Towards a Performance-Based Fire Design Framework for Composite Steel Deck Construction in Canada

by

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Abstract

In Canada, buildings are designed for fire safety in a predominately prescriptive manner, especially the structural design of those buildings for the fire limit state. This is done on the basis of fire-resistance ratings which are determined from standardized testing. The research presented herein first assessed the Canadian literature to determine if performance-based fire design (PBFD) could be implemented nationally and then analyzed what precedents existed in Canada. There was found to be a clear trend towards PBFD but a large competency and knowledge gap exists relative to international practice.

Next, benchmark modelling was performed to transparently demonstrate the competency that is needed to assess a structure for a real fire. This benchmark modelling started with simply supported steel beams and progressed to modelling a portion of a building that was subjected to full-scale, full-floor fire tests. Building on this, a novel alternative solution was proposed which incorporated PBFD to achieve quantifiable benefits in robustness, economy, and resilience. By modelling a whole floor of a structure it was possible to quantify connection forces and behaviour, demonstrate mechanisms in the response of the composite steel deck, and identify knowledge gaps including partial composite action and differences between materials tested and materials used in contemporary construction. These research needs are just as applicable internationally as they are in Canada.

Finally, a framework is proposed for progressing PBFD responsibly in Canada. This research contributes a novel and complex alternative solution that is sorely lacking in the Canadian literature if we are to progress our treatment of the fire limit state and provides next steps for developing and implementing more structural PBFD solutions.

Co-Authorship

The work contained herein has been influenced by input from my co-authors through the production of three articles. This thesis is written in a traditional format, with the first two research papers serving as input for various chapters prior to finalization of the thesis. In both cases the author was the lead author of those publications. The third research paper was led by a current graduate student who interned at the authors firm during the summer of 2016. In that case, the author was second author and the results of this thesis informed the research that was conducted and that research paper has since been submitted.

Portions of Chapter 3 have been modified from:

- Smith, M, Gales, J, Masoud, S, Mostafaei, H (2015). Structural Fire Design for Composite Steel Deck Construction in Canada, *Proceedings of the Firth International Workshop on Performance Protection & Strengthening of Structures under Extreme Loading*, East Lansing, 821-828.

Portions of Chapters 3, 4, and 7 have made use of the tools developed for and referenced:

- Smith, M, Gales, J (2016). Integrating Fire as a Load Case with BIM. *Advantage Steel, 56.*

The results of this thesis helped form the basis for developing the manuscript:

- Quiquero, H, Smith, M, Gales, J, (2016). Developing Fire Safety Engineering as a Practice in Canada. *NRC Research Press, 16pp* (Submission Number CJCE-2016-552)

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The past two years have been an incredible opportunity to learn about fire safety engineering and understand how it can make Canadian structures safer. I would first like to thank my supervisor Dr. John Gales for introducing me to this exciting field, being an incredible mentor, and making this research possible. It would not have happened without his encouragement, guidance, and expertise. I would also like to thank the other members of his fire safety group who have been great colleagues during my time at Carleton, specifically Hailey Quiquero and Lauren Folk.

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I was inspired to learn about fire safety having grown up an incredibly proud son of a firefighter, and I am so grateful to be able to tackle the fire problem from an engineering perspective. This thesis will have lots of equations, graphs, and technical jargon, but when fires occur it is the first responders who risk their lives to keep us safe. We should all be especially thankful for the service these men and women provide.

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List of Acronyms

ACNBC - Associate Committee on the National Building Code

ACNFC – Associate Committee on the National Fire Code

AHJ – Authority Having Jurisdiction

AISC – American Institute of Steel Construction

BJFRO - British Joint Fire Research Organisation

CFD – Computational Fluid Dynamics

CISC – Canadian Institute of Steel Construction

CSA – Canadian Standards Association

DBR – Division of Building Research

FDS – Fire Dynamics Simulator

FEM – Finite Element Method

FRS – Fire Research Station

iTFM – Improved Travelling Fire Methodology

NBC(C) – National Building Code (Canada)

NFC – National Fire Code

NRC – National Research Council (Canada)

PBFD – Performance-Based Fire Design

SFPE – Society of Fire Protection Engineers

TFM – Travelling Fire Methodology

TMA – Tensile Membrane Action

1 Chapter: Introduction

1.1 Current Structural Fire Design in Canada

In Canada, structures are currently designed for fire by following the prescriptive guidelines of the building code. These guidelines generally prescribe that the structure shall be a fire-resistance rated assembly with a certain fire-resistance rating, depending on the building properties and occupancy type. The fire-resistance rating is a measure of how long it took that assembly to meet a range of failure criteria in standard fire testing. In this test, assemblies are subjected to a fire that increases rapidly in temperature in the first ten minutes, then increases at a mostly linear but decreased rate for the remainder of the test. This fire curve, the "standard fire", is shown in Figure 1.1.

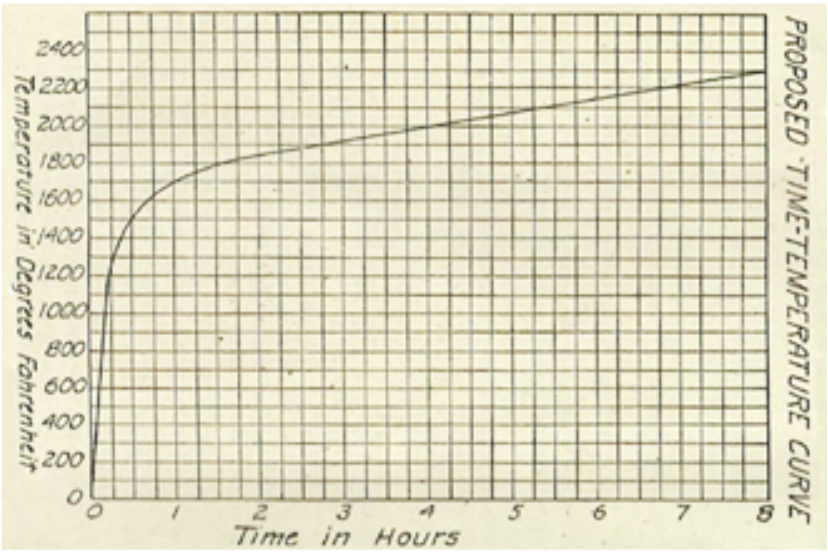

Figure 1.1: Standard fire curve (UL, 1916)

The standard fire dates back to the late 1800's, when the first North American fire test standard was disseminated as part of the New York Building Code in 1899 (Babrauskas & Williamson, 1978). The standard fire curve as well as the acceptance criteria for a fire-resistance test were codified in the first edition of the ASTM-E119 (ASTM, 2016) test standard in 1918, which at the time was designated as ASTM C19. It

has not fundamentally changed in nature since its first introduction. However, this standard fire does not consider the compartment geometry, ventilation, or fuel load. These are all parameters research has proven impact the fire. As well, this prescriptive means of designing structures for fire does not determine thermal expansion forces in the surrounding structure or interaction of the structural elements; both are aspects that research has again shown impact the performance of a structure in fire (Flint et al., 2013). The Canadian building code is objective-based meaning every prescriptive clause is tied to objective and functional statements that describe the goals of the code. It has a section on "alternative solutions" which allows certain elements to be substituted for others on the basis of equivalency (MMAH, 2012). This equivalency is done with regards to the objective and functional statements of that approved solution. In Canada, it can be seen that there is a push towards performance-based solutions, solutions which are now commonplace in other parts of the world, and this alternative solutions clause is regulatory proof of that trend.

1.2 International Performance-Based Design

Internationally, there is a trend towards designing structures for real fire scenarios. This is made possible by the extensive research that was conducted by early pioneers of fire safety research that were able to determine how a real fire may behave in a compartment and what parameters impact this. It was also enabled by full-scale testing and accidental fire observations that gave insight into the mechanisms that structures undergo during fires. In particular, the Cardington Test series which was funded by British Steel where a full-scale composite steel building was exposed to real fire scenarios with varying levels of fire protection applied.

These international designs are done by quantifying a range of realistic fire scenarios and treating this as a fire load case to design the structures for. Examples that will be discussed include Mincing Lane (Lamont et al., 2006), Building EW 11 (Block et al., 2015), and Wates House (Kho et al., 2015). The benefits of doing so are architectural benefits (exposed structural steel), economic benefits, and reduced risk by quantifying the performance during fire. It is further made possible by educating the Authorities Having Jurisdiction (AHJ's) in these regions so that they understand the designs being presented to them and have confidence in the designers and the peer review process. This last point is especially important, as early criticism of performance-based fire design (PBFD) was that it brought with it an "anything goes" mentality (Bergeron et al., 2004) that relied more on consultants than approving authorities (Buchanan, 1999) and where the political challenges often overshadowed the technical ones (Alvarez et al., 2013).

1.3 Motivation

The motivation for this research is to assess the current state of structural fire design in Canada and understand the trajectory for where it is headed. The literature review will show there is a dearth of published Canadian case studies that have transparency and detail on par with our European peers. The majority of structural fire case studies appear in a magazine published by the steel industry since the economic benefits of PBFD have been well demonstrated internationally. However, it is dangerous to let PBFD proliferate for the sole purpose of saving money without having the appropriate checks and balances in place. The larger motivation for this research is to determine what this framework looks like to ensure competency in structural fire design and to ensure that public safety is never compromised in the name of reducing fire

protection. There is potentially a very large, untapped market in Canada for providing PBFD solutions if we can enable the profession to grow with confidence and buy-in from all the stakeholders.

1.4 Research Focus and Scope

1.4.1 Potential for Performance-Based Design in Canada

The research for this thesis focuses around the existing regulations in Canada that performance-based design will inherently need to be mindful of and take into account during its implementation. This research will include the development of the building code itself, precedents from international practice, precedents from Canadian case studies, and evaluation of the Canadian and International education systems that support the knowledge foundation of fire safety practitioners and AHJ's.

1.4.2 Benchmark Modelling, Verification, and Case Study

Once the potential for PBFD is understood, the research will shift to how competency is developed outside of the education system if that education system is found to not provide all the necessary skills. This will include research on the tools available to model structures in fire, as well as available experimental data that can be used to validate and verify computer models and the user's competency. Afterwards, research will shift towards acceptance criteria that can potentially be used within a performance-based design, and also what metrics exist to compare a performance-based design to a prescriptive design so that stakeholders can quantify the benefits and determine if the approach is applicable for their specific case.

1.5 Research Objectives

The objectives of the research are as follows:

1) Evaluate the potential for PBFD in Canada by considering the international state-of-the-art, and the Canadian regulations governing the current fire protection strategies;
2) Develop an alternative solution for a whole floor plate that takes into account the state-of-the-art in structural fire design, and quantify the benefits/challenges of this design as it relates to contemporary Canadian construction. To date, a Canadian case study of this detail and complexity has not been published but must become the norm for the PBFD profession to grow;
3) Provide insights for the international community based on this case study which utilizes the improved travelling fire methodology (iTFM) in an actual Canadian building as compared to theoretical building layouts in the current literature

Once the potential for PBFD is understood from objective (1) and demonstrated from objective (2), the last objective of this research is:

4) Build a framework for moving the PBFD profession forward transparently and responsibly in Canada. This framework must consider and address all stakeholders from consultants, to authorities, to the education system itself.

