AN ABSTRACT OF THE THESIS OF

Walker Maddalozzo for the degree of Master of Science in Civil Engineering presented on June 10, 2021.

Title: Behavior of Steel Structures in Fire Following Earthquakes

Abstract approved:

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Fire following earthquake (FFE) events are cascading hazards that have caused great damage in the past and pose a continued threat to our communities. The increased risk for fire after a seismic event stems from potential damage to utilities such as gas or fuel pipelines, which can lead to ignition. Additionally, damage to water utilities can limit or prevent fire suppression efforts. Earthquake hazard studies performed to evaluate risks to Portland, OR and the California Bay Area found that seismic events in the areas could lead to damaging fires. The behavior of the lateral force resisting system and gravity system in a structure during a fire is critical to life safety. Damage to either of these systems could lead to loss of compartmentalization of a fire, which would allow further spread horizontally or vertically through a structure.

In order to model and observe FFE structural performance, the authors developed modeling methodologies in the open-source finite element (FE) program, OpenSees. The methodologies were developed through a series of validation studies,

benchmarking the results of OpenSees against experimental testing and against another FE program. The validation studies spanned from simple to complex 2D thermal-mechanical modeling techniques. These validation studies were then made available to the public on the OpenSees documentation website. The benchmarked methodologies were applied to a 2D steel moment resisting frame (MRF) to model FFE hazards through two investigations.

The first FFE investigation examined the effects of three compartment fire locations and two fuel load levels (high and low). Results of the FFE analysis were compared to a fire-only hazard, investigating the influence of fire following earthquake would have on the steel moment frames behavior. The first study was also completed to identify vulnerabilities for further study in the second FFE investigation. Damage was greater in the first compartment fire location and was investigated further in the second FFE study.

To investigate the 2D steel moment frame further, a coupled Incremental Fire Analysis (IFA) and Incremental Dynamic Analysis (IDA) was performed. The IDA considered three ground-motions recorded in Oakland, CA during the Loma Prieta earthquake scaled to 1, 3, and 5 times Peak Ground Acceleration (PGA). Fire hazards for the IFA analysis were developed using 10 parametric time-temperature curves then scaled to temperatures, 600^{o}C, 800^{o}C, and 1000^{o}C. Fire hazard Intensity Measure (IM) was maximum compartment gas temperature, and the Engineering Demand Parameter (EDP) was compartment beam displacement. Seismic hazard IM was PGA and the EDP selected was inter-story drift. Behavior of FFE and fire-only analysis was compared.

Behavior of Steel Structures in Fire Following Earthquakes

by

Walker Maddalozzo

A THESIS

submitted to

Oregon State University

in partial fulfillment of

the requirements for the

degree of

Master of Science

Presented June 10, 2021

Commencement June 2022

Master of Science thesis of Walker Maddalozzo presented on June 10, 2021

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Understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Walker Maddalozzo, Author

ACKNOWLEDGEMENTS

I would like to thank my major professor Dr. Erica Fischer for all that she has done for me. For providing the opportunity to pursue my education further and develop professionally. For all the wonderful doors she has opened for me in my life. The mentorship and guidance she has provided for me has been life changing.

I would like to thank Dr. Michael Scott, for his continued support with OpenSees. His help and guidance in de-bugging my models throughout my research has been priceless.

To my committee members, Dr. Andre Barbosa and Dr. Tyler Radniecki, thank you for your contributions and support throughout this process.

I would like to thank the Fischer Research Group for their constant support, proof reading, comedic relief and feedback. I would not have gotten through this process without them.

I would also like to thank my friends and family for their constant support and motivation, allowing me to call them and random hours of the day to complain about my models and the thesis writing process.

Lastly, I would like to thank my wonderful cat Ricky Martin. Without his support and endless love I would not have been able to make it through my gradate experience through a global pandemic.

CONTRIBUTION OF AUTHORS

Dr. Erica Fischer assisted with interpretation of the data and the analysis of the work in Chapters 2 and 3.

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CHAPTER 1: Introduction

1.1 Fire Following Earthquakes (FFE)

The world we live in is susceptible to many hazards, both natural and manmade. Traditionally, these hazards are analyzed independently. However, as the engineering community moves towards building more resilient communities, multi-hazard approaches to infrastructure design are required. Cascading hazards are multi-hazard events where hazards occur sequentially due to one hazard causing another. This document will focus on the cascading hazard of fire following earthquakes (FFE). A large seismic event can cause damage to utilities, increasing the risk of fire. Fires can ignite from damaged gas lines and electrical utilities, while damage to water utilities can prevent access to water for fire suppression.

Historically, FFE has caused widespread damage. 80% of the cost of damage due to the 1906 San Francisco earthquake was due to the fires occurring after the earthquake, lasted for three days. (Scawthorn, 2011). Additionally, the Tokyo earthquake that took place in the summer of 1923, resulted in FFE that caused 77% of the total losses (Mousavi et al. 2008). Within hours after the 1995 Kobe Earthquake, 142 fires were reported (Mousavi et al. 2008).

Earthquake scenario studies of Portland, Oregon and the Bay Area of California have highlighted the probability of FFE and the potential economic damage these fires could cause in the regions. The HayWired Earthquake Scenario (Hudnut et al. 2018) concluded that a Hayward Fault rupture in the California Bay Area could cause fires with the potential to destroy 52,000 single-family homes. The Portland Bureau of Emergency Management (PBEM, 2012) reported that FFEs after a Cascadia subduction-zone earthquake may cause fires due to gas and fuel line ruptures. In addition to damage to fuel and gas lines, the water distribution system and fire suppression systems could be damaged and limit firefighting resources.

1.2 Structural Fire Engineering

1.2.1 Current prescriptive approaches to structural fire engineering

The *International Building Code* (IBC) (ICC, 2018) provides two options for fire protections design: prescriptive design and alternative means. The prescriptive design is commonly used throughout the US for fire protection design. This design methodology assigns a fire resistance rating (FRR) in units of hours to each structural member of the building. This hourly rating is based upon building characteristics such as type of construction, building area and height, and occupancy type. The FRR corresponds to the time the structural assemble (structural member and thickness of fire protection) meets certain failure criteria during a standard fire test (ASTM, 2020). The failure criteria is not always structural failure, but can include the standard fire exposure time until material reaches a certain temperature or, often in the cases of floors, something can be ignited on the unexposed side of the floor (often referred to as fire integrity failure).

1.2.2 Structural Fire Engineering

Following the World Trade Center (WTC) tower collapses (WTC 1, 2, and 7) in 2001, significant research occurred in the field of structural fire engineering. Structural fire engineering entails engineers designing structural components, members, and frames to maintain load-carrying capacity throughout a design-basis fire (AISC, 2016). This design methodology includes calculating deformations of structural components and ensuring that these deformations do not damage the compartment boundaries such that fire spread can occur vertically throughout the building or horizontally along the floor plate.

Designing or evaluating a structure for fire safety can be performed structural fire analysis. This structural fire analysis has two parts: calculating the thermal load on the structure (time-temperature response of the structural member materials) and then calculating the structural response to that thermal load. The thermal load is determined through a heat transfer analysis of structural components in response to fire gas temperatures. These gas temperatures can be calculated through computational fluid dynamics (CFD), parametric time-temperature curves, or fire tests. CFD analysis can be highly complex to perform, while fire tests can be expensive. Therefore, often the gas temperatures are calculated using a parametric time-temperature curve that is dependent upon the compartment geometry and materials (CEN, 2004a).

Unlike the standard fire curves (ASTM 2020; ISO 2014) the Eurocode parametric fire curves include a cooling phase. This cooling phase is important to global structural behavior because during this phase thermal contraction can cause large axial forces in members and connections. Compared to CFD models and fire tests, the Eurocode model provides adequate predictions of average compartment temperature (Pope and Bailey 2006). Because the Eurocode model is easy to use, it is a popular choice in structural fire design and research (Wang et al. 2013). The gas temperatures are utilized to perform heat transfer analyses of the structural members subjected to the fire. The heat transfer analysis results are nodal temperatures through the depth of the section that can be utilized in a stress-based analysis to calculate the structural response to the fire. The thickness of the fire protection or the size of the structural members within the building are designed to meet the performance objectives of the owner or those listed at the beginning of this section.

Structural fire engineering allows the designer and owner some flexibility in the design of the structure to meet the performance criteria. However, many analysis tools used in structural fire engineering are expensive and additionally computationally inefficient and time-consuming.

1.2.3 Incremental Fire Analysis (IFA)

IFA is an approach to structural fire engineering to strategically subject a structure to a suite of fire scenarios. This approach is based on earthquake engineering's incremental dynamic analysis (IDA) (Moss et al. 2014). IFA exposes a structure to a fire and then increases the intensity measure (IM) of that fire incrementally Researchers have not come to consensus on the best IM to be used in IFA. Potential IMs that have been previously used include peak compartment gas temperature (Lange et al. 2014), total compartment fire load (Gernay et al., 2016), radiant heat energy (RHE) (Moss et al. 2014), and area under the time-temperature curve (Devaney, 2014).

1.2.4 Fire Following Earthquake (FFE) Analysis

Over the past decade, various researchers have explored FFE behavior through different modeling techniques. Usmani (2008) outlined research needs to promote better performance of structures in fire following earthquake events. Because cascading FFE events are vastly complex in nature, FFE demands need to be categorized into a risk-based framework to comprehensively characterize uncertainties in FFE and Fire hazards.

Zaharia and Pintea (2009) modeled a two and five story 2D frame to evaluate FFE behavior. Pushover analysis was performed represent the earthquake loading before exposing a frame to both the standard fire (ISO, 2014) and natural fire curves. The frames were both exposed to two levels of ground motion, near and far field. The level of fire-resistance was determined via the level of damage from the push-over phase. The study found that the frames that had less damage from the push-over analysis via lateral drifts were able to maintain load carrying capacity for a longer period of time throughout both the standard and natural fires.

Behnam and Ronagh (2014) investigated a two-dimensional (2D) 10-story moment frame accounting for seismic loading by residual deformations and stiffness degradation. The study examined at three fire scenarios with fires at different stories, and varying time delays (5 and 25 minutes) before the compartments fires spread vertically to other floors observing the effects of vertical travelling FFE. This analysis approach resulted in compartments in different phases of fire, some in the cooling phase while others were in the heating phase. The results showed that the FFE resistance was greater with the longer 25 minutes. The results also showed that subjecting the frame to a delayed fire of 25 minutes leads to the collapse of the frame during cooling phase, while the five minute delay of the vertically traveling fire led to collapses during heating phase of the fire.

Khorasani et al. (2015) performed a reliability-based FFE analysis in OpenSees comparing FFE to fire-only behavior in a 10 story steel moment-resisting frame (MRF). The fire hazard selected for the analysis was a 1MP fire event (Garlock and Quiel 2007), and ground motions were selected from the Loma Prieta earthquake event. Comparisons to the fire-only and FFE analysis found similar behavior. Findings from the analysis concluded that the seismic loading decreased the time for plastic hinges in the beams during fire loading. Khorasani et al. (2015) also concluded that FFE can cause larger lateral drifts in exterior columns while decreasing drifts in interior columns due to thermal expansion of the beams.

1.2.5 Fire Analysis in OpenSees

OpenSees is an open-source object-oriented finite element program developed at University of California, Berkeley for the purpose of simulating the seismic response of structural and geotechnical systems (McKenna et al. 2010). OpenSees is now used internationally by researchers and engineers. The ability of OpenSees to simulate large plastic deformations provides an opportunity for this software to be used to simulate the behavior of buildings during a fire, particularly steel-frame buildings. Previous researchers have significantly contributed to the development of fire analyses in OpenSees (Jiang et al. 2013; Khorasani et al. 2015).

Jiang and Usmani (2013) introduced the implementation of thermal-mechanical analysis of steel frames in OpenSees. The degradation of mechanical properties of steel at elevated temperatures was added to existing OpenSees objects, updating at each time step. Additionally, a new thermal load class was created to store temperature distributions through the depth of the section of a structural member. These developments made structural fire analysis possible within OpenSees, thereby providing the opportunity for coupled FFE analysis.

Khorasani et al. (2015) further developed the temperature-dependent steel material model to be bi-directional and to consider the effects of strain reversals. The elastic-perfectly plastic stress-strain relationship assumed for seismic analysis cannot be applied to fire analysis because as temperatures increase, the steel stress-strain relationship becomes more nonlinear. In addition, natural fires have a heating and cooling phase during which results in large tensile forces in members and connections. Therefore, these temperature-dependent material models must be bi-directional to account for tension and compression in members. Because of the addition of thermal modeling capabilities to OpenSees, it allows the ability to perform coupled FFE analysis, ability to apply thermal loads directly after seismic loads with computational efficiency.

1.3 **Motivation and Research Need**

Recently there have been a number of newly published guidelines and code changes as it pertains for fire protection design. Structural Fire Engineering has been incorporated into structural engineering standards, particularly, *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-16) (ASCE, 2016). In this standard, Chapter 1 allows for advanced analysis or an alternative analysis for fire protection engineering. The ASCE/SEI Fire Protection Committee published a Manual of Practice, *Structural Fire Engineering* (LaMalva, 2018). This manual is an orderly presentation of material that engineers can use in their daily practice to implement structural fire engineering as it is presented in ASCE 7-16. The new Structural Stability Research Council (SSRC) *Guide to the Stability Design Criteria for Metal Structures,* will include a new chapter on addressing stability under fire loading. Lastly, the American Institute of Steel Construction (AISC) *Specification* (AISC, 2016), Appendix 4 has been expanding over the last few editions. This expansion has provided more guidance to engineers on how to perform simplified analysis of structural members subjected to fire loading.

