Post earthquake fire behaviour of composite steel framed structures

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Post-earthquake fire behaviour of composite steel-framed structures

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ABSTRACT: This paper utilises a 3D numerical model to simulate behaviour of composite steel frames in fire following earthquake. The aim of this study is to provide insight into the effect of prior earthquake damage on structural behaviour in fire. It compares structural behaviour of the frame in two different conditions, fire in normal conditions (F) and post-earthquake fire (PEF). The earthquake analysis is performed using pushover analysis and then is followed by fire analysis using the Eurocode parametric fire. Additionally the effect of fire insulation delamination during earthquake is also taken into account. The contrast in behaviour of the two conditions is presented and explained.

1 INTRODUCTION
Fire following earthquake has been demonstrated to be a major threat for buildings in seismic-prone areas. A number of recorded experiences indicate that damage caused by fire following earthquakes can be more severe than that caused by earthquake itself. For instance, 80% of total damage in the 1906 San Francisco Earthquake was caused by fires following the earthquake. In the 1923 Tokyo earthquake, the losses due to fire following the earthquake event was over 70% of total building losses and resulted in 140,000 deaths. More recently, post-earthquake fire destroyed 7000 buildings in the 1995 Kobe earthquake.

Despite its potential as a cause of major devastation, it is a widely held view that fire following earthquake is not considered as a specific loading case in many prevalent design approaches. The current philosophy of seismic design permits a certain degree of damage in the structural elements, which make the structure more vulnerable when subjected to fire following earthquake. With the significant damages and losses recorded in past events, it would seem necessary to quantify the behaviour of structures under multi-hazard events such as fire following earthquake.

Steel-framed composite structures have been widely utilised in multi-storey building construction since they offer many advantages. Primarily, the composite interaction between the slab and steel beams enhances load carrying capacity and stiffness. Moreover, metal decking on the top of the steel beam can act as a permanent formwork thus eliminating external formwork. Hence, the use of composite slab reduces construction and workforce cost.

However, there is currently a lack of understanding on the behaviour of steel-framed composite structures subjected to fire following earthquake. Della Corte et al. presented finite element analysis to investigate structural behaviour of steel moment resisting frames (MRF) under fire following earthquake. Initially, a residual deformation was imposed to the building to represent the earthquake effects. Thermal-mechanical analysis was performed to simulate fire effects. This study showed that seismic design philosophy can influence the structural behaviour of a MRF under fire following earthquake. Faggiano et al utilised pushover and thermal-mechanical analysis to simulate structural behaviour of steel frames subjected to earthquake and then fire. It was observed that fire resistance of the steel frames subjected to fire following
earthquake are the same when performance of the frames during earthquake is still within the operational performance limit. However, there is a reduction in fire resistance of the structure when the performance of the frame surpassed ultimate design capacity and reached near-collapse levels.

Moreover, Memari et al. presented finite element simulation to study post-earthquake fire performance of steel moment resisting frames with reduced beam section (RBS) connections. The frames were subjected to a suite of ground motion records to simulate earthquake using nonlinear dynamic analysis. Thermal-mechanical analysis was then performed to simulate post-earthquake fire. The frames were considered fireproofed. Thus, fire was applied only at location of the RBS connection considering that fireproofed delamination occurs at the RBS connection during earthquake. Global and local responses of the frames were investigated according to ASCE standard 41-06 performance limit.

All of the aforementioned studies have focused on non-composite steel frames. Most of the structures have been analysed assuming 2D plane-frame behaviour, without considering the presence of a composite slab. Although some essential issues of fire behaviour can be captured, 2D frames fail to consider the load redistribution path in a realistic structure in particular the tensile membrane action of the concrete slab. Previous studies have shown that the effects of 3-D behaviour on undamaged steel-composite structures under fire loads are significant and can offer great reserves of strength. Especially since the UK’s Cardington fire test, research has been conducted to investigate and understand the behaviour of steel-framed structures in fire. It was confirmed that the composite slabs have an important role in the survival of the frame, through tensile membrane action.

This paper presents a study conducted on a generic three dimensional composite structure under fire following earthquake. A series of analyses are carried out to simulate a post-earthquake fire event. In this research, fire insulation delamination and residual displacement are applied to the structure to represent earthquake damage. This is followed by fire analysis using standard fire and natural fire. Fire analysis in the absence of earthquake is also performed to the undamaged structure as a reference to observe the effect of earthquake damage in the post-earthquake fire analysis.

2 GENERIC BUILDING

To represent a typical commercial office building, a generic five-storey composite steel-frame structure is analysed in this study. The frame is designed according to Eurocode 3, 4 and 8 for high seismicity. The structure’s plan and elevation views are shown in Figure 1. The combined dead and live loading at fire is taken as 5.4 kN/m² (load factor of 1.0 dead load and 0.5 for live load). Seismic design of the building is undertaken for a moment resisting frame (MRF) with medium ductility. The MRF is designed to be flexible by satisfying deformation criteria under seismic loading or the limitation of P-Δ effects under design earthquake loading.