1.6 Thesis Outline

The first chapter of this thesis introduces the reader to the topic, sets out the research areas, and provides objectives for the research. From there, Chapter 2 is a further introductory chapter that summarizes the key concepts in fire safety engineering. It is provided for the reader to understand the key concepts and terminology which impact the structural behaviour in fire, such as the fuel load, expected compartment temperatures, heat transfer, material behaviour, and simplified structural analysis methods.

Chapter 3 is a literature review of PBFD nationally and internationally. It has an emphasis on structural case-studies as opposed to smoke management or human behaviour. The literature review begins by looking at how current fire safety engineering came to be in Canada. This includes the events that triggered inertia and relations with international thrusts. A review of key Canadian and international structural fire design case studies is provided. A particularly novel aspect is the Canadian structural fire case study analysis, as to date there has not been a timeline created that shows the implementation of structural solutions in Canada. This is needed to identify gaps in current practice and opportunities for growth and improvement. The last aspect of the literature review is how Canadian literature addresses design fires and material behaviour, which has many similarities to the international.

Chapter 4, Benchmark Modelling and Verification, develops the competency necessary to model structures in fire. It does so by modeling the experimental configurations of several published tests and demonstrating the results match the experimental output. These experiments gradually and purposely increase in complexity from a simply supported steel beam up to the Cardington test. This transparent progression of knowledge is novel in the context of Canadian structural fire design, and will likely inform a framework to support PBFD. It will demonstrate to Canadian practitioners and authorities alike what methods are available to validate models and their users, and how important it is to continue publishing transparent case studies and experimental test data. The software used for this analysis is well documented and validated in the literature, meaning the emphasis of this chapter is validating that the user is able to use this software to competently predict the performance of structures in fire.

Building on the literature review, discussion of fire safety and structural principles, and the benchmark modeling, Chapter 5 is a proposed alternative solution for a Canadian building that has been designed structurally for the fire load case using PBFD. This complex case study is itself novel in a Canadian context since published case studies are generally simplified and lack sufficient detail, but it also allows the benefits of performance-based design to be quantified. This is important for Canadian practitioners and other stakeholders to be aware of. The benefits demonstrated and discussed are increased robustness of the structure, economic savings, and the ability of the building to enable a resilient operation within it. This is the first published case study that uses a Canadian specific structural design with the available Canadian fire design literature and Canadian construction prices to demonstrate the efficacy of PBFD on a national level. Further novel findings of the case study include application of the proposed ASCE Fire Protection Committee acceptance criteria, demonstration of internal mechanisms that have been demonstrated in experimental tests of composite beams in fire, and the application of structural fire performance to a much broader operational resilience discussion. This last item is particularly novel on the international level and identified a research need which has already materialized as a multi-year NSERC Collaborative Research and Development peer-reviewed grant between Carleton University, CISC, and Entuitive.

The last chapter is a proposed framework for performance-based structural fire design. One opportunity from Canada being behind other countries is that we can incorporate their lessons learned as this profession grows from infancy. It is crucial to have a framework set out so that performance-based designs done in practice have the

necessary competency and peer review in place so that all stakeholders can have confidence in the designs and embrace this new direction of assessing the fire limit state. As we begin to deviate from the code, it's important that the public safety remain a priority in our designs. Any missteps as PBFD sees more implementation has the potential to bring the methods into question and keep Canada locked into current prescriptive designs. Proposing a framework and beginning that discussion in the Canadian fire protection industry is a crucial step to ensuring that structural fire design develops responsibly and that we are providing structural solutions that safeguard public interests while exceeding all stakeholder criteria. This is the first framework that informs the future of performance-based fire design in Canada while holistically considering the support mechanisms and stakeholders involved.

2 Chapter: Basics of Fire Safety Engineering

This chapter will explore the basic principles underlying fire safety engineering. This includes an overview of what fuels are and how they burn, what types of fires may exist in a structure, how these fires transfer their heat to the structure, and how to assess that structure once the temperatures are understood. The goal of this chapter is to introduce the stakeholders of a structural fire design, reader included, to the principles underlying the engineering analysis of that building for the fire load case. Unfortunately, these principles do not necessarily have to be well understood or accounted for in the current prescriptive approach, although most practitioners are likely to have a baseline understanding to ensure their design meets the unique needs of their client.

2.1 Fuels

The term fuel refers to anything that is burning. For most fires affecting buildings and their users, fuels will be carbon based. Much of the testing that has occurred on fuels and their energy content has been done using wood cribs, so typically fuel loads are expressed in an amount of wood per square meter. This can then be expressed as an amount of mega joules per square meter (MJ/m^2). Common fuel loads for occupancies are available (Buchanan, 2002), however they can also be determined experimentally by surveying the material quantities in an apartment and the available energy in these materials (Zalok et al., 2009).

2.2 Combustion Process

Combustion is an exothermic reaction in which a fuel source combines with oxygen and releases energy in the form of heat and light, and possibly other combustion products. In its most general form, the combustion reaction can be written as:

$$C_uH_vO_wN_xS_y + \left(u + \frac{v}{4} - \frac{w}{2} + y\right)\left(O_2 + \frac{X_{N_2}^o}{X_{O_2}^o}N_2\right)$$

$$\rightarrow uCO_2 + \frac{v}{2}H_2O + ySO_2 + \left(\frac{X_{N_2}^o}{X_{O_2}^o}\left(u + \frac{v}{4} - \frac{w}{2} + y\right) + \frac{x}{2}\right)N_2 \tag{2.1}$$

The above equation is said to be *stoichiometric*, meaning there are no excess reactants in the product stream of the reaction. In reality, combustion will rarely be stoichiometric and there will often be too much fuel or too much oxygen. For this reason, a fuel equivalence ratio is used to describe the actual ratio of fuel to air.

$$\phi = \frac{(n_f/n_{air})_{actual}}{(n_f/n_{air})_{shoich}} \tag{2.2}$$

where n_f/n_{air} is the ratio of moles of fuel to moles of air. When $\phi < 1.0$, the combustion is fuel lean and air rich. There will be excess air in the products of the reaction. When $\phi > 1.0$, the combustion is fuel rich and air lean. In this scenario, there is incomplete combustion and fuel will appear in the products of the combustion.

Combustion can also be both smouldering and flaming. Smouldering combustion is much slower and occurs at the surface of a fuel source, while flaming combustion is what one typically associates with a building fire and involves the combustion of the fuel source in a gas phase. The fuel source undergoes pyrolysis, and it is the gaseous form of the fuel source which combusts.

When performing a performance-based fire design (PBFD) on a structure, it is important to understand the particular hazards associated with the fuels within the building as this will impact the design decisions that are made. In particular, the hazard of a fuel source is impacted by:

- Material properties and configurations
- Ease of ignition and flame growth rate

- Potential for flame spread
- Combustion products created

The Heat Release Rate (HRR) of a fuel is an important parameter in fire safety engineering which describes the rate at which combustion reactions produce heat in units of kilowatts (kW). It essentially describes how large a fire is. A HRR curve for a fuel source can be determined analytically by assuming a burning rate of the fuel (i.e. slow, medium, or fast), as well as a maximum HRR. An example of this is a t^2 fire. For rooms with multiple fuel sources, these curves can also be superimposed with assumptions regarding ignition times to arrive at a composite HRR curve. For more complex fuel sources, a HRR can be found experimentally by burning the fuel source in an open-burning HRR calorimeter or in a room fire test and measuring the release rate as a function of time. Experimental HRR has been published for many common objects that a designer can use if their specific fuel source matches that which was tested, an example of which is shown in Figure 2.1:

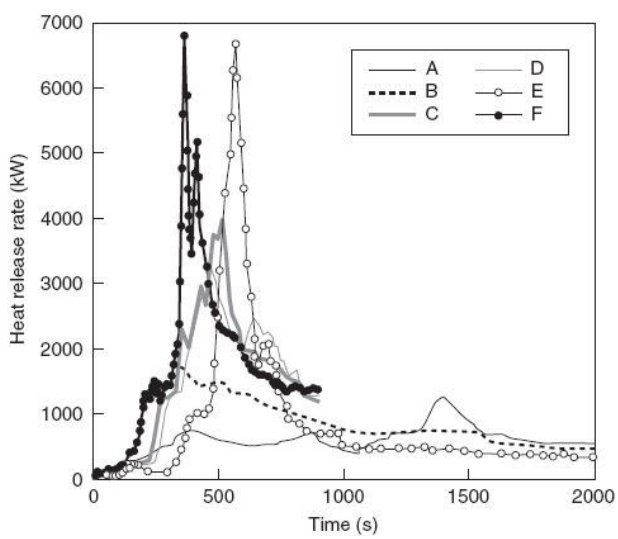

Figure 2.1: HRR for an office workstation; reproduced from Babrauskas (2016)

Once the hazards of a fuel are understood and the amount of energy that can potentially be released from combustion is quantified, the energy transfer to the surrounding structure can be analyzed as well as the interaction between the structural compartment and the fire occurring within it.

2.3 Standard Fire

The standard fire dates back to the late 1800's, when the first North American fire test standard was disseminated as part of the New York Building Code in 1899 (Babrauskas & Williamson, 1980). Prior to 1903, fire testing throughout the rest of the United States was inconsistent with each lab specifying its own time-temperature curves that just needed to meet an average minimum temperature. From 1903 to 1917, attempts were made to standardize the fire resistance testing that was occurring and this generally meant adopting a version of the New York Building Code provisions (Babrauskas & Williamson, 1980). The standard fire curve as well as a refined acceptance criteria for a fire resistance test were codified in the first edition of the ASTM-E119 (ASTM, 2016) test standard in 1918, which at the time was designated as ASTM C19 (Babrauskas & Williamson, 1980). The most novel aspect of this new standard was the standardization of a fire curve for testing, shown below in Figure 2.2, which was first published just before C19 in 1917.

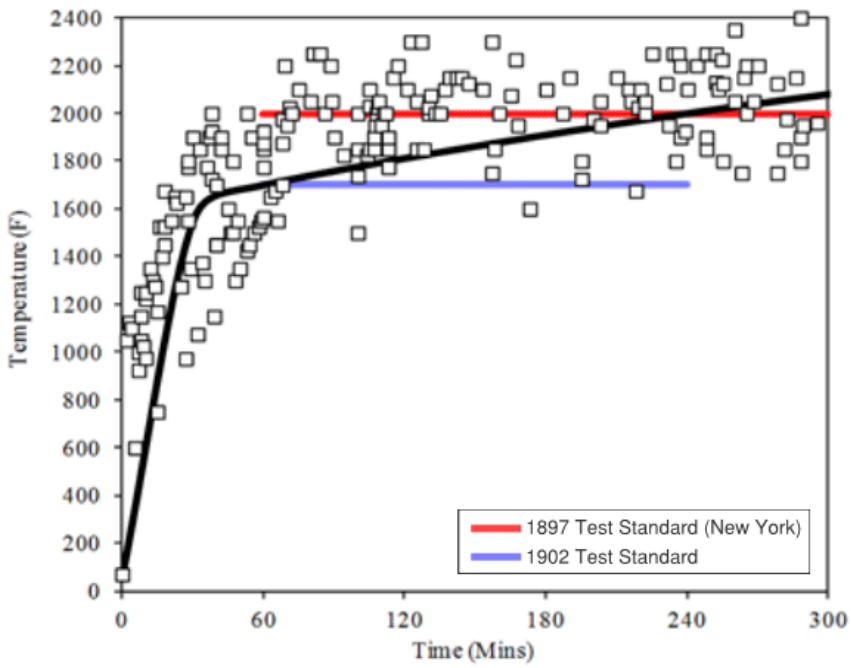

Figure 2.2: ASTM standard fire curve relative to legacy tests; adapted from Gales et al. (2012) and The Engineering Record (1897)

The standard fire does not consider the compartment geometry, ventilation, or fuel load. These are all parameters research and science has proven impact the fire (Drysdale, 2011), but the standard fire was prescribed before estimates of temperatures in real building fires could be estimated much less quantified (in North America at least) (Babrauskas & Williamson, 1980). This prescriptive means of designing structures for fire does not determine thermal expansion forces in the surrounding structure or interaction of the structural elements, both are aspects that research has again shown impact the performance of a structure in fire (Flint et al., 2013).