In each of these publications, there are two methods of analysis: (1) Simplified equations for the calculation of individual member capacities, and (2) Advanced analysis approaches that require detailed finite element modeling. The first method of analysis is on a member-by-member scale, which does not include the analysis and evaluation of connection performance. Whereas the second method of analysis, the advanced analysis, requires simulation of realistic connection boundary conditions for the gravity framing in a building.

To assess the behavior of structures in fires, structural engineers must employ finite element programs that can simulate potentially large plastic deformations that can occur in structural members during a fire. A variety of finite element programs are used for these simulations: Abaqus, ANSYS, SAFIR, LS Dyna. However, these programs are not open source (i.e. difficult to modify for new applications like multi-hazard analysis) and can be costly. Therefore, researchers are forced to redevelop modeling capabilities and benchmarking examples for every research endeavor and it is difficult for structural engineering practitioners to include structural fire engineering in their practice areas. OpenSees is uniquely positioned to have the capabilities for structural fire engineering analysis due to the open-sourced nature of the program. However, the current version of OpenSees has some limitations that prevent researchers and structural engineers from being able to simulate the performance of steel-frame buildings in a fire. This project aims to overcome these limitations so that engineers throughout the world can utilize OpenSees for fire or FFE analysis of steel-frame buildings.

1.4 Research Goals and Objectives

The purpose of this research is to perform fire and FFE analysis of a 2D steel frame in OpenSees. These goals will be achieved through the following research objectives:

1. Benchmark fire modeling methodologies within the current version of OpenSees
2. Evaluate the FFE performance of a 2D moment-resisting frame to identify vulnerabilities within the frame,
3. Compare the fire-only behavior of a 2D moment-resisting frame to the FFE behavior of the frame using IDA and IFA analysis techniques.

4. Identify vulnerabilities within the 2D moment-resisting frame when subjected to a coupled IDA-IFA FFE analysis that prevents the frame from maintaining load-carrying capacity throughout the analysis.

To complete these research objectives, the research project is divided into the following tasks:

- **TASK I – Development of benchmarking examples in OpenSees:** To benchmark fire modeling methodologies within the most recent version of OpenSees, a series of benchmarking examples were developed. The results of these examples were compared with either experimental testing data or the analysis results from other finite element (FE) modeling software (Abaqus). The modeling methodologies developed throughout this task are used in **Task II** and **Task IV**.
- **TASK II – Preliminary FFE analysis of 2D moment-resisting frame:** This task focused on using the modeling methodologies to evaluate the FFE performance of a 2D moment-resisting frame. Three compartments within the frame were considered and two different fires were considered within the three compartments. A short duration and long duration fire were considered by varying the fuel load within the compartment. The results of the FFE analysis were compared with a fire-only analysis of the same compartments. The worst-case compartments were chosen to perform an IDA-IFA coupled FFE analysis in **Task IV**.
- **TASK III – Development of IFA fire curves:** There are many different approaches to choosing the IM utilized within IFA. This task included a thorough literature review of IFA and different IMs utilized in previous research. Fire curves were then developed considering variation of ventilation and fuel loads within a compartment in the 2D moment-resisting frame used for the analysis in **Task II**. Fire curves were scaled to three different

maximum temperatures of the fire to develop a suite of fire curves that will be used in the IDA-IFA coupled FFE analysis and the fire-only analysis in **Task IV**.

- **TASK IV – Performance of IDA-IFA coupled FFE analysis on 2D moment-resisting frame:** This task consists of the performance of the IDA-IFA coupled FFE analysis of the 2D moment-resisting frame and the fire-only analysis for all of the fires developed for the IFA. The analysis was performed using the modeling methodologies developed in **Task I** and in the worst-case compartments identified through the preliminary analysis in **Task II**. The FE models simulated both the seismic and the fire performance of the 2D moment-resisting frame using temperature-dependent material properties to account for large plastic deformations. The results of this analysis were utilized to identify vulnerable areas of the structure and under what hazard conditions significant damage can occur during an FFE.

1.5 Thesis Outline

This thesis follows a manuscript format that includes two manuscripts developed as part of this research work. Each chapter provides the necessary background, research methods, and results for achieving the research objectives. Chapter 2, the first manuscript, accomplishes research objectives 1 and 2. This chapter documents the development and benchmarking of thermal analysis examples in OpenSees as well as preliminary FFE analyses. Chapter 3, the second manuscript, accomplishes research objectives 3 and 4. This chapter documents an IDA-IFA coupled FFE analysis and provides a synthesis of the results to characterize under what circumstances of both earthquake and fire hazard significant damage may occur within a steel-frame building. Chapter 4 summarizes the research, presents final conclusions, discusses implications of the research, and provides suggested areas for future research.

CHAPTER 2: Benchmarked Examples and Preliminary FFE Analysis

Material from:

Maddalozzo, W., & Fischer, E. C. (2020). Post-earthquake fire performance of steel buildings. 17th *World Conference on Earthquake Engineering, 17WCEE*: Sendai, Japan.

DOI:

Abstract

A large seismic event can lead to an increased risk for fires. During an earthquake, seismic demands on steel structures are primarily resisted by lateral-force-resisting systems (LFRS). While large plastic deformations of steel members are required to dissipate energy produced by an earthquake; during a fire, the demands are primarily resisted by the gravity framing members. Therefore, damage to the gravity framing system during an earthquake could increase the vulnerability of the building to de-compartmentalization of a fire, fire spread, and partial or full collapse of the building during a fire. The ability of OpenSees to model coupled fire following earthquake analysis allows these vulnerabilities to be investigated. This paper summarizes validation studies completed by the authors which were benchmarked against experimental data, other finite element programs, and temperature-dependent material properties. The authors used the validated modeling techniques to investigate the fire following earthquake and fire-only behavior of a two-dimensional moment-resisting frame. The location of the compartment fire and the fuel load were varied as parameters in the study to conclude if damage to the moment frame during an earthquake would influence the fire following earthquake behavior of the moment frame.

Keywords: Fire following earthquakes, Nonlinear Analysis, Opensees

2.1 Introduction

A large seismic event can cause damage to utilities, increasing the risk for building fires. Damaged gas and electrical utilities can ignite fires, while damaged water utilities prevent access to water for fire suppression. Historically, fire following earthquakes (FFE) has caused widespread damage such as after the 1906 San Francisco earthquake (Humphrey et al. 1907), 1995 Kobe earthquake (Himoto, 2019), 2012 Tohoku earthquake and tsunami (Himoto, 2019), and the 2019 Ridgecrest earthquake (Fischer et al. 2019). Earthquake scenario studies of Portland, Oregon, and the Bay Area of California have highlighted the probability for FFE and the potential economic damage these fires could cause in the regions. The HayWired Earthquake Scenario (Hudnut et al. 2018) concluded that a Hayward Fault rupture in the California Bay Area could cause fires that destroy 52,000 single-family homes. The Portland Bureau of Emergency Management (PBEM 2012) reports that FFEs after the Cascadia subduction-zone event may cause fires due to ruptures of gas and fuel lines. In addition, damage to fuel and gas lines, the water distribution system and fire suppression systems will be damaged such that firefighting resources may be limited.

During an earthquake, seismic demands on steel structures are primarily resisted by lateral-force-resisting systems (LFRS). Large plastic deformations of the steel members and connections are required to dissipate energy produced by an earthquake. The gravity framing system is designed to maintain structural integrity throughout an earthquake by deforming with the structure. However, during a fire, the gravity framing system is designed to resist the imposed fire loads, and observations after previous fires have shown that large plastic deformations can occur (Routley et al. 1991; Usmani et al. 2000). Therefore, any damage to the gravity framing system during an earthquake could increase the vulnerability of the building to a fire leading to de-compartmentalization of the fire, fire spread, and partial or full collapse of the building.

Researchers have previously used OpenSees to simulate the behavior of buildings during earthquakes. The ability of OpenSees to simulate large plastic deformations provides an opportunity for this software to be used to simulate the behavior of buildings during a fire. Previous researchers have significantly contributed to the development of OpenSees for the ability to perform FFE analyses (Jiang et al. 2013; Khorasani et al. 2015; Walls et al. 2018).

Jiang and Usmani (2013) introduced the implementation of thermal-mechanical analysis of steel frames in OpenSees. Modifications to existing OpenSees objects to include the degradation of mechanical properties of steel at elevated temperatures were added, updating at each time step. Additionally, a new thermal load class was created to store temperature distributions throughout the depth of the section. These developments resulted in the ability of structural fire analysis within OpenSees, thereby providing the opportunity for coupled FFE analysis to be performed.

Khorasani et al. (2015) further developed the temperature-dependent steel material model to be bi-directional and to consider the effects of strain reversals. The elastic, perfectly plastic stress-strain relationship that is assumed for seismic analysis cannot be applied to fire analyses. As temperatures increases, the steel stress-strain relationship becomes more nonlinear. In addition, during a natural fire, there is a heating and a cooling phase, therefore these temperature-dependent material models must be bi-directional to account for tension and compression in members.

Walls et al. (2018) developed a Fiber Beam Element (FBE) in OpenSees. This element is a Euler beam element; however, rather than the axial stiffness, bending stiffness, and neutral axis remaining constant throughout the analysis, the neutral axis changes as a function of temperature and is used to recalculate the section stiffness at each step. The development of the FBE is valuable as it allows modeling of the effect of composite slabs in steel-frame buildings throughout a fire, while also reducing analysis time. The composite slab contributes to the survival time of the building through the catenary action and capacity of the composite floors. However, as the steel beams are heated and lose stiffness through degrading material properties, the location of the neutral axis will shift towards the cooler portions of the steel section.

During a fire, the restraint of the surrounding structure will impose large axial force demands (compression and tension) in the connections. Therefore, the location of the compartment fire within a building could greatly influence the global behavior of the structure. The work performed by previous researchers (Jiang et al. 2013; Khorasani et al. 2015; Walls et al. 2018) has not been validated in the newest version of OpenSees, preventing new researchers from incorporating their developments. This paper will: (1) summarize a validation study by the authors to benchmark modeling capabilities for the fire behavior of steel structures in OpenSees both with experimental tests, temperature-dependent thermal material models, and other finite element programs; (2) summarize a study on the FFE behavior of a two-dimensional (2D) LFRS to examine the effects of compartment fire location and fuel load (low and high), and compare the behavior of the structure in an FFE to only a structural fire.

2.2 Validation studies in OpenSees

This section summarizes five validation studies performed within OpenSees for fire-only analysis. These studies vary from a single beam model to frame analysis, benchmarking modeling capabilities, and validating material properties of steel within OpenSees. The results of the validation studies are compared with temperature-dependent material properties, test data, or finite element modeling results obtained using the commercially available software, Abaqus (Abaqus 2017). All validation models use displacement-based beam elements. Figures summarizing the models are shown in Figure 2.1.

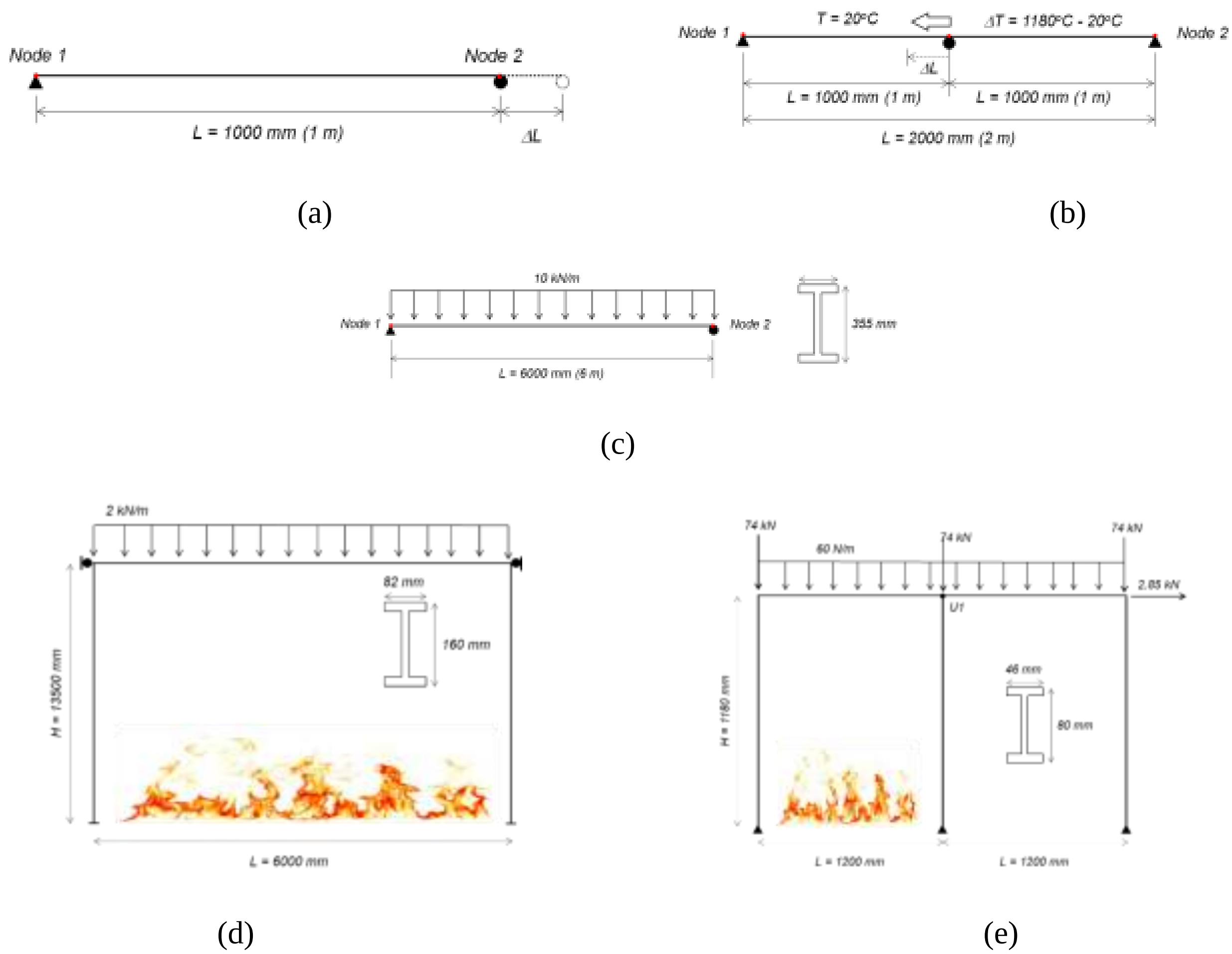


Figure 2.1. Summary of validation studies used to benchmark modeling capabilities in OpenSees (a) Study 1, (b) Study 2, (c) Study 3, (d) Study 4, and (e) Study 5