The steel and concrete properties at ambient temperatures are presented in Table 1 and Table 2. Main beams and columns are assumed to be protected with lightweight insulating material which has thermal conductivity of 0.2 W/mK, specific heat of 1100 J/kgK and density of 300 kg/m³. In order to utilise the tensile membrane action in the slab panels, which are surrounded by the primary beams, the secondary beams are left unprotected. The concrete and steel properties at elevated temperatures are adopted according to Eurocode 4.

<table>
<thead>
<tr>
<th>Material</th>
<th>Modulus of elasticity E (Gpa)</th>
<th>Poisson ratio ν</th>
<th>Thermal expansion α (°C⁻¹)</th>
<th>Yield strength σy (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mild Steel</td>
<td>210</td>
<td>0.3</td>
<td>1.35 x 10⁻⁵</td>
<td>300</td>
</tr>
<tr>
<td>Rebar</td>
<td>210</td>
<td>0.3</td>
<td>1.35 x 10⁻⁵</td>
<td>450</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Poisson ratio ν</th>
<th>Thermal expansion α (°C⁻¹)</th>
<th>Compressive strength σc (Mpa)</th>
<th>Ultimate strain εu (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>0.25</td>
<td>9 x 10⁻⁶</td>
<td>30</td>
<td>0.002</td>
</tr>
</tbody>
</table>
Figure 1. Generic frame structure

3 NUMERICAL MODEL
The finite element software ABAQUS v6.14 is used to model and analyze the structure. Steel columns and beams are discretized using 1-D line elements and concrete slabs are modelled using shell elements. A solid flat concrete slab is considered as an idealization of the trapezoidal slab with metal decking. This was done to evaluate the slab effect but avoid the difficulties in using shell elements to simulate the ribbed composite slab. A tie constraint is applied to accommodate composite action between the steel beam and the concrete slab. For simplicity, it is assumed that the beam-to-column and secondary beam-to-primary beam connections behave as rigid and pinned, respectively.

In this study, a three-step analysis procedure is performed. First, the building is subjected to gravity load. Second, nonlinear pushover analysis is performed to simulate the earthquake event. Earthquake damages are discussed in Section 4.1. Third, a thermal-mechanical analysis is conducted to investigate post-earthquake fire behaviour. Material temperature distributions are discussed in Section 4.3.

4 POST-EARTHQUAKE FIRE ANALYSIS

4.1 Earthquake simulation
According to FEMA 356\textsuperscript{16}, Earthquake damage levels of the building are categorised into three different performance levels, i.e. Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) that represent minor to major damage. The performance levels are tied to the inter-storey drift ratio (IDR) as an indication of global stability of the structure. The IDR value is less than 0.7\%, 0.7-2.5\% and 2.5-5\% for performance level of IO, LS and CP, respectively.

In this study, it is assumed that earthquake damages are conservative and the performance of the building subjected to worst case scenario of earthquake will be considered. Therefore, col-
lapse prevention (CP) performance level is selected. This can be achieved by performing non-linear static pushover analysis. In this analysis, the building is pushed using a specific lateral load to arrive at a target displacement represented by the roof displacement at the center of mass of the building. Load duration is not essential for this analysis since long-term effects such as creep are not taken account. It should be noted that no dynamic effects are considered in this study.

Figure 2 shows base shear subjected to the building against roof displacement. As can be seen, the building was loaded up to a horizontal displacement of 0.6 m (IDR 3%) and was then unloaded. The result shows that there is a residual displacement of 0.37 m in the top of the building.

As mentioned above, the philosophy of seismic design permits a certain degree of damage in the structural elements. There is a possibility that the active fire protection system is compromised by ruptured water supply and delayed response of firefighting after the earthquake event. Thus, a passive fire protection system, such as sprayed fire resistive material (SFRM), may play an important role in the building response in fire.

However, the role of SFRM can be also compromised if the SFRM gets detached from the steel structures. Braxtan and Pessiki\textsuperscript{17} conducted experiments in SFRM on steel subjected to earthquake. Two common types of SFRM were used in the experiments: a dry-mix material and wet-mix material. The results showed that there is SFRM delamination concentrated in the beams where plastic hinges are formed at both ends. Results of the heat transfer analysis indicated that SFRM damage on the beam cause an increase in temperature in the adjacent column. Further study by Kodur and Arablouei\textsuperscript{18} showed that fire insulation delamination can reduce the failure time of the beam.