The standard fire is still used in current practice as a means to test fire rated assemblies and prescribe fire-resistance ratings to them. The curve itself has not changed substantially since 1917. Standard fire testing still has a role in contemporary fire design as a means of comparing various assemblies against one another and comparing these with historically accepted levels of performance for that standard test. However, standard

fire testing does not quantify the performance of a structure for fire or seek to ensure that structure is safe for all expected fire scenarios. On the development of a standardized fire test:

> *"The object of all tests of building materials should be to determine facts and develop results that may be of practical value in future designing. In order that such facts and results may have real value, three conditions are necessary: first, that the materials tested shall be identical with what is commercially available in the open market; second, that the conditions, methods, and details of constructions conform exactly to those obtainable in practice; third, that the tests be conducted in a scientific manner"* (Himmelwright, 1898)

In the 119 years after the above quote, we have still not satisfied all three criteria. Standard fire testing does not represent the spans and heights typical in contemporary practice. The fire curve used does not represent reality and in fact ignores a cooling phase which is known to be onerous on structural connections. We extrapolate these assemblies to entire buildings without considering the actual response to a fire. The following sections will provide introductory details on how we can actually quantify real fires, the resulting temperatures, and what impact these temperatures will have on the structure. This demonstrates that the knowledge and experience exists to alternatively design our structures for a fire load case and not rely on standard fire testing of single elements to ensure our buildings stand during extreme events.

2.4 Design Fires

Contrary to the standard fire discussed in Section 2.3, the term "design fire" refers to a realistic fire scenario that takes into account various parameters of the compartment.

The standard fire is not referred to as a design fire because the intent of the fire curve was purely to compare various assemblies and not to reflect a real fire within a building.

Depending on the type of design fire being developed, the parameters considered for a design fire may vary. Typical design fires used in practice today include a localized fire, an interior compartment fire which may or may not reach flashover, an exterior fire, and a travelling fire. Flashover is a critical stage in a compartment fire where the temperatures become so severe that all flammable material ignites; it poses a severe risk to firefighters and is structurally significant. Compartment fires are functions of the available fuel load, the properties of that fuel, the thermal properties of the compartment, and the geometry of the compartment including ventilation. A travelling fire is a design fire assumed to occur in a very large compartment that does not reach flashover. For that reason, the thermal properties of the compartment and the ventilation are not parameters. Instead, the fuel load and compartment geometry are the main parameters of the time-temperature curves that various parts of the structure are exposed to. A travelling fire is inherently non-homogenous throughout the compartment. As well, realistic design fires also include a cooling phase which is an important phase of the fire when contraction of the different elements imposes forces that vary in nature from those developed during heating. The standard fire does not account for cooling. An example of a time temperature curve with a cooling phase, the Eurocode parametric fire, is shown in Figure 2.3.

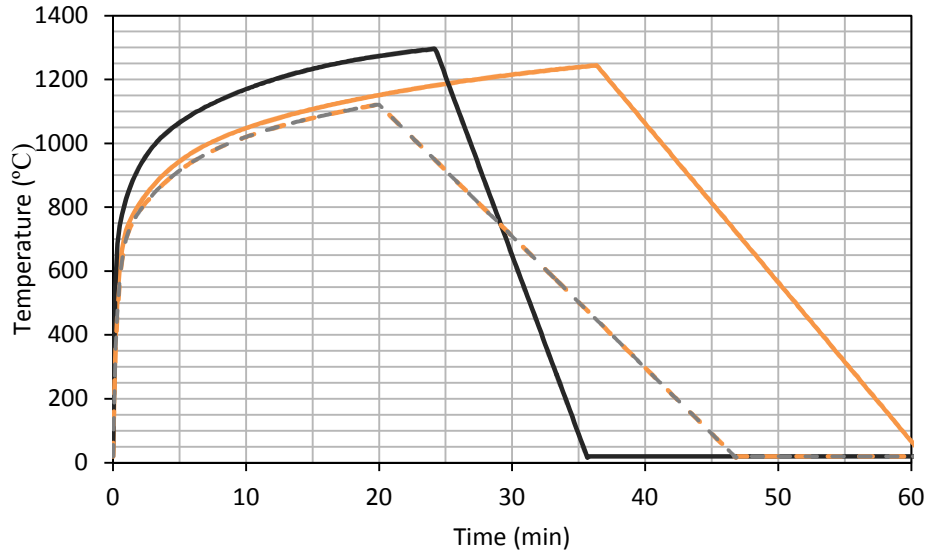

Figure 2.3: Sample eurocode parametric design fires for various ventilation conditions

A more thorough description of each of the types of design fires can be found in Sections 3.7.4.1 to 3.7.4.4, including how the available Canadian design guidelines address each design fire. In most cases, the Canadian practitioner is referred to international literature for the calculation of the design fires.

2.5 Heat Transfer

With the fuel load determined and its HRR, as well as the definition of the design fire itself, the temperatures of the structure can then be assessed. The three mechanisms for heat transfer are conduction, radiation, and convection. For structural applications, the predominant mechanisms are radiation (for heat transfer to the structure) and conduction (for heat transfer within the structural elements).

2.5.1 Conduction

Conduction is the heat transfer through and within solids. Heat will flow from a region of high temperature to one of low temperature, with this flow being expressed as a heat flux. In one direction, this heat flux is given as:

$$\dot{q}_x'' = -\kappa \frac{\Delta T}{\Delta x} \tag{2.3}$$

where

$\dot{q}_x''$ = conductive heat flux, W/m^2

κ = thermal conductivity, W/mK

ΔT = temperature difference, K

Δx = distance over which heat flux is measured, m

From a fire hazard perspective, conduction is important for problems relating to ignition and flame spread. In structural applications conduction is necessary to determine the temperature distribution within structural elements and the rate of heat transfer through vertical or horizontal assemblies.

2.5.2 Radiation

Radiation is the transfer of energy by electromagnetic waves. According to the Stefan-Boltzmann equation, the total energy emitted is proportional to T^4 and can be expressed as:

$$E = \varepsilon \sigma T^4 \tag{2.4}$$

where

E = emissive power, W/m^2

ε = emissivity

σ = Stefan-Boltzmann constant, 5.67x10^{-8} W/m^{2}K^4

T = temperature, K

In a typical building fire, most of the radiation is emitted by solid particles of soot. Radiation is critically important in structural fire applications as it dominates the energy transfer to, and subsequent heating of structural elements.

2.5.3 Convection

Convection is the method of heat transfer associated with movement of fluids, in particular gas or liquids adjacent to a solid. The empirical relationship for convection is given as:

$$\dot{q}'' = h\Delta T \tag{2.5}$$

where

$\dot{q}''$ = convective heat flux, W/m^2

h = convective heat transfer coefficient W/m^{2}K

ΔT = temperature difference, K

The convective heat transfer coefficient is a property of the specific system, not of the materials. Typical values can range from 5-25 W/m^{2}K for free convection and 10-500 W/m^{2}K for forced convection in air (Drysdale, 2011). Convection is particularly important in the early stages of the fire when temperatures are too low to make radiation dominant (since radiation is a function of T^4).

2.6 Fire Resistance

Fire resistance refers to the ability of an element of a building, called an assembly, to resist the effects of a fire. The term fire resistance is generally associated with the prescriptive approach to fire protection and is usually expressed as a period of time that an element can withstand exposure to a standard fire (Section 2.3) until it meets failure criteria defined in a fire resistance standard that has been adopted by that particular municipality. This period of time is referred to as a fire-resistance rating. As an example, most assemblies in Canada with a fire resistance rating have been tested according to CAN/ULC-S101 which outlines the test procedure and the failure criteria (ULC, 2014).

This test standard is very similar to the ASTM E119 test standard (ASTM, 2016). The failure criteria are expressed in terms of stability, insulation, and integrity. Stability refers to the ability for the structure to support load without structural collapse during the fire, integrity refers to the ability for the assembly to prevent the passage of hot gases or smoke, and insulation refers to the temperature on the unexposed side not being high enough to cause injury or ignition in adjacent compartments (ULC, 2014). Not all three failure criteria are necessarily required for each assembly. For example, a column only needs to satisfy stability criteria as it is not intended to provide compartmentalization. Likewise, a door or partition is critical for compartmentalization and must satisfy the integrity and insulation criteria, while there is no stability requirement.

Most assemblies used in Canada have been tested to the CAN/ULC-S101 (ULC, 2014) for a fire-resistance rating, however not all assemblies have. Some typical assemblies have been assigned a generic rating that can be found in the NBCC. These are either simple assemblies, such as a concrete slab of known thickness and rebar cover, or a composite assembly that has fire resistance ratings added up for the multiple layers using the component additive method of the NBCC. There is also the opportunity to use calculation methods to achieve a fire resistance rating. These are currently quite limited in the Canadian building code. Some examples include the fire-resistance rating of mass timber which has an equation provided by the building coded or the fire-resistance rating of a steel tube column filled with concrete. More advanced calculation methods such as the capacity of a structural steel beam are currently addressed in an introductory manner by Annex K of CAN/CSA-S16-09, but these generally fall into the alternative solution category of the building code (CSA, 2009).

2.7 Material Behaviour

Composite construction, which is the building type focused on herein, uses both steel and concrete structurally to be efficient with material use. By connecting the concrete floor to the steel supporting beams, the concrete is engaged to resist the load as long as an adequate longitudinal shear transfer exists between the steel beam and the concrete slab. The flexural moment that must be resisted by the composite floor section then puts the steel into tension and the concrete into compression; stress states that both materials are ideal at resisting. For this reason however, it is important to understand how both the steel and concrete elements of the structure will perform when exposed to fire. This includes the mechanical response and the thermal response.

2.7.1 Structural Steel

The discussion on structural steel will be limited to normal strength, carbon steels that are typically used in building construction. Available Canadian design guidelines, in the form of Annex K in CAN/CSA-S16-09 (CSA, 2009), tend to adopt Eurocode material properties for steel. The Eurocode does not make specific reference to what type of steel it is using, just that it is for carbon steel and shall be considered characteristic values unless known otherwise. The Eurocode values were assumed to be applicable to Grade 350W steel used in Canadian construction for hot rolled wide-flange shapes.

In general, the mechanical properties of steel decrease with elevated temperature. The main parameters to consider in design are typically the elastic modulus, the yield strength, and the proportional limit (the stress level to which the stress-strain relationship is linear). The reduction of these properties with temperature is shown in Figure 2.4.