Validation Study 1 is of a steel beam heated to 1180°C linearly with respect to time. The temperature through the beam cross-section is uniform. One end of the beam is restrained against the vertical and horizontal movement, while the other end is free to expand horizontally. The horizontal displacement of the free end of the beam is recorded throughout the simulation and normalized with respect to the original length of the beam displayed in Figure 2.1a. This study validates the ability of the steel temperature-dependent material property (*Steel01Thermal*) within OpenSees to simulate the thermal expansion of a steel member during a fire. The normalized displacement is the thermal strain, or thermal expansion, and is compared with the calculated values in Eurocode 3 (CEN, 2004c) (see Figure 2.2a).

Validation study 2 consists of a simply supported steel beam with vertical displacement restrained at the mid-span node of the beam as shown in Figure 2.1b. The beam is created out of two elements where only one of the elements is heated to 1180°C. The thermal expansion of the mid-span node is recorded throughout the simulation in addition to the internal forces that are generated in the restrained beam element. The horizontal movement of the mid-span node and internal forces are plotted in Figures 2.2b and 2.2c, respectively, and compared against the results from performing the same analysis in Abaqus. These plots show that OpenSees is able to simulate the restraining forces in a structure at elevated temperatures.

Validation study 3, shown in Figure 2.1c is of a simply supported I-shape steel beam that has sequential mechanical loading then heating. A distributed load of 10 kN/m is applied to the beam. With the load sustained, the beam is heated to 1180^{o}C linearly with respect to time with uniform temperature throughout the section. Mid-span vertical displacement is recorded during the fire analysis. This study also investigates the impacts of including 2nd order geometric transformations, and the impact that restraining and un-restraining the horizontal displacement of the boundary conditions has on the mid-span vertical displacement of the beam. The results of this comparison are shown in Figure 2.2d. The results of the study are also compared with mid-span displacements obtained using a finite element program, Abaqus (labeled AB in Figure 2.2d). The comparison between the two analysis methods (Figure 2.2d) demonstrates that OpenSees has the ability to perform a coupled analysis with mechanical loading followed by heating of the elements. In addition, the comparison of the analysis types (linear versus nonlinear) plotted in Figure 2.2d shows that the inclusion of 2^{nd} order geometric transformation (or nonlinear analysis) is critical to simulate the large displacements that occur in a structure during a fire.

Validation study 4 focuses on system-level behavior and implementing a user-defined nonlinear time-temperature curve for the heating of the steel members. A one-bay frame is modeled using fixed column bases and horizontal restraints applied to the upper-corner frame nodes representing lateral bracing (Figure 2.1d). The frame is sequentially loaded with a distributed beam load of 2 kN/m and then the frame is heated using a user-defined parametric time-temperature curve representing a compartment fire, calculated using ISO 834 (Figure 2.2e) (ISO, 2014). Heat transfer analysis is performed in Abaqus to calculate nodal temperature histories. The nodal temperature histories are used as inputs into the structural fire analysis in OpenSees. Mid-span vertical displacement of the beam is recorded. The results of the analysis in Figure 2.2e shows the beam mid-span deflection throughout the fire. The maximum deflection was $L/14.5$, where L is the length of the beam. The mid-span displacements plotted in Figure 2.2e are compared with the displacement results of an analysis performed in Abaqus (Figure 2.2e). Figure 2.2e shows that these two analysis programs have good agreement with one another.

Validation study 5 applies modeling methodologies developed through the other studies to a two-bay frame with a fire in one of the bays, shown in Figure 2.1e. The two-bay frame is modeled using pinned column bases. The beam is subjected to a vertical 60 N/m distributed load, the columns each have a 74 kN concentrated vertical loads applied, and the frame is subjected to a horizontal concentrated load of 2.65 kN at the upper-right node (shown in Figure 2.1e). The left compartment is heated to 550°C linearly with respect to time with uniform temperature through the beam and column sections. Horizontal displacement is recorded at the upper-corner node, U1, of the frame with a maximum drift ratio of 2.7%. The results from this validation study are compared with experimental test data (Rubert et al. 1986) and have good agreement with one another as shown in Figure 2.2f.

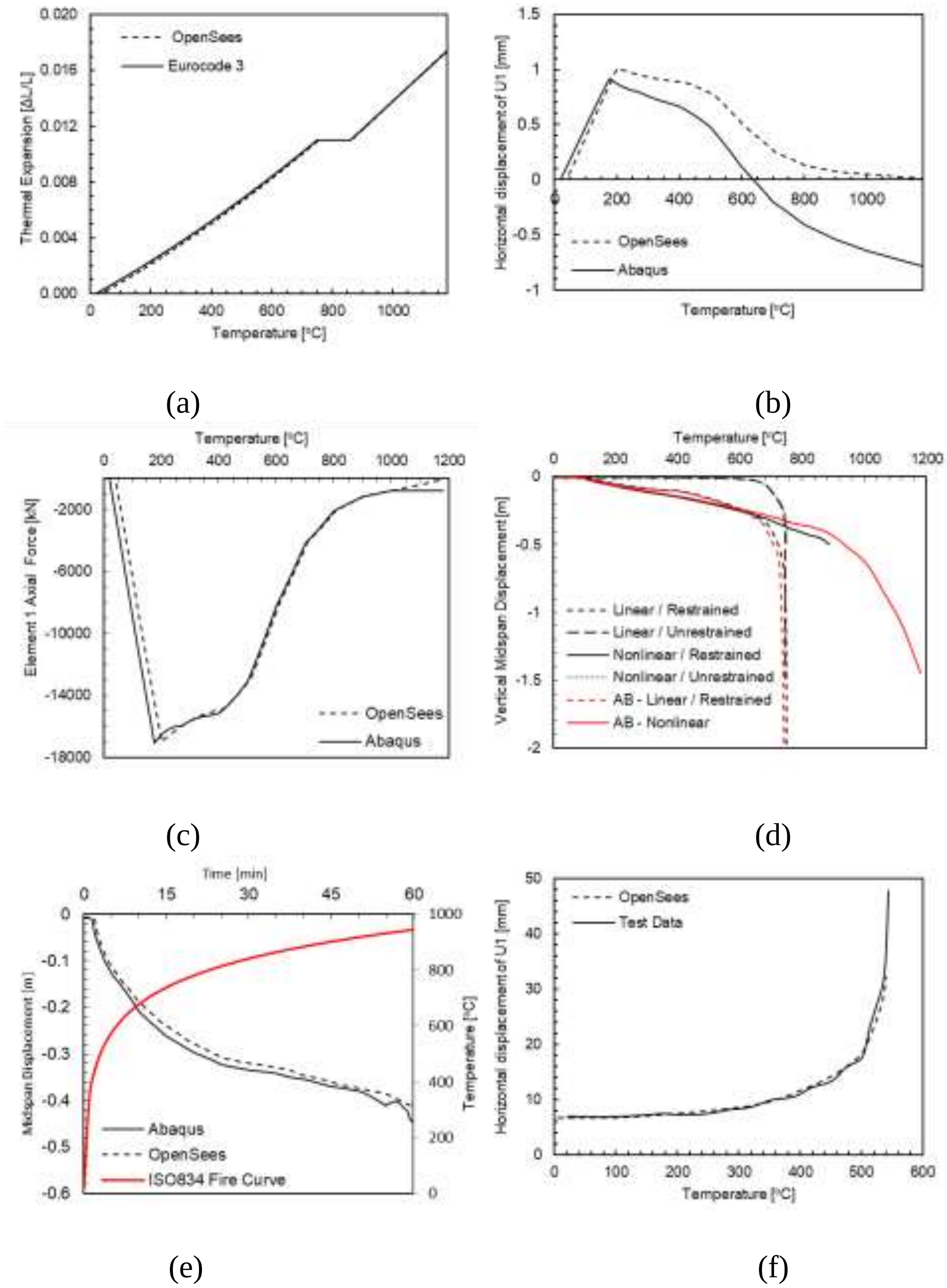


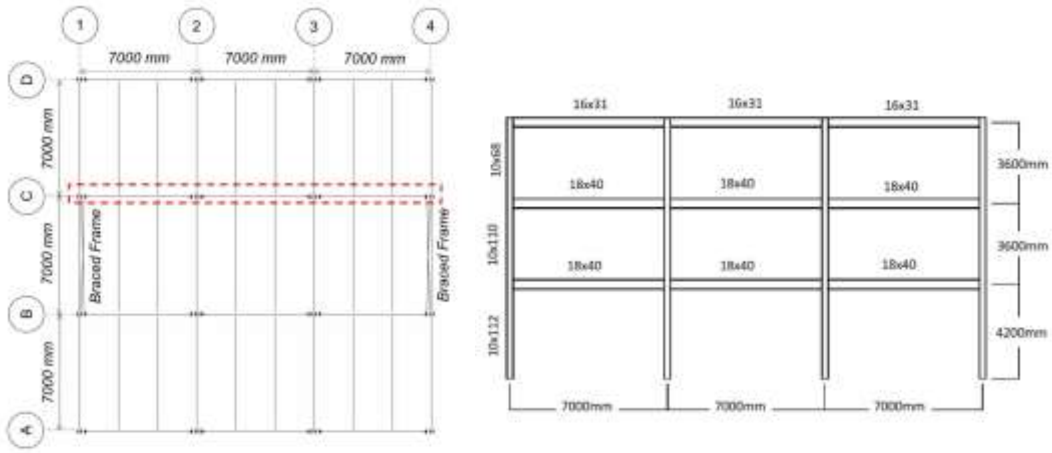
Figure 2.2. Results from validation studies compared with Abaqus and/or test data (a) Study 1, (b) Study 2, (c) and (d) Study 3, (e) Study 4, and (f) Study 5

2.3 FFE Analysis for a Multi-story Frame

The modeling methodologies developed through the validation studies were applied to perform 2D FFE and fire-only analyses of a three-story, three-bay, moment-resisting frame (MRF) (Figure 2.3). The previously designed building is of a typical low-rise, long-span office building with Occupancy Category II and an Importance Factor of 1.0 (Aksoylar et al. 2011). Figure 2.3a shows a typical floor plan of the structure, where the red boxed frame indicates the frame analyzed. Beam-to-column connections and support boundary conditions were assumed to be rigid for the analysis.

2.3.1 Modeling parameters

Displacement-based elements (*dispBeamColumnThermal*) and the *Steel01Thermal* material was used for all of the analyses in OpenSees. To account for distributed plasticity throughout the beam and column members, ten elements were used along the lengths of the beams and heights of the columns. Intermediate nodes were placed at a location of one-third the depth of the beam away from beam-column connections, where plastic hinges were predicted to form. The remaining seven beam or column nodes were located at equal distances between the locations of the nodes at the hinges. All of the steel members were grade A992 steel, with ambient temperature yield strength, F_y, and Young's Modulus, E, of 348 MPa and 210 GPa respectively. Dead loads of 3.20 kN/m^2, including self-weight, were applied to both the roof and floor levels of the building. Live loads of 1.00 kN/m^2 and 3.80 kN/m^2 were applied to roof and floor levels respectively. Gravity loads of 1.0D + 1.0L were applied during both the ground-motion and fire analysis which is consistent with previous FFE research (Khorasani et al. 2015).



(a) (b)

Figure 2.3. MRF office building used for FFE and fire-only analysis (a) plan view of a typical floor with representative MRF indicated and (b) elevation view of selected EW direction MRF

2.3.2 Ground motion Analysis

Ground motion data from the 1989 Loma Prieta earthquake was selected for earthquake analysis with a PGA of 0.479*g*, recorded at USGS station 57007. For the seismic analysis, a uniform excitation was applied to nodal masses at beam-column joints. For each analysis, gravity loading was applied then held constant for the remainder of the analysis. The seismic mass of the structure is resisted in the EW direction by four moment resisting frames. The selected frame for analysis presented in Figure 2.3 resists ¼ of the structures seismic mass.

