of 100% dead load and 20% live load is used to find the required mass for calculating the earthquake load (ACI318 2008). Furthermore, the frames are exposed to standard fire and two different situations of fire: only beam and only columns. For the thermal analysis, it is assumed that the concrete moisture content is 20 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 order to improve our understanding of the behavior, the fire analyses is also performed for the undeformed frame.

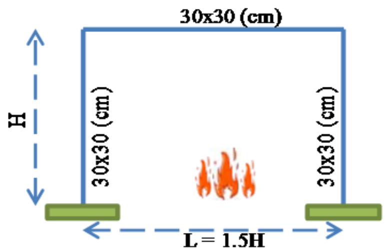
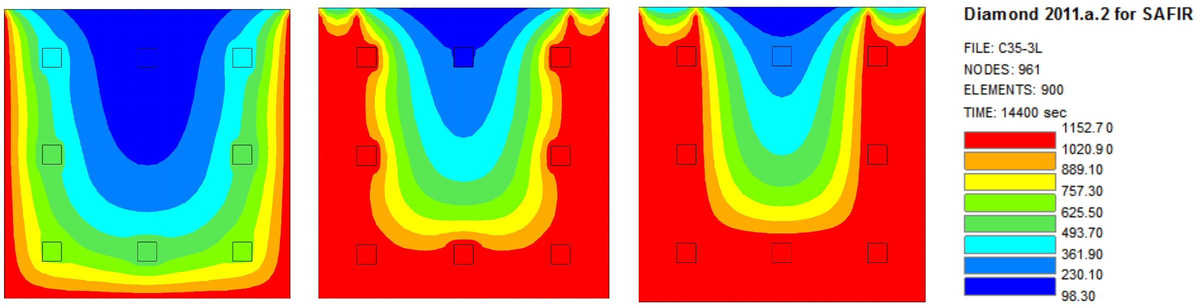


Figure 5: Geometric properties of the frame, H=3.0 m

5 RESULTS

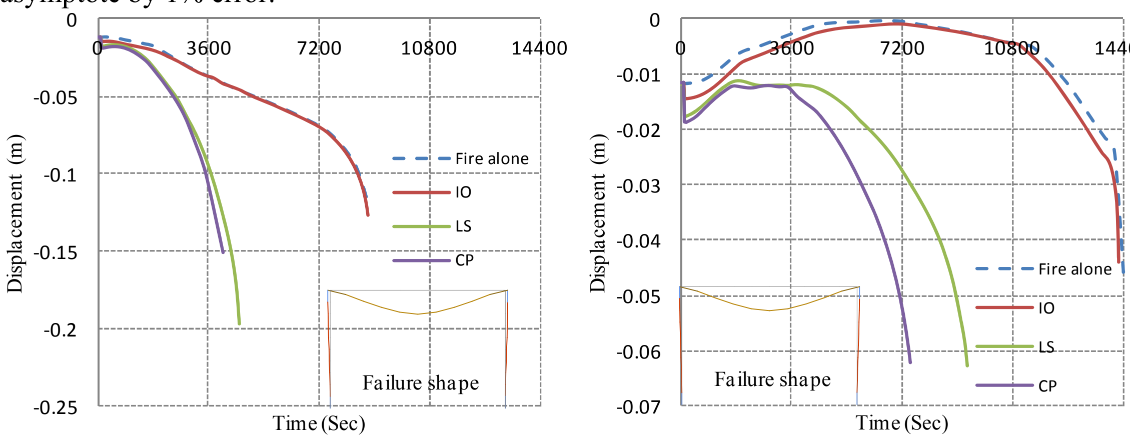
The sequential analysis comprises three main stages, which are gravity loading, followed by seismic pushover analysis, and finally post-earthquake fire. In the seismic analysis, the structure is subjected to a monotonically increasing lateral load to meet the specified performance levels. Accordingly, three different levels of performance, i.e. IO, LS, and CP, are met after the pushover analysis.The lateral forces corresponding to the target displacement at every performance level are extracted from the SAP2000 program, and are then input to the SAFIR program for performing the sequential analysis. Final stage of the sequential analysis is to apply a post-earthquake fire to the frames. Figure 6 shows the temperature distribution in a column at different levels of damage, from minor to major.



a) IO level of performance b) LS level of performance c) CP level of performance d) Temperature

Figure 6: Distribution of temperature in a column according to ISO 834

Figure 7 shows displacement against time for the frame, which implies the fire resistance in seconds for both scenarios (fire and PEF). The fire resistance is defined as the time at which the displacements, either globally or locally, go beyond chosen thresholds. The thresholds have been identified by the curve for displacements versus time step merging towards a vertical asymptote by 1% error.



a) Only the **beam** exposed to fire b) Only the **columns** exposed to fire

IO: Immediate Occupancy , LS: Life Safety , CP: Collapse Prevention

Figure 7: Fire resistance of the frame

As is seen in the figures, regardless of subjecting a structure to fire alone or PEF, there is a correlation between the fire-resistance rating and the performance levels. Indeed, along with increasing the lateral displacement in the frames, the fire resistance decreases such that the fire resistance of the frames pushed to CP level of performance is much lower than that of the frames pushed to LS or IO levels of performance. The figures also show a minor difference between the fire resistance at IO level of performance and fire alone. That is mostly because at IO level of performance, only minor damage occurs, resulting in insignificant residual displacement and/or degradation in strength and stiffness. It is also seen that fire resistance declines considerably when only the beam is exposed to fire, compared to exposing the columns to fire. In other words, it seems that the beam is more sensitive to fire than the columns.

6 CONCLUSION

Post-earthquake fire (PEF) is one of the most problematic situations in seismic regions. In this research, sequential nonlinear analysis is proposed for PEF. An RC portal frame (L=1.5H and, where H=3.0 m) was selected and then pushed to arrive at three different lateral displacements corresponding to three different performance levels, i.e. Immediate Occupancy, Life Safety and Collapse Prevention. That is, the maximum allowable inter-story drift was assumed to satisfy the mentioned performance levels. Pushover curves were then extracted for use in the subsequent analysis. Sequential loading, consisting of gravity and lateral loads followed by fire loads, was a key aspect of the study, conducted using SAFIR software. In SAFIR, the P-Δ effect and the residual lateral deformation as well as degradation in stiffness were considered. Defining the damaged sections (in terms of spalling of 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:

1)While there exist no computer program that can trace the response of an element in the full range of loading consisting of gravity loads, earthquake loads and fire loads up to collapse; sequential analysis using a combination of softwares and simplifications as performed here is proved to be a functional tool for considering the effect of residual deformations resulted from an earthquake, as well as degradation in stiffness and strength while performing the fire analysis.

2) There was a considerable difference between the results of fire-alone and PEF resistance when the frame was pushed to arrive at LS and CP level of performance. However, the fire resistance of fire-alone situation and IO level of performance were roughly identical. The results showed that while the fire resistance in fire-alone situation was about 2 hours and 30 minutes, it reduced to about 70 minutes and 50 minutes at the LS and CP level of performance, respectively.

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