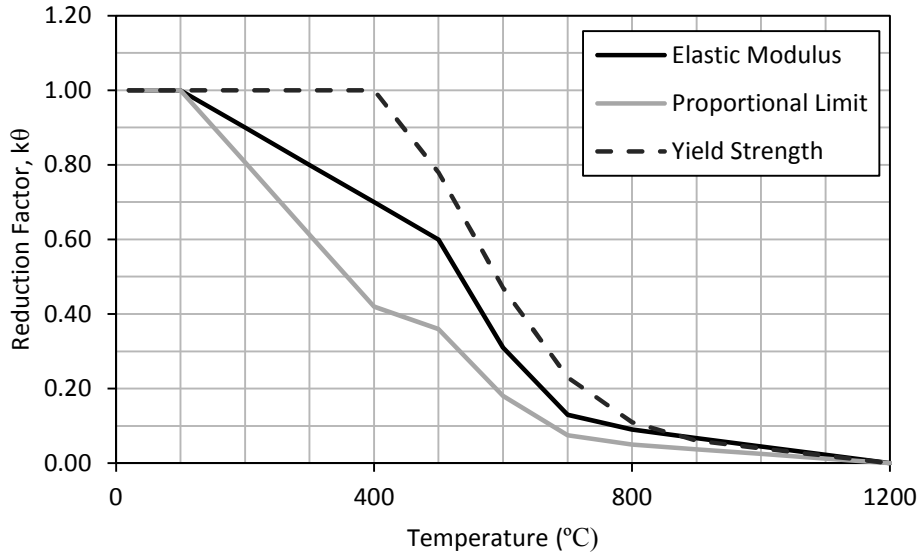

Figure 2.4: Stress-strain reduction factors for carbon steel at elevated temperature (CEN, 1993-2005)

The mechanical stress-related strain outlined above is just one component of strain that can affect structural steel. A more general form of strain can be written as:

$$\Delta\varepsilon = \varepsilon_{th}(T) + \varepsilon_{\sigma}(\sigma, T) + \varepsilon_{cr}(\sigma, T, t) \tag{2.6}$$

In the above equation, the mechanical stress-related strain is ε_{σ} and is a function of both the applied stress and the temperature. As well, strain caused by thermal expansion is denoted ε_{th}. The thermal expansion strain, $\varepsilon_{th,}$ is equal to (CSA, 2009):

$$\frac{\Delta L}{L} = 14x10^{6}(T - 20) \tag{2.7}$$

In the above, ΔL/L represents the strain and T is the elevated temperature. In the Eurocode, the relative thermal elongation is itself dependent on temperature (CEN, 1993-2005). The last component of strain to be considered is creep strain. In ambient design, creep strain of structural steel is typically negligible (Buchanan, 2002). Creep can have a significant impact on the predicted behaviour of structural steel, however most computer models do not account for it explicitly because the calculations are onerous and the available data on creep at elevated temperature is limited. Instead, it is rationalized that

the mechanical properties at elevated temperature are effective properties that include the effect of creep for typical fire scenarios and that creep therefore does not need to be calculated on its own (CEN, 1993-2005).

As outlined above, the mechanical properties of steel cannot be fully characterized until the temperature of the steel itself is quantified. As will be shown later, each structural fire analysis is preceded by a thermal analysis of the structural elements. The specific heat of steel, the amount of energy required to raise the temperature of a certain mass by one degree C, varies with temperature from 425 J/kgK up to 650 J/kgK. Interestingly, there is a spike up to 5000 J/kgK at 735ºC which is the result of a metallurgical change in the carbon steel as the crystal structure transitions from a body-centred structure to a face-centred structure (Wang et al., 2013). This spike is seen in Figure 2.5. Another thermal property of steel to be considered is the thermal conductivity, which is the extent to which the material can conduct heat. This parameter is also temperature dependent and decreases from 54 W/mK at 20ºC down to 27.3 W/mK above 800ºC, as shown in Figure 2.5.

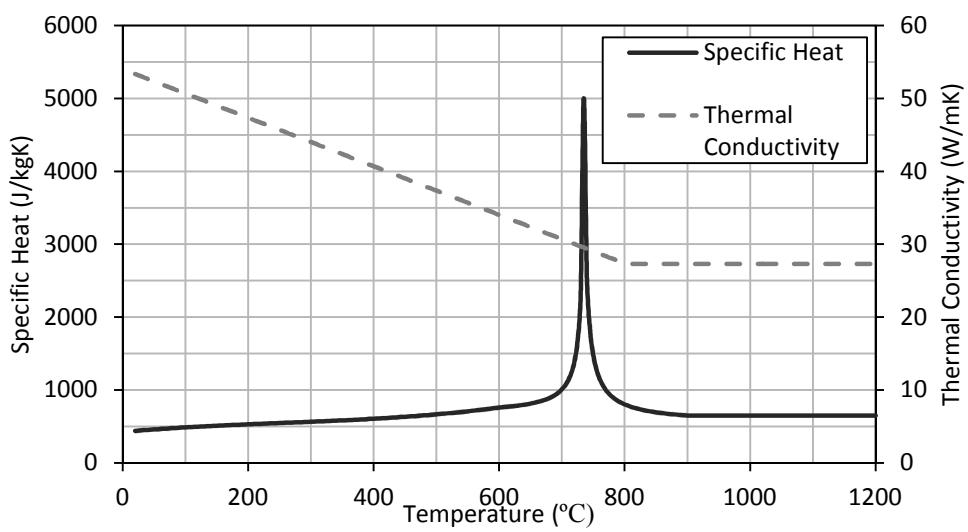

Figure 2.5: Specific heat and thermal conductivity of carbon steel as a function of temperature (CEN, 1993-2005).

When using composite construction, there are often multiple grades of steel present. The wide-flange beams are typically CAN/CSA-G40.20 Grade 350W (fy=350 MPa), while the columns may be a higher grade of steel such as ASTM A913 Grade 70 (fy=485 MPa) in high rise situations. The shear studs providing the composite action between the beam and concrete slab are typically AISI grade 1010 or 1020 and have similar yield strength to the beams. However, they are a different grade since they are cold drawn. Lastly, the deck itself is a lighter grade of steel, Grade 230, which has a much lower yield strength than the beams. Despite all of these different grades of steel, the analysis typically assumes the same thermal properties and very similar mechanical properties except for different yield strengths. These assumptions are valid at ambient temperature, however it is unknown if all these grades of steel will experience the same changes in mechanical and thermal properties at elevated temperature as assumed above. Once exact steel grades are specified for a project, the designer is encouraged to research the steel grades and see if more current information is available.

2.7.2 Concrete

As mentioned, the concrete floor slab is engaged structurally in composite construction. The mechanical properties of concrete will degrade with elevated temperature similar to steel, although both materials are “noncombustible” by definition in the NBCC. Concrete loses mechanical strength with elevated temperature due to physiochemical changes in both the cement paste and aggregate, as well as thermal incompatibility between these two (Khoury, 2000). These changes are shown in Figure 2.6, which is adapted from Khoury (2000). The response of concrete to fire is also

influenced by the fire itself, the rate of heating, the magnitude of load applied, and the moisture content of the concrete.

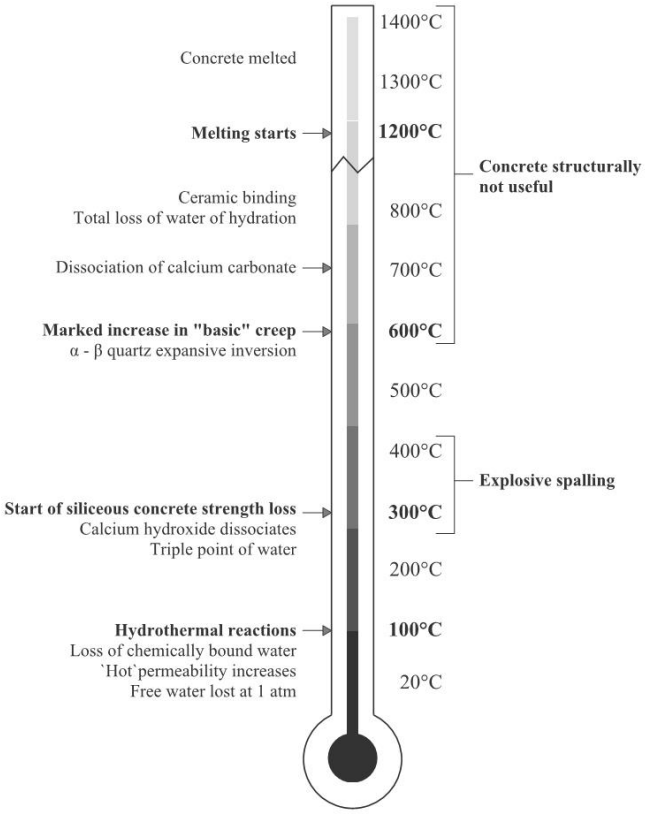

Figure 2.6: Physical and chemical changes in concrete due to temperature increase; adapted from Khoury (2000)

The Eurocode contains a series of reduction factors on the ultimate compressive strength of concrete, which decreases with increasing temperature. These reduction factors, for siliceous and calcareous concrete, are reproduced in Figure 2.7. It should be noted that these factors for concrete are reproduced in CAN/CSA-S16 Annex K, although the annex only contains siliceous concrete and not calcareous. Figure 2.7 shows that using the siliceous reduction factors for all concrete types may be conservative since it has more reduction in compressive strength for a given temperature, if the designer does not know otherwise.

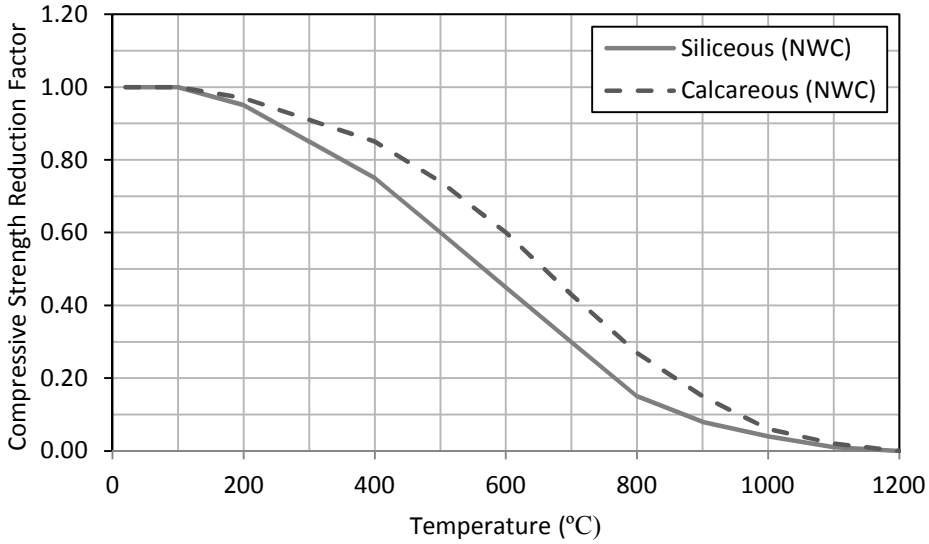

Figure 2.7: Reduction factors for siliceous and calcareous concrete

To complete the full curve of stress versus strain, the Eurocode also provides a strain at maximum stress and an ultimate strain for different temperatures. The resulting curves for different temperatures are shown in Figure 2.8 below.