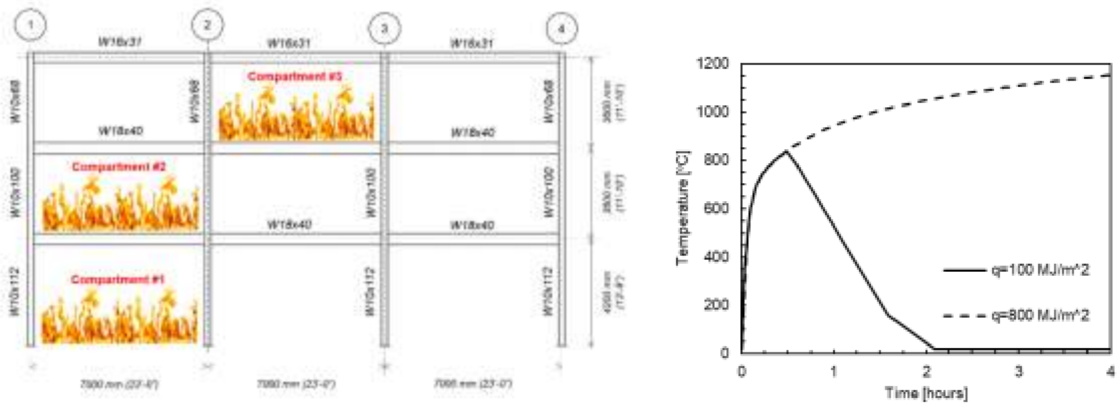
2.3.3 Fire analysis

To investigate the impact of fire on the building, the authors repeated the analysis through three different compartments using two fire scenarios. This resulted in six FFE and fire-only analyses to compare the behavior of the structural components and quantify the influence of an earthquake on the FFE performance of the LFRS. The compartment fire locations varied by floor level and are shown in Figure 2.4a. For each compartment, two parametric time-temperature curves were used to simulate a fire with low and high fuel loads, 100 and 800 MJ/mm^2 respectively (Figure 2.4b). These values represent occupancies of a transportation (public space) and dwelling for 100 and 800 MJ/m^2 respectively within the fire load densities in Table E.4 of Eurocode 1 (CEN, 2004a). These time-temperature curves were calculated using the Eurocode parametric time-temperature curve (CEN, 2004a). The authors assumed a one-zone model for the compartment where the temperature distribution through the height of the compartment was uniform.

Varying the fuel load in the compartment results in one short-duration, high-intensity fire and one long-duration, low-intensity fire. The purpose of using these two different fires is to examine the impact of the cooling phase of the fire on structural performance. As the steel temperatures increase, the heated beams will expand. Due to the thermal restraint of the surrounding cooler structure, large compression force demands will be imposed in the beams and connections. Conversely, during cooling as the beams contract, large tensile forces will develop in the connections. The inclusion of these two fire scenarios allowed the authors to examine these behaviors on the FFE performance of the LFRS.

The analysis of the fire performance of the building occurs in two phases. The first phase is a heat transfer analysis to determine the thermal distribution through the cross-sections of each member exposed to the gas temperatures shown in Figure 2.4b. The heat transfer analysis results in nodal temperatures at various points through the cross-section. This first step can occur in any commercially available finite element program. This study used Abaqus to perform the heat transfer analysis. The second step uses the nodal temperatures as input for the stress-based analysis performed in OpenSees.

In the heat transfer analysis, the column and beam members were assumed to have no fire protection. It was assumed that an earthquake would damage the spray-applied fire protection. The beam heat transfer analysis is performed including the effects of the 203 mm thick concrete slab. Columns were heated on three sides to simulate the effects of the compartment fire. Red dashed lines in Figure 2.5 represent temperature input locations labeled Y1-Y9 for the OpenSees model. OpenSees linearly interpolates temperatures between input locations to create a thermal gradient throughout the fibered section.



(a) (b)

Figure 2.4. Compartment fires used in the analyses (a) Location of Compartment Fires and (b) gas time-temperature curves for low and high fuel load fires

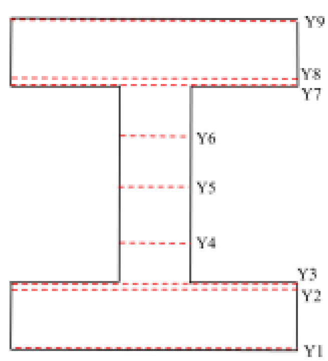
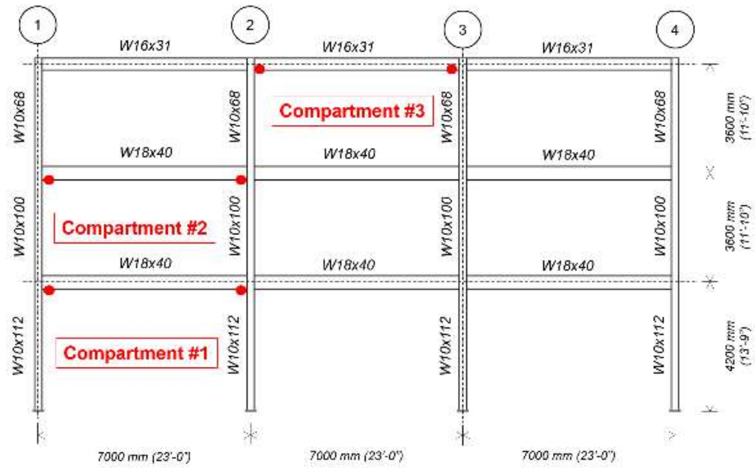


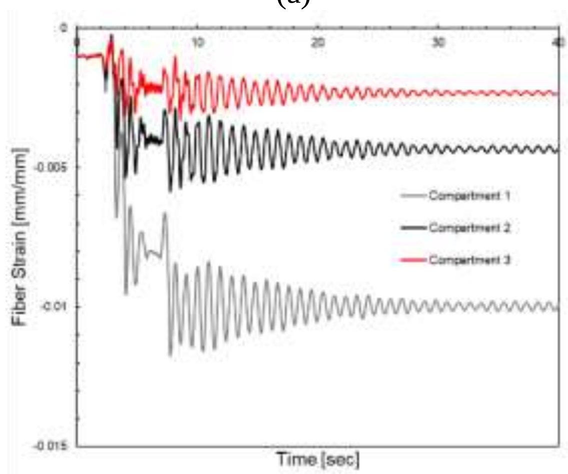
Figure 2.5. Beam and column nodal temperature locations for OpenSees

2.3.4 Results of 2D OpenSees FFE and Fire-only Analyses

To quantify the influence of an earthquake on the FFE performance of an MRF LFRS, the authors performed six FFE and six fire-only analyses in OpenSees. Fiber strains were recorded at nodes on the beam at a distance of one-third of the beam depth from the beam-column joints during the seismic analysis only. These locations represented the plastic hinge regions of the MRF beams. The locations of these regions are shown in Figure 2.6a. The recorded strains, shown in Figure 2.6b showed that the yielding of the plastic hinge region occurred during the earthquake. Beams in Compartments 1 and 2 experienced permanent plastic strains near the beam-column connection of 0.0045 and 0.0 respectively. The third compartment's beam developed permanent strains of 0.0023, significantly less than Compartments 1 and 2. The comparison of these strains provides context to the damage state of the building *prior* to the compartment fires.



(a)



(b)

Figure 2.6. Strain at plastic hinge locations in LFRS beams (a) Locations of extreme fiber strain recorders and (b) Extreme fiber strains LFRS beams

During the fire, vertical displacements were recorded at the mid-span nodes of each compartment beam for FFE and fire-only with both high and low fuel loads. A comparison of mid-span displacements for high and low fuel loads during an FFE analysis and a fire-only analysis is shown in Figure 2.7. The displacements from a low fuel fire can be seen in red, while high fuel can be seen in black. FFE analysis is represented by solid lines and fire-only analysis by dashed lines. Initial displacements from the ground motion analysis are zeroed before plotting FFE in order to compare fire analysis separately from the initial strain state.

The plots shown in Figure 2.7 show that regardless of the large plastic strains in the LFRS, the displacements of the beams in an FFE versus a fire-only analysis are about the same. During a fire, the only load demands on the beams are due to gravity and the thermal expansion of the members. The gravity demands are significantly lower than the demands the beams of MRFs are designed for when including seismic loading. For the W18x40 beams in Compartments 1 and 2, the demand-to-capacity (DCR) ratios of the beam for flexure are 0.25 at the midspan and 0.50 at the beam ends, not considering the contribution of the composite deck. For the W16x31 beam in Compartment 3, the DCR of the beam for flexure is 0.22 at the midspan and 0.43 at the beam ends, not considering the contribution of the composite deck. The significant excess gravity capacity of the beams within an MRF provides additional resistance for the fire demands. Therefore, plastic strains in the beams caused by an earthquake will not significantly affect the performance of the MRF beams in a subsequent fire. This is consistent with the results from other FFE analyses of MRF LFRS (Chicchi 2017; Khorasani et al. 2015).

The magnitude of the strain at the plastic hinge location will slightly influence the behavior of the beam in an FFE. As the strain in the beam decreased from Compartments 1 to 3 (as shown in Figure 2.6b), the similarity between the two deflections (fire-only versus FFE) increased. Figure 2.7c shows that when there are small amounts of plastic strains in the beams, this almost has no influence over the behavior of the beam in an FFE versus a fire-only analysis.

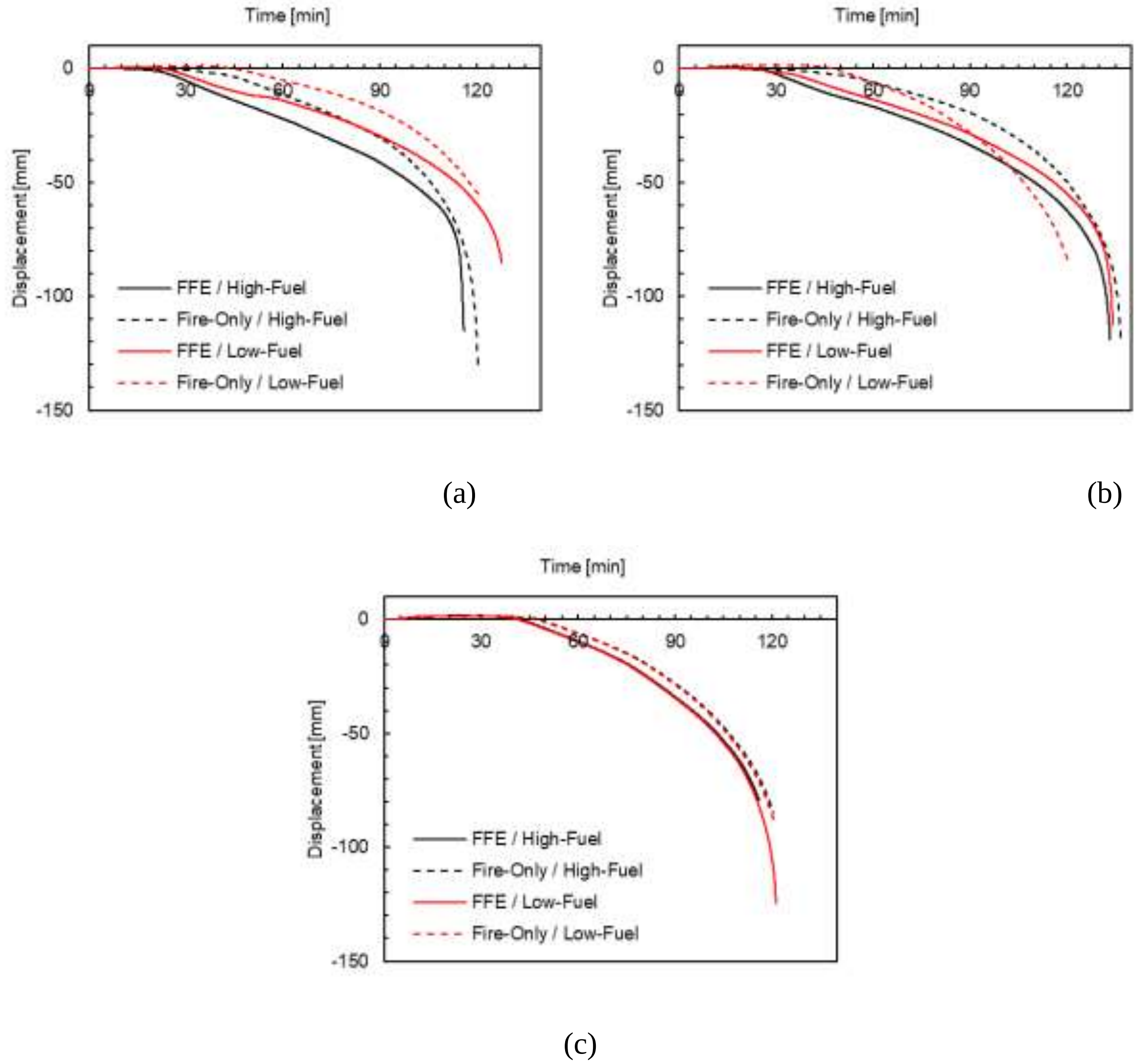


Figure 2.7. Mid-span displacement of beams subjected to compartment fires (a) Compartment 1, (b) Compartment 2, and (c) Compartment 3

2.4 Summary and Conclusions

The authors performed a series of validation studies to demonstrate the ability of OpenSees to perform structural fire engineering and FFE analyses. The results of these studies showed that OpenSees was able to simulate the displacement of structural members and calculate the internal forces in members due to a fire. The authors used the modeling capabilities developed in these validation studies to perform an investigation into the FFE and fire-only behavior of a 2D MRF LFRS of a three-story steel-frame building. The results of the study showed that due to the low gravity demands on the beams of MRFs, the fire-only and FFE behavior of the beams were similar, regardless of the level of plastic strain developed at the hinge locations during an earthquake. As the magnitude of the plastic strain developed in the hinge during the seismic analyses decreased, the similarity between the FFE and fire-only analysis increased. The authors tested this hypothesis on two different types of fires (low and high fuel load) to consider the duration of fire exposure as a parameter.