The above studies proved that passive fire protection system has a significant role in the vulnerability of steel structures subjected to fire following earthquake. Therefore, to determine the effect of fire insulation delamination, the steel structures at plastic regions in the beams are left unprotected. Delamination length is assumed to be 1.0 m from face of columns as shown in Figure 3.
4.2 Fire analysis

Two different fire curves as shown in Figure 4 are used to simulate the fire event: Standard Fire ISO 834\textsuperscript{19} and Natural fire. The natural fire is defined using a parametric fire curve according to Eurocode 1\textsuperscript{20}. Unlike Standard Fire\textsuperscript{19}, the natural fire covers two main phases in a fire event including the heating phase and cooling phase, and it is known that cooling can be as damaging for a structure as heating\textsuperscript{21}. The natural fire also considers compartment size, fire load, ventilation and the amount of combustibles. In this study it is assumed that only the first floor of the building is affected by fire and that the fire compartment encompasses the entire floor.

![Figure 4. Fire curve](image)

4.3 Structural temperatures

To perform thermal-mechanical analysis, it is essential to know the temperature of materials. The procedure in Eurocode 3\textsuperscript{22} is used to calculate steel temperature data. Figure 5 shows steel temperatures against time for unprotected and protected steel beams.

In order to obtain slab concrete temperature, 1-D heat transfer analysis with radiative and convection boundary conditions using the parametric fire curve is performed. The temperatures of top and bottom surface of concrete slab are plotted in Figure 6.

![Figure 5. Steel temperatures exposed to parametric fire](image)

a. Standard fire

b. Natural fire
5 RESULTS AND DISCUSSIONS

5.1 Impact of fire insulation delamination

To demonstrate the effect of fire insulation delamination under seismic loading, fire analyses only are taken into account in this study. The mid-span deflection of primary beam B12 (see Figure 1) with and without delamination under standard fire and natural fire are plotted in Figure 7 and Figure 8. It can be observed that the influence of fire insulation delamination is negligible, up to 200°C and 100°C under standard fire and natural fire, respectively. But beyond those temperatures, there is a sudden drop in deflection. This is expected because the temperature of delamination regions which are unprotected in primary beams reach 400°C, the point at which the yield stress of steel starts decreasing. The differences of mid-span primary beam deflection between delamination and without delamination become more obvious with increasing temperature.

It is commonly postulated that the failure of a beam is defined by mid-span deflection exceeding L/20 in which L is the beam span when the plastic hinges are formed and the beam is not able to transfer the load. The results show that the beam deflections are still within the limit for all cases. This is because the analyses consider the composite action between slab and the beams. The composite slab reduces deflection in the beam and thus it contributes to resistance against collapse.
5.2 Effect of residual displacement

This section compares structural behaviour of the frame in two different conditions, fire in normal conditions (F) and post-earthquake fire (PEF). Earthquake damage in PEF analysis is represented by residual displacement as explained in section 4.1.

Figure 9 shows mid-span deflection of primary beam B12 in normal condition (F) and post-earthquake fire (PEF) under standard fire and natural fire. It can be seen that for the PEF case, there is an initial displacement of 0.02 m in the primary beam. However, the pattern of deflection between fire in the normal condition and post-earthquake fire are the same. It is confirmed by the same pattern of the axial force of the beam in normal condition and post-earthquake fire as shown in Figure 10.

It can be seen that composite slabs produce different axial forces along the beams. On the other hand, previous studies on two dimensional frames indicate uniform axial forces along the beams. A previous study on a non-composite frame revealed that the tension forces developed during the cooling phase of fire are insignificant in terms of the overall beam behaviour. This conclusion is questionable for the composite frame in this study since it is found that the cooling phase creates high tensile forces. It can be seen in Figure 10b, high tension is found in the right side of the beam during the cooling phase. Thus, the cooling phase can be as damaging to the structure as the heating phase.

Figure 9. Comparison of mid-span beam B12 deflection between fire only and post-earthquake fire

Figure 10. Comparison of axial forces at B12 between fire only and post-earthquake fire
Outward horizontal displacements of column B4 (see Figure 1) against time are shown in Figure 11. As is seen, the beam initially pushes out the column thus compression occurs in the edge of beam. This behaviour continues until tension develops in the beam and pulls in the column. The results show that the residual displacement due to earthquake increases horizontal displacement of the columns that may endanger the building due to enhanced P-Δ effects.

Figure 11. Horizontal displacement of column B4

6 CONCLUSION

This study presented a numerical investigation of a steel-framed composite floor system under fire following earthquake. Based on the study, the following conclusions can be made:

1. There is no structural collapse on the MRF designed for seismic actions subjected to fire following earthquake. However, residual displacement due to earthquake may deteriorate structural performance of the building.
2. Fire insulation delamination results in sudden drop in the deflection of primary beams when the steel temperature at the delamination region reaches 400°C, the point at which the yield stress of steel starts decreasing.
3. The pattern of deflection for both conditions, fire in normal condition and fire after earthquake, are the same.
4. The effect of the slab must be considered in the analysis since the composite action between beam and slab can enhance the fire performance.
5. Post-earthquake fire increases horizontal displacement of the columns potentially endangering building stability as a result of the ensuing P-Δ effects.

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