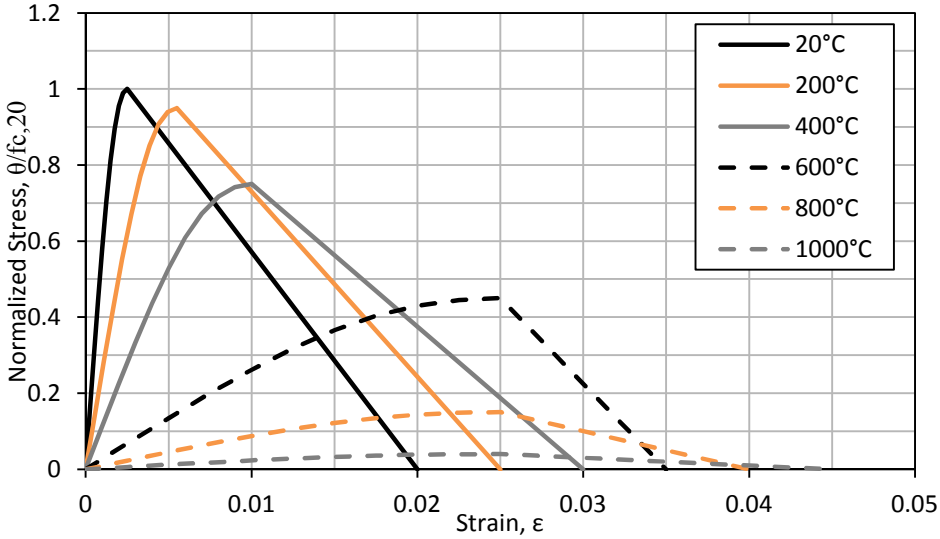

Figure 2.8: Normalized stress-strain curves for siliceous concrete at elevated temperature

2.8 Structural Analysis of Steel and Composite Structures

For steel structures, the analytical methods available for fire conditions are very similar to those used for ambient conditions, albeit with reduction factors appropriately

applied to the material properties to account for elevated temperature effects. The two material properties that affect the behaviour of steel during a building fire are the progressive degradation of the stress-strain curves of the steel with temperature increase, as well as the thermally induced strains from temperature increase (Wang et al., 2013). The former is accounted for in reducing the material properties as discussed in Section 3.7.3.1, while the latter materializes in the form of greater forces and deflections to account for in our capacity calculations. In Load and Resistance Factor Design (LRFD), the factor accounting for material and geometric uncertainty, ϕ, is typically taken as unity in fire scenarios. As well, the factored load case is different, which is given by the Canadian steel design code in Annex K (CSA, 2009) as:

$$D + T_s + (\alpha L \text{ or } 0.25S) \tag{2.8}$$

where

D = dead load

T_s = expansion, contraction, or deflection effects due to temperature

α = 1.0 for storage, equipment, or services areas. 0.5 otherwise

L = occupancy live load

S = variable load due to snow

The methods provided by the Canadian steel design code (CSA, 2009) are outlined below, adapted from Annex K of the design code. The equations provided are applicable for fire scenarios where temperatures exceed 200ºC. In discussing these methods, it is assumed the reader has some understanding of structural mechanics, although parameters and equations will be elaborated on where the methods or parameters vary from those at ambient.

2.8.1 Steel Tension Members

The resistance of steel in pure tension is a relatively straightforward calculation. The strength of the member is controlled by the cross-sectional area and the yield strength of the steel.

$$T_r(T) = AF_y(T) \tag{2.9}$$

where

$T_r(T)$ = Tensile capacity at temperature T

A = Cross-sectional area of tension member

$F_y(T)$ = Yield strength at temperature T, see section 2.7.1

2.8.2 Steel Compression Members

The compressive strength of a steel member is a function of the unbraced length of that member, the geometric properties of the cross-section, the yield-strength of the steel, and the elastic modulus. As the unbraced length increases, naturally the compressive capacity decreases as the member approaches the limit state of buckling. For short unbraced lengths and fairly stalky members, buckling will be less of an issue and the capacity will be governed more by the yielding of the steel cross-sectional area. At elevated temperatures, the capacity is calculated simply by substituting in the reduced mechanical properties of the steel:

$$C_r(T) = AF_y(T)(1+\lambda(T)^{2dn})^{-1/dn} \tag{2.10}$$

where

$\lambda(T) = \frac{kL}{r}\sqrt{\frac{Fy(T)}{\pi^2 E(T)}}$

d = 0.6

n = 1.34 for standard rolled shapes as defined in the ambient design equations

2.8.3 Steel Flexural Members

In flexural loading, the capacity of a steel member is governed by both the plastic moment capacity, that is the moment capacity when the steel cross-section has yielded and is completely tension/compression blocks, and the elastic critical load which is when the member experiences lateral torsional buckling.

The plastic moment at elevated temperature is defined as:

$$M_p(T) = Z_x F_y(T) \tag{2.11}$$

The elastic critical load at elevated temperature is defined as:

$$M_u(T) = \frac{\omega_2 \pi}{L}\sqrt{E(T)I_y G(T)J + I_y C_w \left(\frac{\pi E(T)}{L}\right)^2} \tag{2.12}$$

The moment capacity at elevated temperature T is then defined as:

$$M_r(T) = C_k M_p(T) + (1 - C_k)M_p(T)\left(1 - \left(\frac{C_k M_p(T)}{M_u(T)}\right)\right)^{C_z(T)} \tag{2.13}$$

where

$C_k = 0.12$

$C_z(T) = \frac{T + 800}{500} \leq 2.4$

2.8.4 Tensile Membrane Action

Sections 2.8.1 to 2.8.3 dealt primarily with isolated structural steel members. In composite steel buildings that have concrete floors supported on steel beams, a global

load carrying mechanism that has been demonstrated experimentally (British Steel, 1999) and analytically (Wang et al., 2013) is tensile membrane action (TMA). If the secondary beams are assumed to be ineffective during a fire scenario, the slab can still support the applied loads through a membrane action that has a tension field in the middle and a peripheral compression ring. This load carrying mechanism is only effective at very large deflections which is why it is used as an ultimate limit state in fire scenarios and not ambient design. TMA is currently used in international PBFD since robustness and economic benefits have been demonstrated. The method can be incorporated using analytical correlations that have been derived based on first principles using yield-line theory (Bailey, 2001; Bailey & Moore, 2000a; Bailey & Moore, 2000b) or using advanced finite element analysis as will be seen in the case studies of Section 3.4. This mechanism is best understood by analyzing how a composite structural steel and concrete slab behaves in a real fire event, and how the load is ultimately supported in the end condition. This process is well articulated by Wang et al. (2013) and has been adapted below.

1. Any unprotected steel beams heat rapidly and expand, although not yet losing their full capacity.
2. The concrete slab heats more slowly and begins to thermally bow towards the heat source, typically underneath the slab.
3. Continued heating of the steel beams coupled with restraint around the slab from cool adjacent structure causes high permanent compressive strains in the steel. At this stage, much of the structural steel capacity has been degraded and the response is controlled by the concrete slab.

4. At this stage it is unlikely the slab can support itself in flexure. If it is supported along the edges into relatively square bays, the slab is forced into a double curvature membrane. Radial tensile forces in the centre of the slab self-equilibrate with a continuous compression ring around the perimeter of the slab as shown in Figure 2.9.

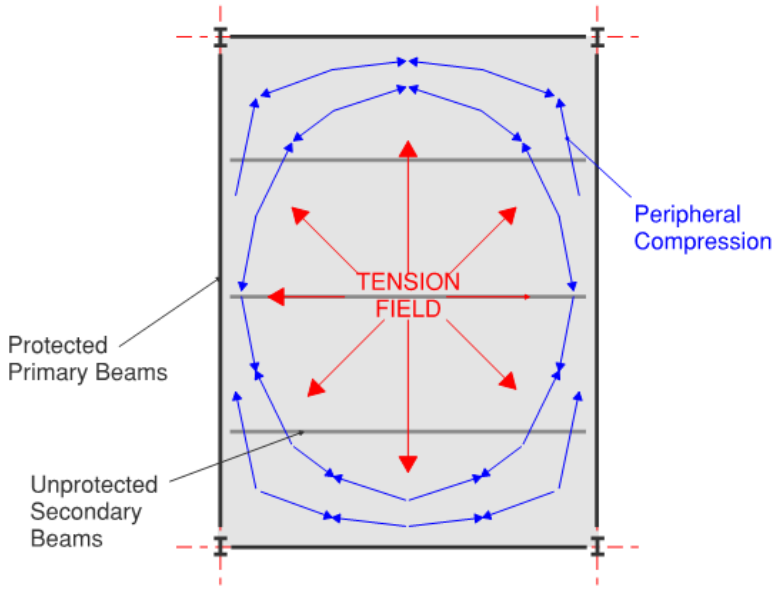

Figure 2.9: Tension membrane action in a regular slab at high deflection

5. If temperatures continue to increase and the slab continues to deflect, failure may occur if the vertical supports at the perimeter of the tensile membrane fail or if a tensile slab fracture occurs in the middle of the slab. The latter will not result in global collapse of the slab but it will cause an integrity failure since hot gas and flame can pass through the slab. If the slab is continuous, tensile cracks may also form at the slab perimeter where hogging curvature and slab continuity can cause tensile forces to develop over the vertical supports. Both of these tensile fracture possibilities are shown in Figure 2.10.

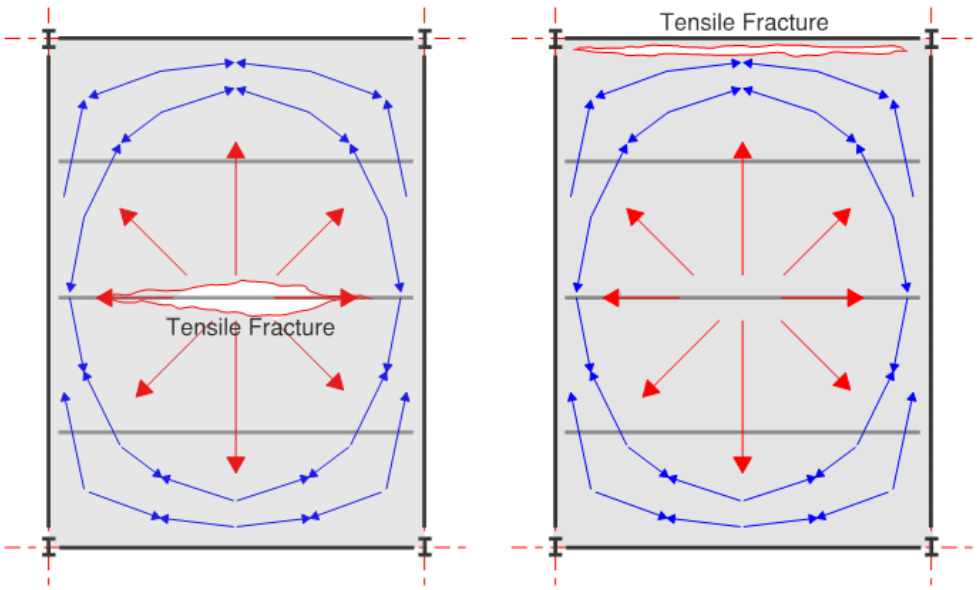

Figure 2.10: Tensile failure mechanisms associated with tensile membrane action

2.9 Summary

This chapter has provided an introduction to the basic concepts needed to perform a structural fire analysis on a building. These concepts include the combustion process and fuel loads, types of realistic fires that can occur, heat transfer to the structural elements, and finally the assessment of the structure itself. The intention of the chapter is to ensure the stakeholders have an understanding of the science that enables these designs. The following chapter is a literature review to assess the state of structural fire engineering within Canada and where opportunities exist to evolve the practice. This will involve a critique of the existing published Canadian case studies, the education system that supports PBFD in Canada, and the available Canadian PBFD literature.