The results of this study are consistent with the results of previous FFE analyses (Chicchi 2017; Khorasani et al. 2015) and highlight the need for further analysis of steel-frame buildings both subjected to fire-only and FFE with the inclusion of the gravity resisting framing.

2.5 Acknowledgments

A special acknowledgment to Dr. Michael Scott, professor at Oregon State University, for his continued help on this research.

CHAPTER 3: Fire Following Earthquake Incremental Analysis

3.1 Introduction

Cascading events such as fire following earthquakes (FFE) can lead to mass destruction of property and loss of life. A large seismic event can lead to an increased risk for fire. Ground motions can cause damage to gas or other fuel utility lines potentially causing ignition. Additionally, water utilities can be damaged leading to issues for fire suppression services.

In the past century FFE events such as the 1906 San Francisco earthquake (Humphrey et al. 1907), 1995 Kobe earthquake (Himoto, 2019), 2012 Tohoku earthquake and tsunami (Himoto, 2019), and the 2019 Ridgecrest earthquake (Fischer et al. 2019) have caused a multitude of damages worldwide. Scenario exercises have been performed in the California Bay Area and Portland, Oregon to determine potential threats to FFE. The HayWired Earthquake Scenario found that an FFE threat to the Bay Area could potentially destroy 52,000 single-family homes (Hudnut et al. 2018). A similar evaluation was conducted in Portland, Oregon by The Portland Bureau of Emergency Management concluded that a Cascadia subduction-zone event could lead to fires from damaged gas and fuel lines. Moreover, damage to water utilities will would significantly affect firefighter crews' ability to fight fire spread.

Compartmentalization of a fire is critical in preventing fire spread along a floor plate or vertically within a building. The objective of fire protection design in buildings is to maintain compartmentalization to prevent fire spread throughout a floor or vertically throughout a building. Therefore, the behavior of both the lateral and gravity framing systems is critical during a fire. Loss of compartmentalization can occur when there is a partial collapse of a building or when structural members undergo large deformations thereby damaging the compartment walls.

3.1 Background

3.2.1 Previous computational analysis on fire following earthquake

Just over a decade ago, Usmani (2008) outlined the research needs to evaluate structural performance of steel structures in fire after following an earthquake event. Because of the complexity of issues surrounding FFE performance, categorizing FFE demands into a risk-based framework would create a compressive characterization of all the uncertainties related to FFE and fire engineering.

Zaharia and Pintea (2009) performed pushover analysis to determine plastic deformation from a seismic event before performing fire analysis. Standard fire (ISO, 2014) and natural fire curves were used for fire hazards in the analysis. This study investigated two frames that were exposed to two levels of seismic hazard, a near and far-field seismic event. The initial structural damage state used during the fire analysis was determined from the previous push-over analysis. The frames that experienced less damage from the pushover analysis maintained load carrying capacity for a longer period during fire loading.

Behnam and Ronach (2014) performed FFE analysis on a five-story steel MRF structure that was designed for two performance levels, immediate occupancy (IO) and life safety (LS). Pushover analysis was performed in SAP 2000, which was then imputed into the software SAFIR for thermal loading. For fire hazards in this study, standard ISO 834 (ISO, 2014) and Eurocode natural fire curves (CEN, 2005a) were considered. The structure designed with a higher fire resistance (IO) performed better than the LS structure. The structures exposed to FFE failed in half the amount of fire exposed time than structures exposed to fire-only.

Khorasani et al. (2015) developed a reliability-based methodology to analyze a 10 story steel moment-resisting frame (MRF) building in FFE. Analysis was performed in OpenSees and compared FFE behavior to fire-only analysis. The OpenSees model used *dispBeamColumnThermal* elements to simulate the behavior of the MRF elements. The fire applied to the structure was based on the 1MP fire event (Garlock and Quiel 2007) and ground motions were considered from the Loma Prieta earthquake. Comparisons with the fire-only and FFE analyses showed similar results to other researchers in that the behavior of the MRF was similar in the fire-only and the FFE analyses. Khorasani et al. (2015) also concluded that the fire that follows an earthquake can increase the lateral drifts of exterior columns while it decreases the lateral drift of interior columns due to thermal expansion of the beams.

3.2.2 Incremental Fire and Dynamic Analysis (IFA & IDA)

The cascading hazards of FFE's have an inherent level of uncertainty. Moss et al. (2014) proposed modifying the previously used earthquake engineering process of Incremental Dynamic Analysis (IDA) into a methodology for fire analysis called Incremental Fire Analysis (IFA). Similar to IDA, where a suite of scaled ground motions are used, a suite of scaled fire loads are applied to the model. In addition, similar to IDA, an intensity measure (IM), is increased incrementally while an engineering demand parameter (EDP) is selected to be compared to an individual IM. Unfortunately, unlike in IDA, the fire engineering research community has yet to form a consensus on the proper IM to use for IFA. Previous research has incorporated peak compartment gas temperature (Lange et al. 2014), area under the time-temperature curve (Devaney, 2014), radiant heat energy (RHE) (Moss et al. 2014), and total compartment fire load (Gernay et al. 2016).

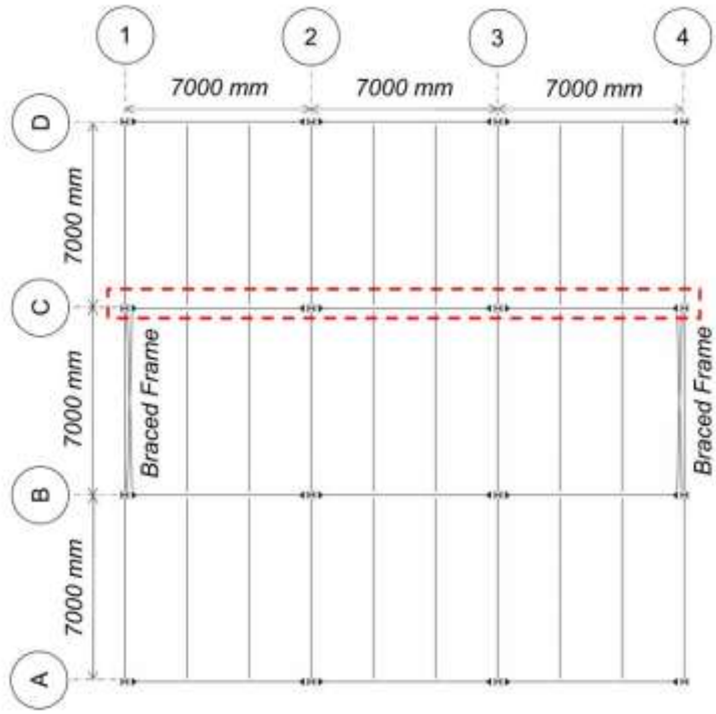
3.3 Objectives

The objective of this study is to perform a comprehensive FFE analysis on a two-dimensional (2D) MRF using IDA and IFA to quantify the damage to steel MRFs in an FFE. The FFE analysis will be performed on the first story compartment of the 2D MRF discussed in Chapter 2. This compartment performed the worst in the initial FFE analysis discussed in Chapter 2 and therefore will represent the worst-case compartment fire behavior and FFE behavior.

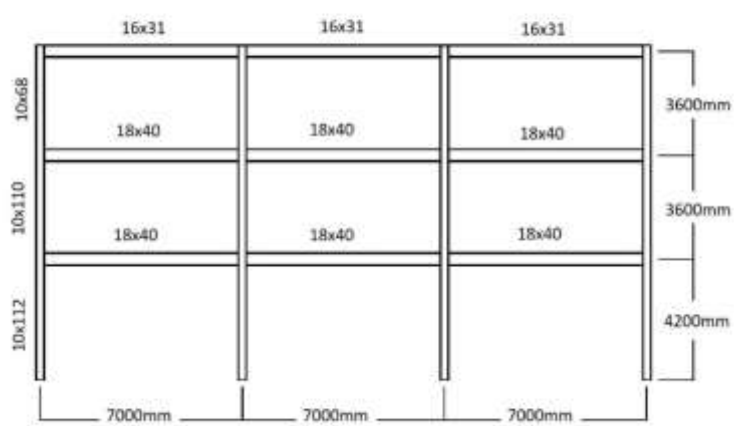
3.4 Modeling methodology

The structure used in this case study incorporated a previously designed building consisting of a three-story, long-span office building of Occupancy Category II with an Importance Factor of 1.0 (Aksoylar et al. 2011). The structure is composed of two-span lengths of 7 and 9 meters with story heights of 4.2 meters for the 1st story and 3.6 meters for the 2nd and 3rd stories respectively (Figure 3.1b). The lateral force resisting system (LFRS) of the building consists of a special moment-resisting frame (MRF) in the East-West (EW) direction and a braced frame in the North-West (NW) direction. The 2D analysis was performed on a special MRF in the EW direction. The MRF analyzed is boxed in red (Figure 3.1a).

Previous FFE analyses were completed by the authors on the 2D frame to locate possible vulnerabilities and compartments of interest (Maddalozzo and Fischer 2020). It was concluded that plastic deformation and damage was focused in the first-story exterior bay of the MRF. The first story exterior bay was only considered for the analyses described in this chapter.



(a)



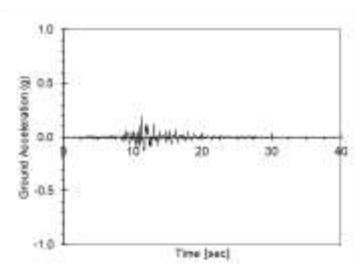
(b)

Figure 3.1. MRF office building used for FFE and fire-only analysis (a) plan view of a typical floor with representative MRF indicated and (b) elevation view of selected EW direction MRF

3.4.1 Earthquake Selection and Scaling

Ground motions were selected from the 1989 Loma Prieta earthquake via the PEER NGA-West 2 ground motion database (Chiou et al. 2008). Ground motions were recorded in downtown Oakland, California at station Oakland - Title & Trust (Latitude: 37.806, Longitude: -122.267), on site class D soils. Because the analysis is 2D, only one the horizontal component of the recorded ground motions was applied during the seismic analysis portion. This earthquake ground motion was chosen so that the authors could compare the results of this study to previous FFE studies (Khorasani et al. 2015) and build upon the conclusions made by previous researchers.

For the IDA analysis, the IM is Peak Ground Acceleration (PGA) and the EDP is peak roof story drift. The IM value was scaled to 1, 3, and 5 times the recorded value. The selected scaled ground motions are displayed below in Figure 3.2, where Figure 3.2a is the original Loma Prieta ground motions, Figure 3.2b is the ground motion scaled to three times the PGA, and Figure 3.2c is the ground motion scaled to five times the PGA. These scaled PGAs correspond to 0.2g, 0.59g, and 0.98g for 1, 3, and 5 times the recorded values, respectively.



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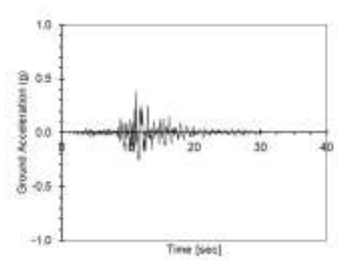

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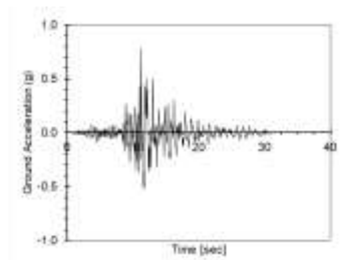


(c)

Figure 3.2. Ground motion records used for IDA (a) Original Loma Prieta ground-motion (b) 3 times scaled ground motion (c) 5 times scaled ground-motion

3.4.2 Fire Selection and Scaling

The compartment fire considered in this analysis represented a post-flashover fire. The fire time-temperature curve was calculated using Annex A of Eurocode 1 (CEN, 2004a), parametric time-temperature (*T*-*t*) curves. The Eurocode parametric fire was selected because it consists of both a heating and cooling portions, as opposed to the standard fire curves such as ASTM E119 (ASTM, 2018) and ISO 834 (ISO, 2014). In addition, the Eurocode parametric fire *T*-*t* curve is calculated considering characteristics specific to the compartment such as ventilation, fire load density ($q_{t,d}$), and thermal inertia of the enclosure (b). The compartment chosen for this analysis was a first-floor compartment, shown in Figure 3.3.

