Post earthquake fire behaviour of composite steel framed structures

Document Version

Accepted author manuscript

[Link to publication record in Manchester Research Explorer](#)

Citation for published version (APA):

Suwondo, R., Gillie, M., Bailey, C., & Cunningham, L. (2017). Post earthquake fire behaviour of composite steel framed structures. In M. Gillie, & Y. Wang (Eds.), *Applications of Structural Fire Engineering 2017*

Published in:

Applications of Structural Fire Engineering 2017

Citing this paper

Please note that where the full-text provided on Manchester Research Explorer is the Author Accepted Manuscript or Proof version this may differ from the final Published version. If citing, it is advised that you check and use the publisher's definitive version.

General rights

Copyright and moral rights for the publications made accessible in the Research Explorer are retained by the authors and/or other copyright owners and it is a condition of accessing publications that users recognise and abide by the legal requirements associated with these rights.

Takedown policy

If you believe that this document breaches copyright please refer to the University of Manchester's Takedown Procedures [<http://man.ac.uk/04Y6Bo>] or contact uml.scholarlycommunications@manchester.ac.uk providing relevant details, so we can investigate your claim.

Post-earthquake fire behaviour of composite steel-framed structures

R. Suwondo, M. Gillie, C. Bailey, L. Cunningham

School of Mechanical, Aerospace and Civil Engineering, University of Manchester, Manchester

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 itself¹. For instance, 80% of total damage in the 1906 San Francisco Earthquake was caused by fires following the earthquake². 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 deaths². More recently, post-earthquake fire destroyed 7000 buildings in the 1995 Kobe earthquake³.

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^{4,5}. 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.⁶ 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.⁷ 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 limit⁸. 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.⁹ 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 41-06⁸ 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 studies¹⁰ 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 test¹¹, 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 3¹², 4¹³ & 8⁵ 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 material¹⁴ 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 4¹⁵.

Table 1. Ambient temperature material properties for steel

Material	Modulus of elasticity E (Gpa)	Poison ratio ν	Thermal expansion α ($^{\circ}\text{C}^{-1}$)	Yield strength σ_y (Mpa)
Mild Steel	210	0.3	1.35×10^{-5}	300
Rebar	210	0.3	1.35×10^{-5}	450

Table 2. Ambient temperature material properties for concrete

Material	Poison ratio ν	Thermal expansion α ($^{\circ}\text{C}^{-1}$)	Compressive strength σ_c (Mpa)	Ultimate strain ϵ_u (Mpa)
Concrete	0.25	9×10^{-6}	30	0.002

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 idealisation 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 356¹⁶, 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.

Figure 2. Base shear against top displacement

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 Pessiki¹⁷ 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 Arablouei¹⁸ 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.

Figure 3. Fire insulation delamination at plastic hinge regions

4.2 Fire analysis

Two different fire curves as shown in Figure 4 are used to simulate the fire event: Standard Fire ISO 834¹⁹ and Natural fire. The natural fire is defined using a parametric fire curve according to Eurocode 1²⁰. Unlike Standard Fire¹⁹, 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 heating²¹. 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

4.3 Structural temperatures

To perform thermal-mechanical analysis, it is essential to know the temperature of materials. The procedure in Eurocode 3²² 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

Figure 6. Peak temperatures in concrete slab

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 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 load²³. 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.

Figure 7. Mid-span deflection of primary beam B12 under standard fire

Figure 8. Mid-span deflection of primary beam B12 under natural fire

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 frame⁹ 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.

ACKNOWLEDGEMENT

Authors would like to thank Indonesia Endowment Fund for Education (LPDP) for funding this PhD research.

REFERENCES

1. Scawthorn, C., Eidinger, J. & Schiff, A. Fire following earthquake. Technical Council on Lifeline Earthquake Engineering Monograph No. 26. American Society of Civil Engineering (2005).
2. Scawthorn, C. Fire following earthquake. USGS Open File Report 200-1150 (2008).
3. Faggiano, B. Fire after earthquake. Proceeding from WG1 Meeting on Urban Habitat Constructions Under Catrastophic Events, Prague 30-31 March 2007

4. ASCE. Minimum design loads for Buildings and other structures. ASCE/SEI 7-10 (2010).
5. BSI, BS EN 1998-1 Eurocode 8 Design of structures for earthquake resistance-Part 1: General rules, seismic actions and rules for building. (2004). British Standards Institution, London, UK
6. Della Corte, G., Landolfo, R. & Mazzolani, F. M. Post-earthquake fire resistance of moment resisting steel frames. *Fire Safety Journal*. 38, 593–612 (2003).
7. Faggiano, B., Esposito, M. & Mazzolani, F. M. Risk Assessment of Steel Structures Under Fire. 14th World Conference on Earthquake Engineering 2008, Beijing, China (2008).
8. ASCE. Seismic rehabilitation of existing buildings. ASCE/SEI 41-06. (2007)
9. Memari, M., Mahmoud, H. & Ellingwood, B. Post-earthquake fire performance of moment resisting frames with reduced beam section connections. *Journal of Construction Steel Research*. 103, 215–229 (2014).
10. Bailey, C. G. Membrane action of slab/beam composite floor systems in fire. *Engineering Structures*. 26, 1691–1703 (2004).
11. Kirby, B. R. British steel technical European fire test program design, construction and results. (1997).
12. BSI, BS EN 1993-1-1. Eurocode 3 Design of steel structures - Part 1-1: General rules and rules for buildings. (2005). British Standards Institution, London, UK
13. BSI, BS EN 1994-1-1. Eurocode 4: Design of composite steel and concrete structures Part 1-1: General rules and rules for building. (2004). British Standards Institution, London, UK
14. Buchanan, A. Structural design for fire safety. John Wiley & Son, UK, (2001).
15. BSI, BS EN 1994-1-2. Eurocode 4: Design of composite steel and concrete structures Part 1-2: General rules : Structural fire design. (2005). British Standards Institution, London, UK
16. Federal Emergency Management Agency. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. FEMA 356, (2000).
17. Braxtan, N. L. & Pessiki, S. P. Postearthquake Fire Performance of Sprayed Fire-Resistive Material on Steel Moment Frames. *Journal of Structural Engineering* 137, 946–953 (2011).
18. Arablouei, A. & Kodur, V. Effect of fire insulation delamination on structural performance of steel structures during fire following an earthquake or an explosion. *Fire Safety Journal*. 84, 40–49 (2016).
19. ISO 834-1:1999, I. Fire-resistance tests - Elements of building construction - Part 1: General requirements. (1999).
20. BSI, BS EN 1991-1-2 Eurocode 1: Actions on structures. Part 1–2: General actions. Actions on structures exposed to fire. (2002). British Standards Institution, London, UK

21. Röben, C., Gillie, M. & Torero, J. Structural behaviour during a vertically travelling fire. *Journal of Construction Steel Research*. 66, 191–197 (2010).
22. BSI, BS EN 1993-1-2. Eurocode 3 : Design of steel structures Part 1-2: General rules — Structural fire design. (2005). British Standards Institution, London, UK
23. Rezvani, F. H. & Ronagh, H. R. Structural response of a MRF exposed to travelling fire. *Proceeding of the Institution Civil Engineering - Structures and Buildings* 168, 619–635 (2015).