3 Chapter: State of the Art for Performance-Based Fire Design and Applications

The purpose of this chapter is to assess the development of fire safety engineering as a field within Canada, with particular focus on the structural design of buildings for fire and the development of performance-based codes. To date, there is not a summary of how structural fire engineering within Canada evolved into its current form of objective-based fire design, along with what research gaps still exist as we progress towards performance-based fire design (PBFD) for our structures. A detailed literature review by Hadjisophocleous et al. (1998) provided insight into how the Canadian code developed relative to international performance-based codes, but it was before any case studies were published for Canadian structural fire design and before the objective-based code was actually introduced. The literature review presented herein will seek to answer where Canada's approach to fire safety engineering deviated from others internationally and why there is a large gap between structural fire design in Canada relative to international practice. From there, it will seek to answer what gaps are preventing the implementation of PBFD in Canada and where opportunity exists for the fire safety practitioner.

3.1 Fire Safety Research in Canada

While the Canadian Society of Civil Engineers (CSCE) and Underwriters Laboratories (UL) were involved in the formation of the standardized fire in 1918, fire safety research in Canada dates back to the post war studies of the late 1940's. The Division of Building Research (DBR) was established in 1947 with Dr. Robert Legget serving as its first director, who then proceeded to organize building research within Canada in the framework of the National Research Council (NRC). During that time, fire

was realized to be one of the major aspects of building construction and safety that needed to be addressed and its inclusion as a research topic was necessary in order to provide scientific and technical support to the Canadian construction industry as well as the Associate Committee on the National Building Code (ACNBC) (Harmathy & Shorter, 1983). The importance of fire safety research receiving renewed attention at the end of the 1940's with a fire incident in Toronto, Ontario. On the evening of September 16, 1949, The S.S. Noronic, a Great Lakes Cruise Ship, was docked in Toronto when a fire broke out in the laundry room. At the time there were 695 passengers on board with many of them being asleep. The fire resulted in 118 deaths. A photograph taken from that evening is shown in Figure 3.1.

Figure 3.1: Fire aboard the S.S. Noronic. Sept. 16, 1949
(Toronto Archives Item C-59-3-0-17-1)

As a direct result of the S.S. Noronic fire, the Fire Research Station (FRS) was established within the DBR on June 13, 1951, at a conference held by the NRC (Harmathy & Shorter, 1983; Bullis, 1997). Prior to this, fire research had mainly been

conducted through the investigation of real fire incidents with an emphasis on urban conflagration events. These includes the Great Fire of Toronto of 1849 (Williams, 1937) and 1904 (Ministry of Government and Consumer Services, 2015), and the Hull-Ottawa Fire of 1900 (Ottawa and Hull Fire Relief Fund, 1900). With the establishment of the FRS in 1951, small-scale tests such as flame spread, combustibility, ignition temperature of plastics, and flash point on flammable liquids were possible (Harmathy & Shorter, 1983). The scale of testing changed in 1958 when a new facility was constructed with large furnaces for the fire resistance testing of roof, floor, and wall assemblies. An additional furnace was added in 1976 that was capable of testing columns. Much of the testing that was done at the NRC in these early years was focused on developing the fire safety building code provisions within Canada. In 1962 the fire test board, now known as the Standing Committee on Fire Performance Ratings, was established and resulted in much of the NRC's focus being on defining fire resistance ratings for different materials and assemblies to be included in the National Building Code Supplement No. 2 (Harmathy & Shorter, 1983).

In 1958 there was a rare opportunity to witness the full-scale testing of 6 houses, a school, and a community hall as part of the St. Lawrence Burns test series. This research was a joint collaboration between the NRC and the British Joint Fire Research Organisation (BJFRO) which was made possible by the fact that the structures were vacant and needed to be demolished anyway (Gales, 2014). The initial outcome of this test series was that the spatial separation of buildings was incorporated into the National Building Codes based on the measured heat radiative intensities (Harmathy & Shorter, 1983). This test series went largely unreported and unstudied until 2014 when it was

revisited as a form of "grey paper" to compare the observed fire behaviour in large compartments to contemporary design fire definitions (Gales, 2014). It was not until the completion of the Fire Research Field Station in 1981 that the NRC was capable to conduct tests of similar scale. The purpose of this new facility in 1981 was to allow for tests of realistic scale, geometry, and fuel arrangement. It was around this time at the NRC that Harmathy began looking at the response of entire buildings to fire and not just the single elements (Harmathy, 1983; Harmathy, 1976a; Harmathy, 1976b). As well, Tamura, another research officer at the NRC, began to look at smoke movement in high-rise buildings to better understand the tenability aspect (Harmathy, 1976b). Around the mid 1980's, performance-based fire design began gaining traction internationally as the prescriptive codes became overly complex and cumbersome. Canada was active in the international fire community during this time through the FRS but it will be shown that several opportunities were missed.

3.2 International Performance-Based Fire Design

While the research within Canada was focused on fire safety from a prescriptive perspective and on defining the fire resistances of different building materials and assemblies, other parts of the world were assessing the overall structure of their codes to approach the fire safety problem. The first country to adopt a more contemporary performance-based code was the United Kingdom in 1985[1]. The Building Regulations had increased to 307 pages with the goal of preventing severe fires such as those experienced in the past, but in doing so the code become "very prescriptive and

[1] The UK Building Regulations are considered the first since they spoke to specific performance-goals that could be demonstrated scientifically during the design phase. Historical codes and laws which prescribe consequences for failure are not performance-based, such as the Law Code of Hammurabi which states a designer is to be slain if their building collapses and causes a death (Urch, 1929)

understood mainly by lawyers" as stated by Margaret Law (Law, 1991). The shift from prescriptive to performance was made possible by replacing multiple, related prescriptive clauses with underlying functional statements. These statements were vague with phrases such as "offer adequate resistance to the spread of flame" or "stability will be maintained for a reasonable period of time" (Meacham, 1996). For this reason, it was necessary to introduce accompanying documents that fire safety practitioners and authorities could use to confidently apply the engineering principles and deviate from the historically approved solutions.

The FRS had engaged internationally during its inception, with previous collaborations having taken place with at least the U.K., U.S., France, Japan, and Australia (Harmathy & Shorter, 1983). Along with the trend of whole building consideration taking place at NRC, it was the international presence that led to Vaughan Beck having a four month sabbatical at NRC in 1987 (Meacham, 1996). Beck and his colleagues at the NRC began work on probabilistic modeling of fire risk that took into account human movement, fire-growth and spread, and smoke movement in Canadian apartment buildings (Beck & Yung, 1990). The method utilized expected risk to life and expected cost as two performance metrics. At the end of the sabbatical, the method had been developed in its initial form but several deficiencies existed such as not including time effects and having overly restrictive sub-models (Meacham, 1996). When he returned to Australia, the risk-based method was further developed and ultimately led the way to performance-based codes being adopted in Australia. The first iteration of this was the National Building Fire Safety Systems Code in 1991 (Beck, 1991) which, as Beck states, was intended to "provide flexible and technologically advanced procedures,

based on risk assessment modeling, to achieve cost-effective building designs which conform to the fire safety levels implicit in the building regulations" (Beck, 1993). Unfortunately, Canada did not see the same opportunities realized at the conclusion of Beck's sabbatical.

3.2.1 Lessons Learned

When performance-based codes were introduced for the design of fire protection systems, designers were given freedom to deviate from the prescriptive clauses of the existing codes with the goal of achieving cost effective fire protection, design flexibility, and quantified risk (Johnson, 1993). However, since the introduction of these codes in the mid 1980's, the process has not been without challenges. The lessons learned from these challenges provide a learning opportunity for Canada as PBFD sees more adoption.

The first challenge was the availability of information necessary to support the designs since published information on loads, criteria, and methods of analysis were not available (Meacham, 2014). Often times, the performance criteria and fire scenarios to consider were left to the fire protection engineer to define for themselves (Alvarez et al., 2013). For this reason, New Zealand is now introducing guidelines to lay out specific performance categories for buildings and prescribe the exact performance criteria and design fire scenarios to consider (Wade et al., 2007).

Another finding was that the expertise for reviewing designs did not immediately exist within the building department for peer review. Many peer reviews were done by other consultants with the authority having to trust the results of it. In particular, New Zealand city staff did not feel they had sufficient knowledge to competently review the designs being proposed to the building department and specifically asked to be educated

on the methods being used and the new process for approving projects (Buchanan, 1999). Still, it was found that there was not consistent enforcement across the country and that the authority's responsibilities were not always given the same interpretation (Buchanan et al., 2006). Even designers themselves required more education to implement PBFD, but it was found that education was slower and less comprehensive than needed (Buchanan, 1999). Meacham (2014) suggests overall there is a lack of education to support the fire safety engineering discipline and a lack of qualified fire safety engineers.

When performance-based designs were proposed by practitioners, the issue of risk was commonly raised. Given the lack of clear guidance in performance criteria, fire scenarios, and methods of analysis, the risk of fire damage was found to vary between designers who had used different assumptions in their designs (Alvarez et al., 2013). This level of risk could be acceptable or not depending on the opinion of the AHJ and the jurisdiction it was applied to. This is compounded by the fact that alternative solutions are typically compared against the acceptable solutions of the building code, which themselves don't have performance or risk quantified (Buchanan, 1999). As well, the concept of risk is not always acknowledged by regulators which can expose fire protection engineers to liability in the case of a catastrophic event even though the design may have seemed robust and appropriate for all design fire scenarios (Johnson, 2002). There are not yet any precedents that the author is aware of for who is liable when a performance-based design faces a real catastrophic fire event; one which may exceed that which it was designed for.

Lastly, once performance-based designs starting being developed in practice, it was found that the level of detail in published case studies was low (Alvarez et al., 2013).

This means the industry doesn't learn as much as possible from these case studies and there is a lack of transparency. Perhaps a mitigating factor to this is that most consultants were both designers and peer reviewers, helping to increase consensus and knowledge sharing across a specific country (Buchanan, 1999). Unfortunately, building codes not being open to internationally accepted standards became a major challenge in allowing this knowledge sharing to exist between countries with different code environments (Johnson, 2002).

3.3 Full-Scale Structural Fire Observations

Shortly after the UK performance-based building code came into adoption, a fire incident occurred which demonstrated the inherent robustness of contemporary steel-framed construction. A 14 storey building under construction caught on fire in 1990 in Broadgate, UK. Being under construction, the fire protection was incomplete and the sprinklers were not functional. The fire burned for 4 hours with no sign of structural collapse. An investigation into the response of the building found that the structure performed well because of load redistribution, and stated that "it would be worthwhile in the future to investigate the effects of major fires in significant structures to gain a better understanding of the most important mechanisms in practice" (SCI, 1991).