Based on previous research (Moss et al. 2014), a suite of fire *T*-*t* curves were created varying the ventilation and fire load density, while maintaining a constant thermal inertia of the enclosure. A total of 12 fire curves were generated using four opening factors (representing ventilation): 0.02, 0.04, 0.08, and 0.12 $m^{1/2}$, and varying fuel loads: 200, 400, and 800 MJ/m^2. From the twelve, ten fire curves were selected F1-F10 (two fire curves were discarded as the selected parameters created redundant fire curves) for scaling and all resulting fire curves varied in length and maximum temperature displayed in Figure 3.4a. The opening factors and fuel loads used to create fires F1-F10 are shown in Table 3.1. The two fires that were discarded had an opening fator of 0.08 $m^{1/2}$ and fuel loads of 400 and 800 MJ/m^2.

The fires generated and selected were scaled to a maximum fire temperature of 600^{o}C, 800^{o}C, and 1000^{o}C (Figure 3.4b-d). The EPD selected for the IFA was beam midspan displacement.

Table 3.1. Eurocode parametric fire curve paramters used to create fires F1-F10

Fire curve Parameters	F1	F2	F3	F4	F5	F6	F7	F8	F9	F10
Opening Factor [$m^{1/2}$]	0.02	0.02	0.02	0.04	0.04	0.04	0.08	0.08	0.12	0.12
Fuel Load [MJ/m^2]	200	400	800	200	400	800	200	400	200	400

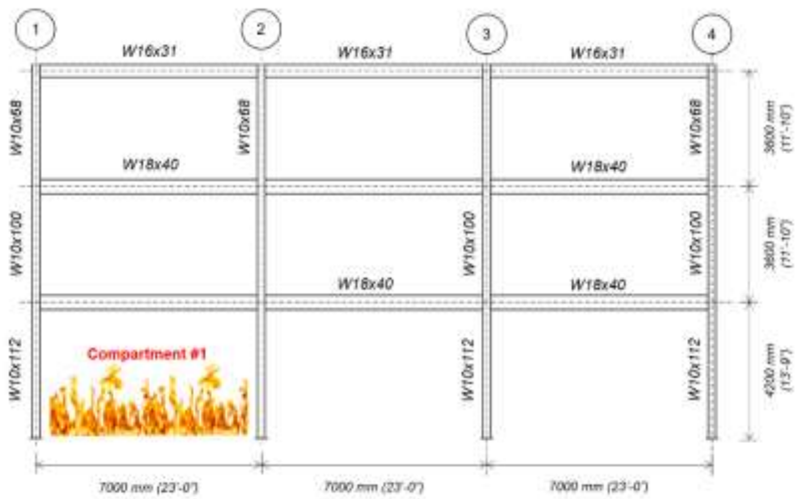
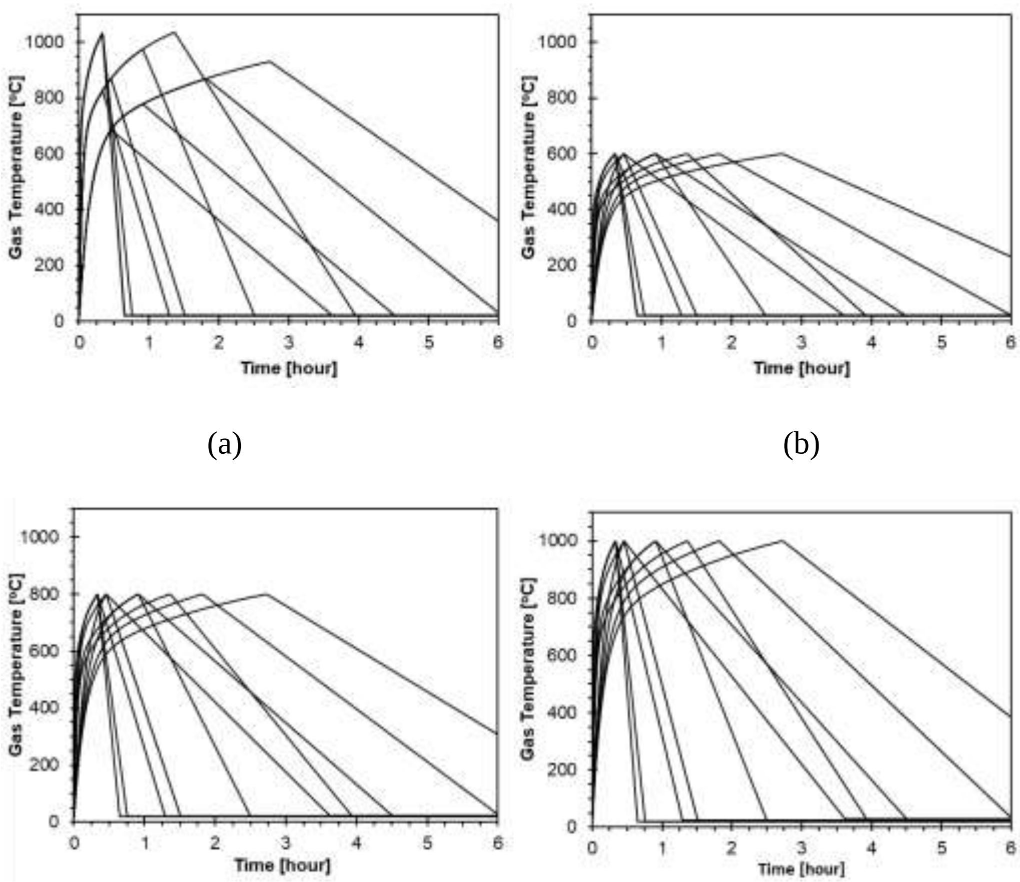


Figure 3.3. Compartment fire location used in the analyses



(a) (b)

(c) (d)

Figure 3.4. Fire time-temperature (*T-t*) curves used for the IFA analysis (a) unscaled gas temperatures, (b) gas temperatures scaled to 600ºC, (c) gas temperatures scaled to 800ºC, and (d) gas temperatures scaled to 1000ºC

3.4.3 Seismic Modeling

The three-story building under dynamic load was simulated using displacement-based elements in each member (*dispBeamColumnThermal*) including the *Steel01Thermal* material model. The seismic mass of the structure is resisted in the EW direction by four moment resisting frames. The selected frame for analysis presented in Figure 3.1.b resists ¼ of the structures seismic mass. To account for distributed plasticity throughout the beam and column members, ten elements were used along the lengths of the beams and heights of the columns. Intermediate nodes were placed at a location of one-third the depth of the beam away from beam-column connections on both the beams and columns. This location of the nodes denotes where plastic hinges were predicted to form in the beam and panel zones in the columns. The remaining seven beam or column nodes were located at equal distances between the locations of the nodes at the hinges.

The MRF connections are modeled as purely rigid connections. Yang et al. (2009) concluded following experimental testing that moment connections can maintain their capacity design strength up to 650°C resulting in a 25% reduction of stiffness. These findings were used to justify the assumed idealized rigid moment connections throughout the analyses. P-Delta effects were not considered throughout the seismic analysis.

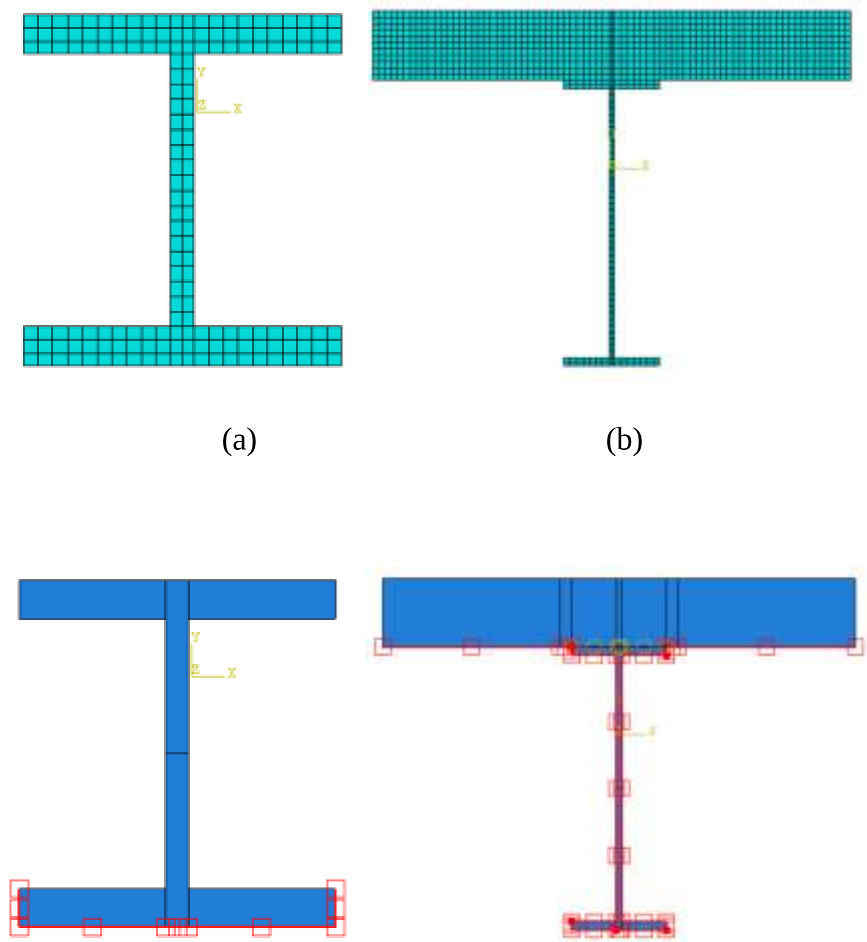
3.4.4 Heat Transfer Analysis

To obtain the temperature histories of the structural members in response to the fires shown in Figure 3.4, 2D heat transfer analyses were performed using the commercially available finite element (FE) software Abaqus (Abaqus, 2019). A one-zone model was assumed for the compartment fires, therefore gas temperature is constant through the height of the compartment. The heat transfer analyses were performed without fire protection on the MRF beam and columns. It was assumed that the spray-applied fire protection would be damaged during the ground motions of the earthquake and would not be present during fire.

The 2D heat transfer analyses on each MRF member were performed using convection, conduction, and radiative heat transfer. Convective and radiative heat transfer accounted for the heat transfer between the hot gasses and the exposed MRF steel members; whereas, conductive heat transfer was used to account for heat transfer depth of each of the MRF cross-sections. The structural components were simulated using 4-node, 2D heat transfer DC2D4 elements. The nodal temperatures through the cross-section of each of the MRF members was calculated using two elements through the thickness of the flange or web of the steel MRF members, as shown in Figure 3.5.

Convective and radiative heat transfer occurred at the fire-exposed surfaces. For the MRF columns, it is assumed that the compartmentation remains intact throughout the earthquake and fire, therefore, the column is exposed to the fire from one side only (Figure 3.5c). For the MRF beam, the fire exposed surface is the perimeter of the beam and the underside of the concrete slab (Figure 3.5d). While the 203 mm concrete slab was not explicitly modeled in the stress-based analysis, the thermal gradient through the beam is dependent upon the presence of the concrete slab, which acts as a heat sink. Therefore, shown in Figures 3.5b and 3.5d, the concrete slab was simulated in the heat transfer analyses.

Temperature-dependent thermal material properties were based on Eurocode for both steel and concrete (CEN 2004b; CEN 2004c). This included thermal conductivity, specific heat, thermal elongation, and density.



(a) (b)

(c) (d)

Figure 3.5. Abaqus heat transfer model overview (a) beam mesh, (b) column mesh, (c) beam heated surfaces, and (d) column heated surfaces

The nodal time-temperature histories from the heat transfer analysis were recorded at seven locations throughout the depth of the steel column and beam sections in the 2D heat transfer analysis (Y1 – Y7) as shown in Figure 3.6. Locations Y1-Y7 from top to bottom were selected at the top and bottom edges of each flange and at quarter and half points of the web sections. For the six-hour design fire duration, nodal temperatures were recorded at one-minute intervals resulting in 360 temperature data points for each node. Two selected examples of the nodal temperature outputs from the 2D heat transfer analyses are presented in Figure 3.7. Where Figure 3.7a are the nodal temperature histories in response to Fire 1 (F1) where gas temperatures are scaled to 1000°C and Figure 3.7b are the nodal temperature histories in response to Fire 5 (F4) where nodal gas temperatures scaled to 600°C. In both fires, the nodal temperatures show a temperature distribution through the cross-section of the member caused by the presence of the concrete slab acting as a heat sink.