While the momentum for contemporary PBFD arguably began with the lessons learned from the accidental Broadgate fire, the experimental data to support the methods came largely from the British Steel Cardington Steel Tests of 1996. Here, a series of large-scale, non-standard structural fire tests were performed in Cardington, UK, on an 8 storey building of contemporary composite steel construction (British Steel, 1999). In total, 7 tests were performed with varying combinations of protected primary members

and unprotected secondary members, as well as varying compartment sizes and fuel loads. Bisby et al. (2013) provide a summary of each of the tests and the key observations that can be made from each. The tests displayed deflections ranging from 180mm to 1200mm, much higher than anticipated during ambient design, and showed no signs of collapse. A plan view of the Cardington floor plate with the location and scope of the 7 tests is shown in Figure 3.2.

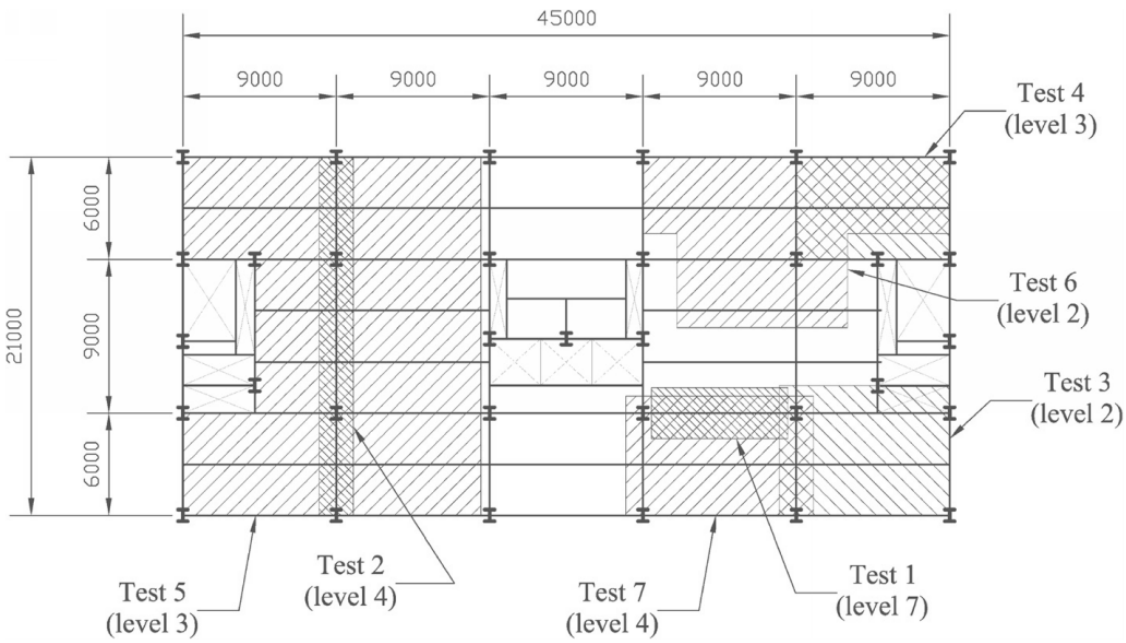

Figure 3.2: Plan view of Cardington test series; reproduced with permission from Bisby et al. (2013)

Specific observations of local failure mechanisms and global behaviour that are relevant to modern PBFD were:

1. Load redistribution of an entire floor plate is possible with tensile membrane action (TMA) at very large deflections (a discussion of TMA is provided in Section 2.8.4);

2. The importance of cooling was demonstrated since connection failures were observed during the cooling phase of test 1 (a single steel beam) and test 2 (a frame running the length of the floor plate); and
3. In order to develop tensile membrane action, higher forces are imparted on the beam-to-column connections, and potentially the beam-to-girder connections. There are also higher stresses where the slab is supported by primary members as demonstrated with significant cracking where reinforcing mesh was missed.

3.4 Modern PBFD Examples

Building on the lessons learned from the Cardington Fire Tests, tensile membrane action has seen considerable adoption as a design mechanism in PBFD, particularly in the UK. Recent trends have also begun to include probabilistic approaches to gain an understanding of the expected risk for a broad range of scenarios, as demonstrated in the following case studies.

3.4.1 Mincing Lane, 2006 (Lamont et al., 2006)

This 11-storey office building in London, UK, had the response of the entire floor considered for the fire design incorporating the tensile membrane behaviour demonstrated in the Cardington Tests. The analysis was first run considering the fire protection requirements of the prescriptive approach. Then, fire protection on the secondary beams was removed and the analysis was repeated. The goal of this was to demonstrate analytically that the protection of the secondary beams was redundant as demonstrated in the Cardington Tests. In addition to considering the whole floor response, another novel aspect of this early case study was the use of several different design fires. These fires included a “short-hot” fire where the glazing is assumed to be

removed, a “long-cool” fire that is predominately ventilation controlled, as well as the standard fire. The reason for including the standard fire as a design fire is presumably because AHJ’s are familiar with it and it gives a point of reference for comparison, even if it has no basis as a realistic fire scenario. The acceptance criteria used for this project was to avoid structural failure (identified by runaway deflections) as well as to maintain compartmentalization. Following the global floor analysis that demonstrated the alternate load support mechanism of tensile membrane action, local connection models were developed to show that connections could withstand the higher forces.

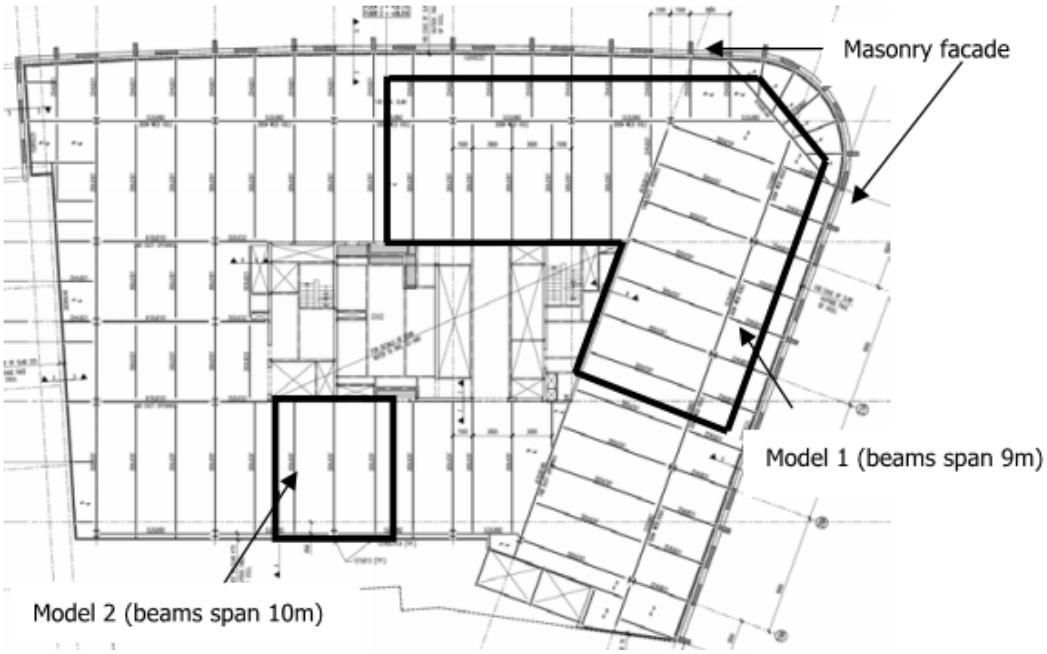

Figure 3.3: Floor plan of Mincing Lane; reproduced from Lamont et al. (2006)

Case studies such as Mincing Lane contributed to a 2012 lessons learned paper by Flint et al. (2013). In this paper, qualitative considerations are discussed for buildings at the fire limit state such as general floor layout, sources of restraint in global structural behaviour, and consideration of connections. These are all valuable considerations for the Canadian practitioner to be mindful of as PBFD increases in usage and demonstrates how they can learn from the experiences of their international peers.

3.4.2 Building EW 11, 2015 (Block et al., 2015)

In this 156m tall residential project in Abu Dhabi, the building had exterior structural steel bracing as the lateral load resisting system of the building. The braces would have required a three hour fire-resistance period based on the prescriptive approach, but instead had a performance-based fire engineering approach applied to eliminate the fire protection on many of them. Although exterior, the braces could potentially be exposed to fire scenarios. The design fires consisted of vehicle fire at ground floor and varying sizes of compartment fires along the height of the building. Where the analysis predicted a relatively low temperature in the brace as a result of the design fires, their residual capacity was calculated using structural fire engineering principles to see if they had to increase in size or have fire protection applied. If the temperatures were found to be much higher, the braces were assumed to fail. Local failure of a brace was acceptable provided that a progressive collapse did not occur and that the forces imparted on the global system did not over-stress any other braces. A rigorous redundancy check was performed on the global structural system for a broad range of fire scenarios, treating fire as an accidental load case according to the American steel design code ANSI/AISC 360-10 (AISC, 2010).

Figure 3.4: Rendering of Building EW-11; reproduced from Block et al. (2015)

3.4.3 Wates House, 2015 (Kho et al., 2015)

In this case study, an existing concrete building had its overall building height increased by adding an additional floor. The prescriptive guidance of the jurisdiction meant that the fire resistance rating of the entire structure would have increased from 60 minutes up to 90 minutes. A probabilistic approach was employed which used a Monte Carlo analysis to calculate rebar temperatures of the reinforced concrete and compare these with the expected rebar temperature at the end of 60 minutes of standard fire exposure (since 60 minutes was the historical period). In order to define acceptance criteria for a probabilistic approach, a risk-based approach was utilized.

$$Risk = (1 - r) \times h^2 \tag{2.14}$$

where

r = reliability index

h = building height

The initial reliability, as defined by Approved Document B (DCLG, 2006), was found to be 80% for an 18m building. A height increase to 20m meant the reliability must be 84% for the same acceptable level of risk. Using the probabilistic approach, it was shown that 84% of all possible design fires had resulting rebar temperatures within the acceptance criteria. By utilizing the probabilistic approach with a Monte Carlo analysis, the design team was able to run 10,000 possible design fires and in doing so eliminated the uncertainties in the variables generally associated with deterministic designs.

Fire engineering that uses a probabilistic approach such as this is now beginning to see more adoption in practice. These probabilistic approaches attempt to quantify levels of risk which had been qualitatively included in fire protection strategies based on historically accepted performance. Woolson (1913) had previously conceived a survey for fire fighters to attempt to incorporate risk associated with fire into the prescriptive guidance of the building code. More recently, methods based on risk quantification were proposed and implemented as the Canadian objective-based building code came into use (Hadjisophocleous & Fu, 2004).