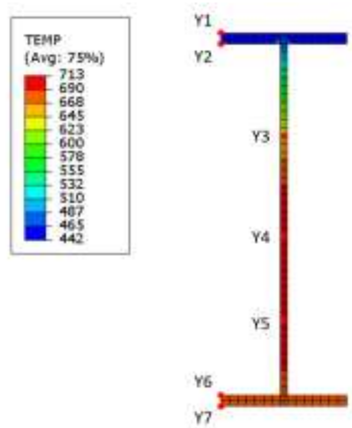
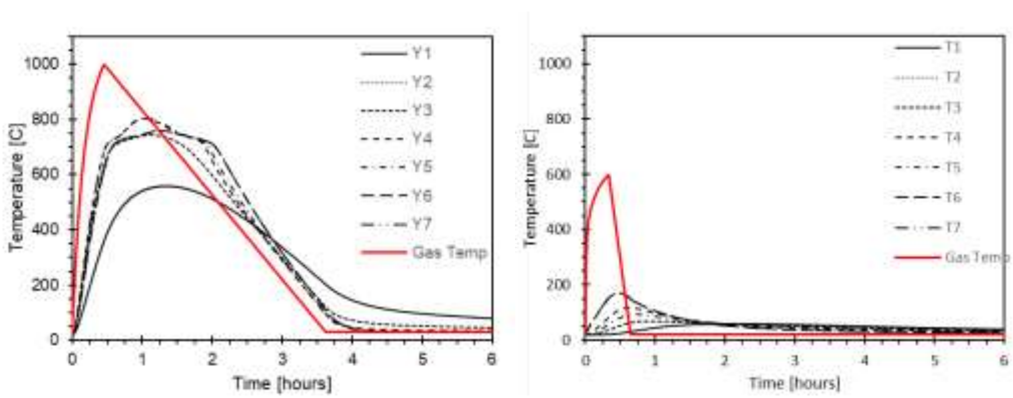


Figure 3.6. Nodal temperature locations Y1-Y7 from Abaqus heat transfer



(a) (b)

Figure 3.7. (a) Fire 1 (F1) nodal gas temperatures scaled to 1000°C (b) Fire 5 (F4) nodal gas temperatures scaled to 600°C

3.4.5 FFE Modelling Approach

The nodal temperatures calculated using the 2D heat transfer analysis were used as input into the 2D FFE and fire-only analysis to simulate the behavior of the 2D MRF. The OpenSees model was built with 10 displacement-based elements in each member (*dispBeamColumnThermal*) including the *Steel01Thermal* material model. To record and model plastic hinging behavior, additional nodes were modeled at one-third the depth of the beam away from beam-column connections.

All members of the frame were modeled with grade A992 steel, with ambient temperature material properties of yield strength, F_y, and Young's Modulus, E, of 348 MPa and 210 GPa respectively. Dead loads of 3.20 kN/m^2, including self-weight, were applied to roof and floor levels of the building. Furthermore, live loads of 1.00 kN/m^2 and 3.80 kN/m^2 were applied to roof and floor levels respectively. Consistent with previous FFE research (Khorasani et al. 2015), unscaled gravity loads using Equation 1 were applied during both phases of analysis, earthquake loading followed by fire loading.

$$1.0D + 1.0L \qquad \text{Equation 1}$$

3.5 Results and Discussion

To investigate the comparison of FFE to fire-only behavior of steel MRFs, a suite of FFE and fire-only analyses was performed. Three scaled ground motions in conjunction with 10 parametric fire curves scaled to three maximum temperatures were considered.

3.5.1 Earthquake Only Analysis

To better understand the initial damage state from the ground motion accelerations prior to FFE analysis, inter-story drifts were recorded for the three scaled ground motions, 0.20g, 0.59g, 0.98g respectively. First story Inter-story drifts for two scaled (0.59g and 0.98g) ground motions were 0.6% and 0.62% (Figure 3.8). These drifts are significantly less than the typical design drifts for buildings (ASCE 2016) using a life safety performance objective. With residual story drifts after ground motions less than one mm, indicating that the structure remained elastic throughout each scaled ground-motion.

Strains were recorded in the beams at one-third of the beam depth from beam-column joints (proposed location of the plastic hinges) in the compartment where the fire would occur, as shown in Figure 3.9. Maximum strains from ground-motion loading in the beams were -0.0013, 0.0017, 0.002 in/in for (0.20g, 0.59g, and 0.98 ground-motions, respectively. The nominal yield strain for the steel is 0.0017 in/in. Therefore, these strains indicate that the beams remained elastic throughout the first two ground motions (1 and 3 times PGA) and may have experienced plasticity throughout the last ground motion.

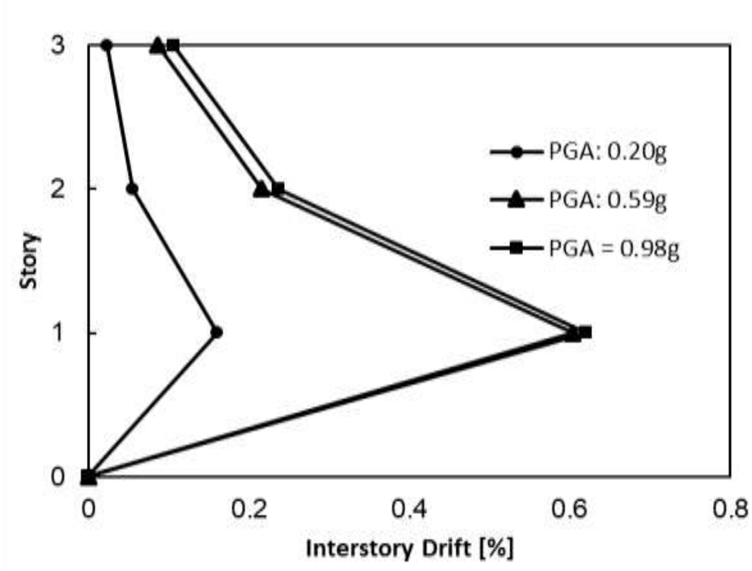


Figure 3.8. Inter-story drift ratios for scaled IDA ground motions only analysis

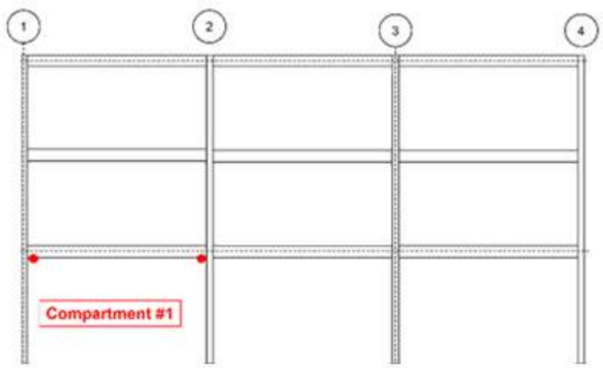
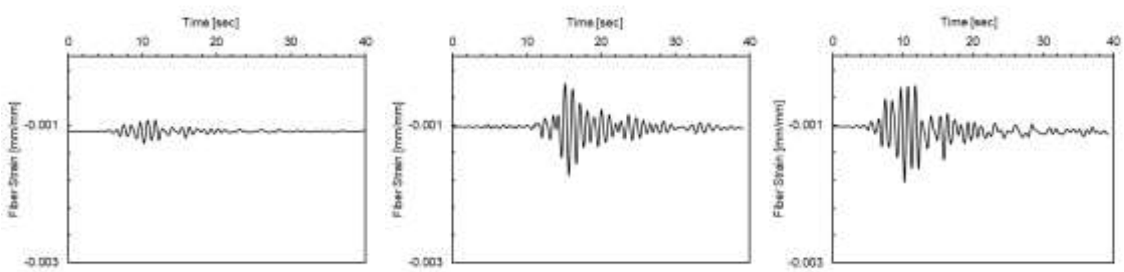


Figure 3.9. Locations of extreme fiber strain recorded in compartment one LFR beam



(a) (b) (c)

Figure 3.10. Extreme fiber strains of LFRS Beam for (a) 1 (b) 3, and (c) 5 times scaled ground motions

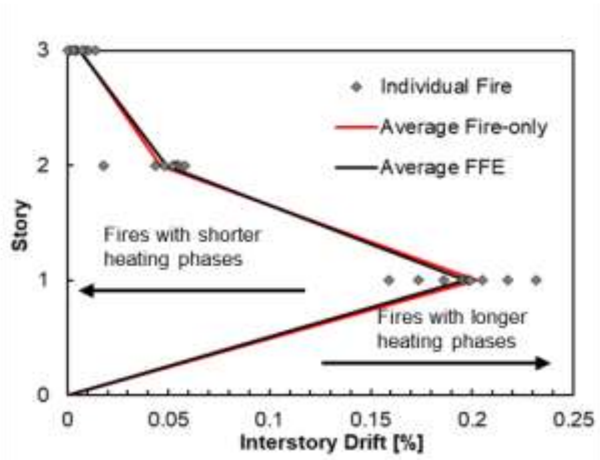
3.5.1 Fire-only and FFE Analysis

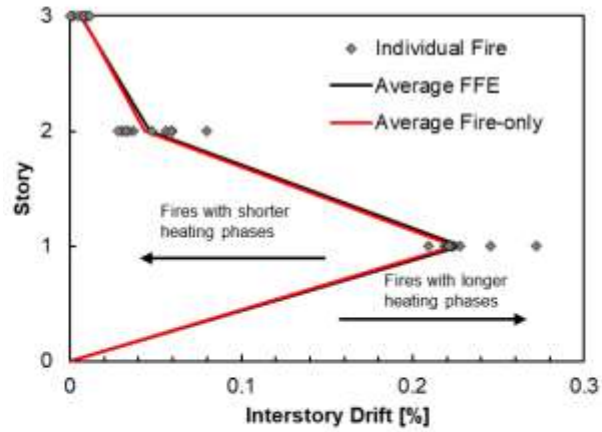
Global behavior

Both global and local behavior was recorded during the FFE and fire-only analyses. Inter-story drifts are displayed for the IDA scaled 0.2g PGA ground motion in Figure 3.11. These figures compare the average inter-story drifts of the fire-only and the FFE analysis. The red line is the average of the fire-only analyses results and the black line is the average of the FFE analyses results. Additionally, the grey diamonds in Figure 3.11 represent individual data points from the 10 fire curves, F1-F10. These plots show that there is less variability of inter-story drift for the fires scaled at 1000^{o}C than the other scaled temperatures. This can be seen by the lack of scatter of the grey diamonds at each story in Figure 3.11c. In addition, regardless of the scaled temperature of the fire, there was less variability in the inter-story drift at the third story than the other stories. This is seen again through the dispersion of grey diamonds at each of the stories shown in Figures 3.11a-c. Lastly, the fires scaled to 600^{o}C resulted in a larger variability of inter-story drifts at the first story whereas the fires scaled to 800^{o}C resulted in a larger variability of inter-story drifts at the second story. The inter-story drift plots in Figure 3.11 show that regardless of the severity of the fire in these analyses, the behavior of FFE and fire-only are very similar during the 0.2g ground motions. The same observations can be made by viewing the data from the 0.58g and 0.98g ground motions in Table 3.2.

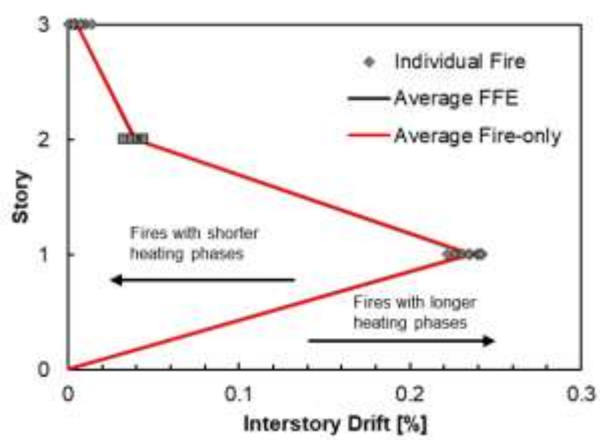
The inter-story drift data is also presented in Table 3.2, comparing FFE to fire-only performance. The results presented in Figure 3.11 and Table 3.2 show that the influence on previous ground motions, regardless of PGA, had a negligible effect on the inter-story drift measured during the fire-only and FFE analysis. However, temperature did have an impact on inter-story drift. For the 0.2g, 0.59g, and 0.98g, the average maximum first-story inter-story drift increased by 26%, 21%, and 15%, respectively when the maximum temperature of the fires increased from 600^{o}C to 1000^{o}C. In comparison, for the fire-only analysis, the inter-story drift increased by 20% when the maximum temperature of the fires increased from 600^{o}C to 1000^{o}C. On average fires with a heating period longer than 30 minutes experienced inter-story drifts of 8%, -1%, and -3% larger for fires at 600^{o}C, 800^{o}C, and 1000^{o}C, respectively for the FFE and fire-only analyses.

The greatest difference between FFE and fire-only structural behavior was for the 600^{o}C maximum temperature fires, where the fire-only case had an 2.9% larger inter-story drift than the 0.98g ground motion coupled with the 600^{o}C scaled maximum temperature fires. The second largest difference between the FFE and fire-only structural behavior was for the 800^{o}C maximum temperature fires, where the fire-only case had a 4.5% larger inter-story drift than the 0.98g ground motion coupled with the 800^{o}C scale maximum temperature fires. The remainder of the FFE analyses were within 2% or less of the fire-only structural behavior.





(a) (b)



(c)

Figure 3.11. Inter-story drifts from FFE analysis for 0.2g PGA for scaled maximum gas temperatures of (a) 600ºC, (b) 800ºC, and (c) 1000ºC

Table 3.2. Summary of average maximum first-story inter-story drifts during FFE and fire-only analysis

Scaled Temp. [°C]	Fire Following Earthquake (FFE) inter-story drift			Fire-only inter-story drift	Percent difference of FFE from fire-only analysis		
	0.2g	0.58g	0.98g		0.2g	0.58g	0.98g
600	0.19	0.19	0.20	0.20	-2.8%	-2.6%	-2.6%
800	0.23	0.22	0.22	0.22	1.9%	-0.8%	-0.8%
1000	0.24	0.23	0.23	0.24	-0.4%	-0.7%	-0.7%

Local behavior

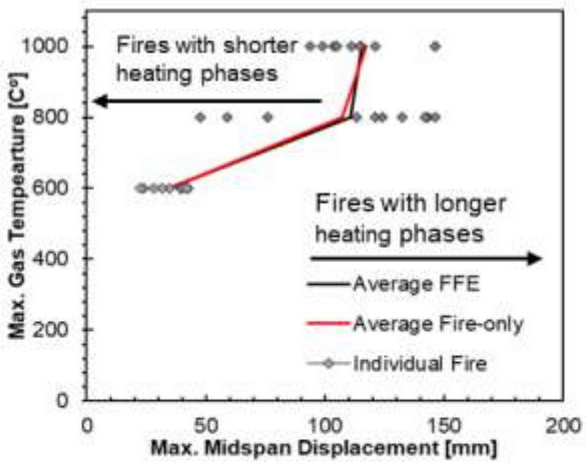
Local behavior was measured via the maximum midspan displacement of the MRF beams. Figure 3.12 presents comparisons of maximum midspan displacements for fire-only (red lines) and FFE (black lines) scenarios. Additionally, the grey diamonds in Figure 3.12 represent individual data points from the 10 fire curves F1-F10 at each scaled level in Figure 3.12a-c. These figures show that as the temperature of the fire increases, the beam midspan displacement increases. For FFE analyses of 0.2g, 0.59g, and 0.98g, the average midspan displacement increases by 235%, 282%, and 243%, respectively as the maximum fire temperature increases from 600^{o}C to 1000^{o}C. In contrast for the fire-only analysis, the average midspan displacement increases by 250% as the maximum fire temperature increases from 600^{o}C to 1000^{o}C.

Figure 3.12 also shows that beam midspan displacement is highly dependent upon the duration of the heating phase of the fire. The grey diamonds to the left of the red and black lines represent beam midspan displacement due to fires with shorter duration heating phases, whereas the grey diamonds to the right represent beam midspan displacement due to fires with longer duration heating phases. For the 800^{o}C scaled fires, individual fires that fell to the left of the average lines in Figure 3.12, were fires with a heating phase of less than 30 minutes. On average, fires with a heating phase longer than 30 minutes caused beam midspan displacement of 13, 57, and 4 mm for fires with maximum temperatures of 600°C, 800°C, and 1000°C, respectively than fires with heating phases less than 30 minutes. These midspan displacements were larger than the overall average midspan displacements for all of the fires with heating phases less than 30 minutes by 49%, 75%, and 7% for fires with maximum temperatures of 600°C, 800°C, and 1000°C, respectively.

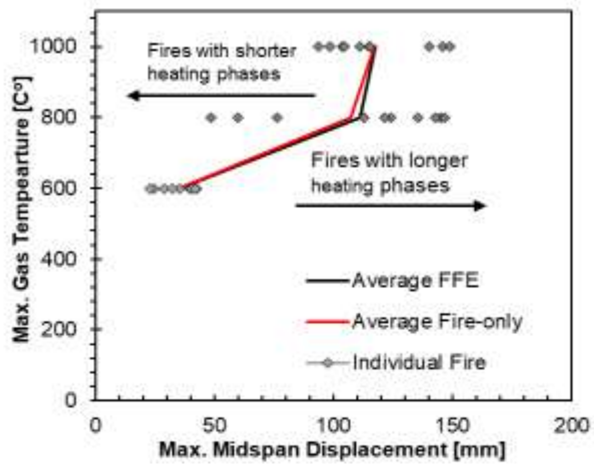
The minor difference between fires at 1000°C could be explained by the rapid heating phase causing beam failure. Duration of heating has a larger impact on beam midspan displacement than on inter-story drifts.

Midspan beam displacement data comparing FFE to fire-only behavior is also presented in Table 3.2. Similar to the global behavior of the structure, there was minimal difference in maximum midspan beam displacements between fire-only and FFE analyses. The greatest difference between FFE and fire-only structural behavior was for the 600°C maximum temperature fires, where the average of the 0.58g ground motion FFE cases resulted in a 5.1% larger midspan displacement than the average of the fire-only scenarios.

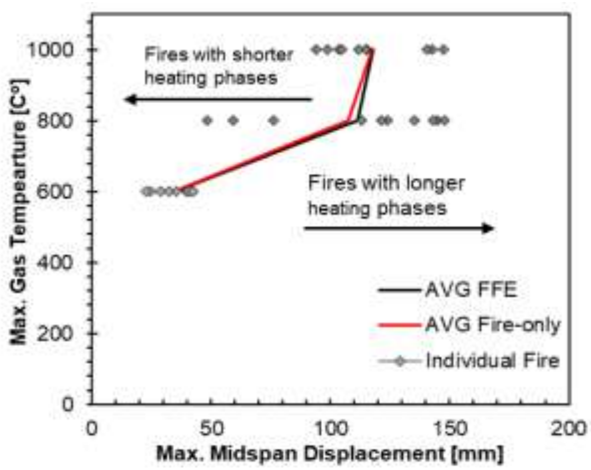
The difference between the fire-only and FFE analysis results could be a result of the excess gravity capacity within the MRF frame. During fire loading, the MRF only resists gravity load demands (Equation 1), and restraint forces due to thermal expansion. Specifically, the MRF beam (W18x40) has a flexural demand-to-capacity at the midspan of the beam of 0.25 using Equation 1 to calculate demands. This surplus in flexural capacity resists the large bending moments due to curvature from the thermal gradient throughout the beam depth.



(a)



(b)



(c)

Figure 3.12 Maximum midspan displacement of compartment one LFR beam at scaled maximum temperatures exposed to (a) 0.2g PGA, (b) 0.59g PGA, and (c) 0.98g PGA ground motions

Table 3.2. Summary of average maximum midspan beam deflections during FFE and fire-only analysis

Scaled Temp. [^{o}C]	Fire Following Earthquake (FFE) [mm]			Fire-only [mm]	Percent difference of FFE from fire-only analysis		
	0.2g	0.58g	0.98g		0.2g	0.58g	0.98g
600	34.5	42.3	34.6	33.5	2.9%	-4.6%	5.1%
800	110.6	111.4	107.4	107.4	3.0%	-3.7%	0.2%
1000	115.6	117.9	117.4	117.4	-1.5%	-0.4%	2.8%

In both midspan beam deflections and inter-story drift, the divergence between FFE and fire-only was greatest in the 600^{o}C scaled suite of fires. The suite of fires scaled to a maximum temperature of 1000^{o}C had the greatest similarities between FFE and fire-only scenarios. This might have been the higher gas temperatures that cause steel member temperatures to reach highly-elevated temperatures quickly, leading to runaway midspan displacements indicating failure of the structure.

3.6 Summary and conclusions

A comprehensive coupled IFA and IDA analysis was performed on a three-story MRF to quantify the FFE behavior of the steel structure. The FFE analysis was compared with a fire-only analysis to also evaluate the differences between the two and conclude how the earthquake prior to the fire may influence the behavior of the structure. Three different ground motions were considered for the IDA and ten different fires were considered for the IFA. Inter-story drift was used at the EDP for the IDA and beam midspan displacement was used as the EDP for the IFA.

The results of the IFA and IDA FFE and fire-only comparisons shows that the behavior of the MRF was negligibly affected by previous ground motions. This conclusion is applicable to both the global behavior of the building (inter-story drift) and the local behavior of the building

(MRF beam midspan displacement). These conclusions are also consistent with previous findings (Chicchi 2017; Khorasani et al. 2015; Maddalozzo and Fischer 2020).

However, the behavior of the building was impacted by the maximum temperature of the fire after the ground motions. Inter-story drifts of the first-story increased by 20-28% when the maximum temperature of the fire increased from 600^{o}C to 1000^{o}C for the FFE analysis and by 20% for the fire-only analysis. Beam midspan displacements increased by 235-282% when the maximum temperature of the fire increased from 600^{o}C to 1000^{o}C for the FFE analysis and by 250% for the fire-only analysis. Lastly, fire duration also influenced the behavior of the structure in FFE and fire-only analyses. Fires with shorter heating durations caused smaller midspan beam displacements and inter-story drifts On average, fires with a heating phase longer than 30 minutes caused beam midspan displacement of 13, 57, and 4 mm for fires with maximum temperatures of 600^{o}C, 800^{o}C, and 1000^{o}C, respectively than fires with heating phases less than 30 minutes. These midspan displacements were larger than the overall average midspan displacements for all of the fires with heating phases less than 30 minutes by 49%, 75%, and 7% for fires with maximum temperatures of 600^{o}C, 800^{o}C, and 1000^{o}C, respectively.

CHAPTER 4: Summary, Conclusions, and Future Work

4.1 Research Summary

Large seismic events can cause damage to fuel utility lines, which can start fires within buildings. At the same time, these seismic events can cause damage to water utilities preventing active fire suppression of the fires within the buildings. During an earthquake, large plastic deformations may occur within the building, particularly the lateral force-resisting system. These plastic deformations, therefore, provide a base damage state for the fire, where the fire can further damage these structural members. The focus of this thesis was to investigate the fire following earthquake (FFE) performance of steel moment-resisting frames (MRF). To accomplish this, a series of numerical analysis studies were performed.

Fire modeling methodologies were developed in the open-sourced finite element (FE) program, OpenSees. These methodologies were developed through a series of validation studies where the results of the OpenSees analysis was compared to experimental tests results or the results of simulations of the same validation study in another FE program. These validation studies were chosen to range from simple to more complex with 2D thermal-mechanical modeling and analysis within OpenSees. The validation studies were made available to the public via the OpenSees documentation website (Fischer and Maddalozzo 2020).

The benchmarked modeling methodologies were utilized to evaluate a 2D steel MRF subjected to FFE hazards. The steel MRF was analyzed in two ways. The first was to investigate how the location of the fire hazard within the building would impact the structural response to an FFE and the second was to investigate the impact of the ground motion intensity and fire intensity on the behavior of the MRF in an FFE. In both investigations, the 2D steel MRF was modeled used *dispbeamcolumnthermal* elements and Steel01Thermal material in OpenSees. Additionally, in both investigations, the FFE behavior of the steel MRF was compared to fire-only scenarios.

The first investigation examined three different compartments, each on a different floor of the building, and two different fires within the compartment for a total of six FFE and fire-only analyses. The fires considered represented a fuel-controlled fire and ventilation-controlled fire by changing the fuel loads within the compartment. The results showed that regardless of the fire in the compartment, the FFE behavior of the compartment was similar to the fire-only behavior of the compartment. The behavior of the steel MRF was also compared across fires showing that a more intense fire (longer duration, higher fuel load) will result in more damage during the fire following an earthquake.

The second FFE investigation involved coupled Incremental Fire Analysis (IFA) and Incremental Dynamic Analysis (IDA) FFE analysis. This IFA and IDA analysis considered three scaled ground motions scaled to 1, 3, and 5 times the maximums peak ground acceleration (PGA) of the earthquake and 10 parametric fire curves that were then scaled to three temperatures, 600^{o}C, 800^{o}C, and 1000^{o}C. Only the first-floor compartment was investigated due to the results of the first FFE analysis. For the fire, the intensity measure (IM) of the IFA was maximum temperature and the engineering demand parameter (EDP) was beam midspan displacement. For the earthquake, the IM was PGA and the EDP was inter-story drift. The results of this coupled IDA-IFA FFE analysis showed that under all combinations of ground motions and fires the beam midspan displacement was the same for FFE and fire-only behavior. However, the intensity of the fire did impact the behavior of the steel MRF, particularly the beam midspan displacement and the inter-story drift at the end of the FFE.

4.2 Conclusions

Based on the investigations performed to develop and execute validation studies for OpenSees to simulate the behavior of steel structures in fire, the following conclusions are drawn:

1. OpenSees was able to simulate the displacement of structural members and calculate the internal forces in members due to a fire.

Based on the FFE analysis performed on a 2D steel MRF considering multiple compartments throughout the buildings, the following conclusions are drawn:

1. The low gravity demands on the beams of MRFs, the fire-only, and FFE behavior of the beams were similar, regardless of the level of plastic strain developed at the hinge locations during an earthquake,
2. As the magnitude of the plastic strain developed in the hinge during the seismic analyses decreased, the similarity between the FFE and fire-only analysis increased.

Based on the FFE analysis performed on a 2D steel MRF considering multiple ground motions and fire scenarios, the following conclusions are drawn:

1. Regardless of the ground motion intensity, the behavior of the steel MRF in an FFE is similar to that in a fire-only scenario,
2. The intensity of the fire demand in a compartment will influence the behavior of that compartment. An increase of 400^{o}C, from 600^{o}C to 1000^{o}C can lead to a 145% increase in midspan beam deflection.
3. On average, fires with a heating phase longer than 30 minutes caused beam midspan displacements of 13, 57, and 4 mm larger for fires with maximum temperatures of 600^{o}C, 800^{o}C, and 1000^{o}C, respectively than fires with heating phases less than 30 minutes.

4.3 Suggestions for Future Work

The research presented in this thesis is just one step in a continuation of research that will hopefully yield more developments and understanding within FFE research and modeling. Future work on FFE research using OpenSees framework should include simulating the behavior of the building with three-dimensional models. In order to accomplish this, xzsed and columns need to be modeled and implemented into the FFE models.

The ability to model damage to gravity framing systems is critical to fully model and predict the three-dimensional behavior of structural members in an FFE or fire-only. One of the findings from this study showed that MRFs often have adequate strength against thermal loadings following an earthquake. However, components of gravity framing systems, such as gravity connections, are not designed for the large axial tensile and compression forces that occur during the heating and cooling phases of thermal loading. By incorporating gravity systems within a 3D FFE model, the global and local behavior of the structure and structural components will be fully modeled.

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