Figure 3.5: Rendering of Wates House; reproduced from Kho et al. (2015)

3.5 Canada's Objective Based Code

Soon after Australia had implemented its performance-based building code, discussions began within Canada on the future of its code and what developments were necessary. Initially, the NRC formed the Associate Committee on the National Building Code in 1948 with the purpose of updating and maintaining the NBCC. A few years later, in 1956, the Associate Committee on the National Fire Code was established in order to update and maintain the NFC. Both of these committees were then merged in 1991 to form the Canadian Commission on Building and Fire Codes (CCBFC), which was the entity to first propose revisiting Canada's current code situation and where development was headed (Bergeron et al., 2004). At this time, Canada was relatively late to the performance-based code system compared to international practice, so the CCBFC and NRC staff were able to assess what was being done elsewhere in the UK, New Zealand, and Australia in order to learn from the best practices and improve on any aspects needing it as described in Section 3.2.1. In 1991, a report was written on behalf of the Canadian Mortgage and Housing Corporation Technical Innovation Division on the Canadian building code development process and found that the requirements had become too complex and that a shift to performance-based code provisions would stimulate innovation (A.T. Hansen et al., 1991). There was growing support for a change in the building code format in the early 1990's. This was a critical time since the CCBFC was in the process of re-evaluating the NBCC.

The CCBFC heard three competing views on performance-based codes while determining the new direction of the NBC. The first group was in favour of performance-based approaches and argued that it fosters more innovation and allows for more

engineering flexibility. The second group was opposed to it and favoured the prescriptive approach due to the efficiency and cost-effectiveness of applying it. The last group was fearful of performance-based codes because international experience had shown that it could lead to an "anything-goes" mentality where it becomes difficult to adequately assess each design and determine which should be rejected or revised (Bergeron et al., 2004). The end result was to retain the prescriptive code in the form of acceptable solutions but to allow innovation through performance-based options, which were referred to as alternative solutions. The CCBFC issued their final report in 1994 that outlined the shift to a more performance-based format within the code as opposed to purely prescriptive. In that report, the reasons for including performance-based provisions were: (CCBFC, 1994)

1. Simplify the code structure
2. Clarify the intent of the code by stating its objectives
3. Allow innovative designs
4. Allow alternative designs that had the same performance as prescriptive solutions
5. Reduce trade barriers in construction and design

The process of revisiting the structure of the code began in 1995 and took ten years to complete, being released in 2005. The key concept behind the updated code is that it is *objective-based*. This means the code still retains all of the prescriptive fire safety provisions, referred to as acceptable solutions, but allows for innovation through "alternative solutions". Unlike traditional performance-based codes that allow the designer to determine the levels of performance required in consultation with all the stakeholders, the performance requirements of the objective-based code are inherent in

the acceptable solutions themselves. This means that any alternative solution must perform at least as well as the acceptable solutions it is replacing. By having this requirement to perform as well as the acceptable solutions, the objective-based code ensures that the inherent safety and performance of the acceptable solutions, which had been deemed acceptable by Canadian society, would be preserved and would not decrease over time (Bergeron et al., 2004). When the code shifted to objective-based format, the CCBFC provided some concerns regarding performance-based codes to ensure that Canada could hopefully learn from international experience with performance-based codes. Those concerns included (CCBFC, 1994):

1. Technical expertise is required by the designers, the regulators, and the AHJ
2. It could be difficult to demonstrate performance when no published method, test data, or verification is available

The objective-based code is organized into three sections, with the first two being the most important with regards to how fire safety is achieved.

Division A: Compliance, Objectives, and Functional Statements

Division B: Acceptable Solutions

Division C: Administrative Provisions

The second division contains the acceptable solutions which are the prescriptive clauses that were previously used to achieve a certain level of fire safety. The first division contains the objective and functional statements that underscore each acceptable solution. The objective statements describe the aim of the code, and speak to overarching themes such as safety, health, accessibility, fire protection, and protection from water damage. The functional statements then describe these overall objectives in terms that are

more specific, although still not quantitative. They describe the desired outcome of the acceptable solution to which they pertain, but they do not describe a method to achieve that. An example of an acceptable solution that might be used for a structure to achieve the required fire resistance is shown below, with the objective and functional statements that apply to that solution.

An acceptable solution that most structures incorporate relates to the fire separation between suites or occupancies. A specific example is the fire separation between the floors of office type occupancy. According to OBC, the office occupancy is Group D, *Business and personal services occupancy*. Once the building occupancy is determined, clause 3.2.2 guides the practitioner to specific requirements related to that occupancy and building size, as seen in Figure 3.6.

3.2.2.49. Group D, Any Height, Any Area

(1) Except as permitted by Articles 3.2.2.50. to 3.2.2.56., a *building* classified as Group D shall conform to Sentence (2).

(2) Except as permitted by Article 3.2.2.16., the *building* referred to in Sentence (1) shall be of *noncombustible construction*, and,

(a) except as permitted by Sentence 3.2.2.7.(1), the *building* shall be *sprinklered* if it is regulated by Subsection 3.2.6.,
(b) floor assemblies shall be *fire separations* with a *fire-resistance rating* not less than 2 h,
(c) *mezzanines* shall have a *fire-resistance rating* not less 1 h,
(d) if the *building* is not *sprinklered*, roof assemblies shall have a *fire-resistance rating* not less than 1 h, except that in a *building* not more than 1 *storey* in *building height* this requirement is waived, and
(e) *loadbearing* walls, columns and arches shall have a *fire-resistance rating* not less than that required for the supported assembly.

Figure 3.6: OBC 2012 excerpt, Group D occupancy requirements (MMAH, 2012)

To understand the intent of this clause, the objective and functional statement must be referenced. The building code contains these statements in the MMAH Supplementary Standard SA-1. In here, each acceptable solution of OBC 2012 is attributed to a series of objective and functional statement. For the two hour fire rated floor described above in Figure 3.6, the specific functional and objective statements are

shown in Table 3.1, and an explanation of these specific objective and functional statements are provided in Table 3.2 and Table 3.3, respectively. All tables are excerpts from OBC 2012.

Table 3.1: Excerpt of Objectives and Functional Statements (MMAH, 2012)

Acceptable Solutions	Objectives and Functional Statements
3.2.2.49	Group D, Any Height, Any Area
(2)	(b),(d) [F03,F04-OP1.2][F04-OP1.3]
	(b)(d) [F03,F04-OS1.2][F04-OS1.3]

Table 3.2: Excerpt of Acceptable Solutions Objectives (MMAH, 2012)

Category	Number	Objective
Safety – Fire Safety	OS1	An *objective* of this Code is to limit the probability that, as a result of the design or construction of a *building*, a person in or adjacent to the *building* will be exposed to an unacceptable risk of injury due to fire
	OS1.2	An *objective* of this Code is to limit the probability that, as a result of the design or *construction* of a *building*, a person in or adjacent to the *building* will be exposed to an unacceptable risk of injury due to fire caused by fire or explosion impacting areas beyond its point of origin
	OS1.3	An *objective* of this Code is to limit the probability that, as a result of the design or *construction* of a *building*, a person in or adjacent to the *building* will be exposed to an unacceptable risk of injury due to fire caused by the collapse of physical elements due to a fire or explosion

Category	Number	Objective
Fire, Structural, Water and Sewage Protection of Buildings – Fire Protection of Building	OP1	An *objective* of this Code is to limit the probability that, as a result of its design or *construction*, a *building* will be exposed to an unacceptable risk of damage due to fire
	OP1.2	An *objective* of this Code is to limit the probability that, as a result of its design or *construction*, a *building* will be exposed to an unacceptable risk of damage due to fire caused by fire or explosion impacting areas beyond its point of origin
	OP1.3	An *objective* of this Code is to limit the probability that, as a result of its design or *construction*, a *building* will be exposed to an unacceptable risk of damage due to fire caused by collapse of physical elements due to a fire or explosion

Table 3.3: Excerpt of Acceptable Solution Functional Statements (MMAH, 2012)

Number	Function
F03	To retard the effect of fire on areas beyond its point of origin.
F04	To retard failure or collapse due to the effects of fire.

The objective of the two hour fire-resistance rated floor assembly prescribed in Figure 3.6 below and above the office occupancy is to prevent unacceptable risk due to fire to persons in or adjacent to the building (OS), as well as to prevent unacceptable risk due to fire to the building itself (OP). The acceptable solution does this by retarding the effects of fire spread and also of collapse due to fire (F03 and F04, respectively).

However, the code does not prescribe which assembly must be used. For that, clause 3.1.7.1 may be referenced as shown in Figure 3.7.

3.1.7.1. Determination of Ratings

(1) Except as permitted by Sentence (2) and Article 3.1.7.2., the rating of a material, assembly of materials or a structural member that is required to have a *fire-resistance rating*, shall be determined on the basis of the results of tests conducted in conformance with CAN/ULC-S101, "Fire Endurance Tests of Building Construction and Materials".

(2) A material, assembly of materials or a structural member is permitted to be assigned a *fire-resistance rating* on the basis of MMAH Supplementary Standard SB-2, "Fire Performance Ratings".

Figure 3.7: OBC 2012 excerpt, determination of fire resistance ratings (MMAH, 2012)

An example of an assembly that can be used to provide the fire-resistance rating is F906, which has been tested to CAN/ULC-S101 and shown to have a two hour fire-resistance rating, if restrained. The assembly consists of an unprotected profiled steel deck with 114 mm of normal-density concrete above the top of the flutes, acting compositely with a steel beam that has spray-applied fire-resistive material. This assembly is illustrated in Figure 3.8.

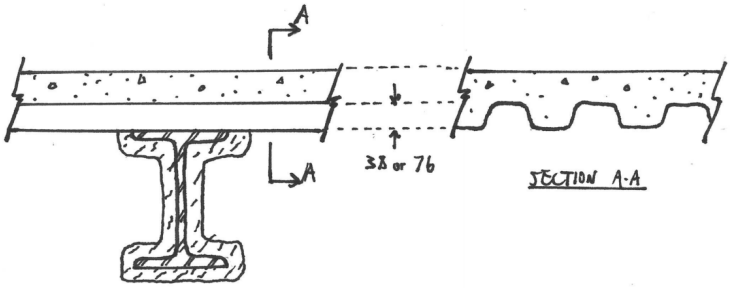

Figure 3.8: Fire-rated assembly F906

The objective and functional statements do not actually describe the performance of the above solution. Rather, the performance of the structure that results from using the acceptable solution is inherent in the solution itself and is not currently quantified within the NBCC. For that reason, if a practitioner wishes to use an alternative solution for the acceptable solution above, it must be done on the basis of *equivalency*. This means the solution must perform at least as well as the acceptable solution, but it is up to the

designer to quantify the performance of the acceptable solution to use as a basis for comparison and this must be agreed upon by all stakeholders. Additionally, because there are often times multiple acceptable solutions for similar situations (for example achieving a floor assembly with a two hour fire-resistance rating), the designer can choose the assembly that has the least level of performance and use this as the basis for comparison. That is to say that the minimum performance of all the acceptable solutions is what will govern any alternative solutions that are implemented.

3.6 Canadian Structural Fire Design

Following the introduction of the objective-based code in Canada in 2005, there did not seem to be a dramatic increase in structural fire design case studies being published in Canada. This is especially true when compared to international practice, where case studies on performance-based structural fire design are quite common. The main source for structural fire case studies within Canada is Advantage Steel, a magazine published by the Canadian Institute of Steel Construction (CISC). The first case study of a project utilizing structural fire design appeared in 2005, the same year as the objective-based code came into effect. A timeline of Advantage Steel structure fire case studies is presented in Figure 3.9, followed by a discussion of common themes that appear in Canadian practice. As noted by Bergeron (2008), there is a trend towards performance-based design solutions and the objective-based code is helping guide the way along that path with the alternative solutions that it permits.

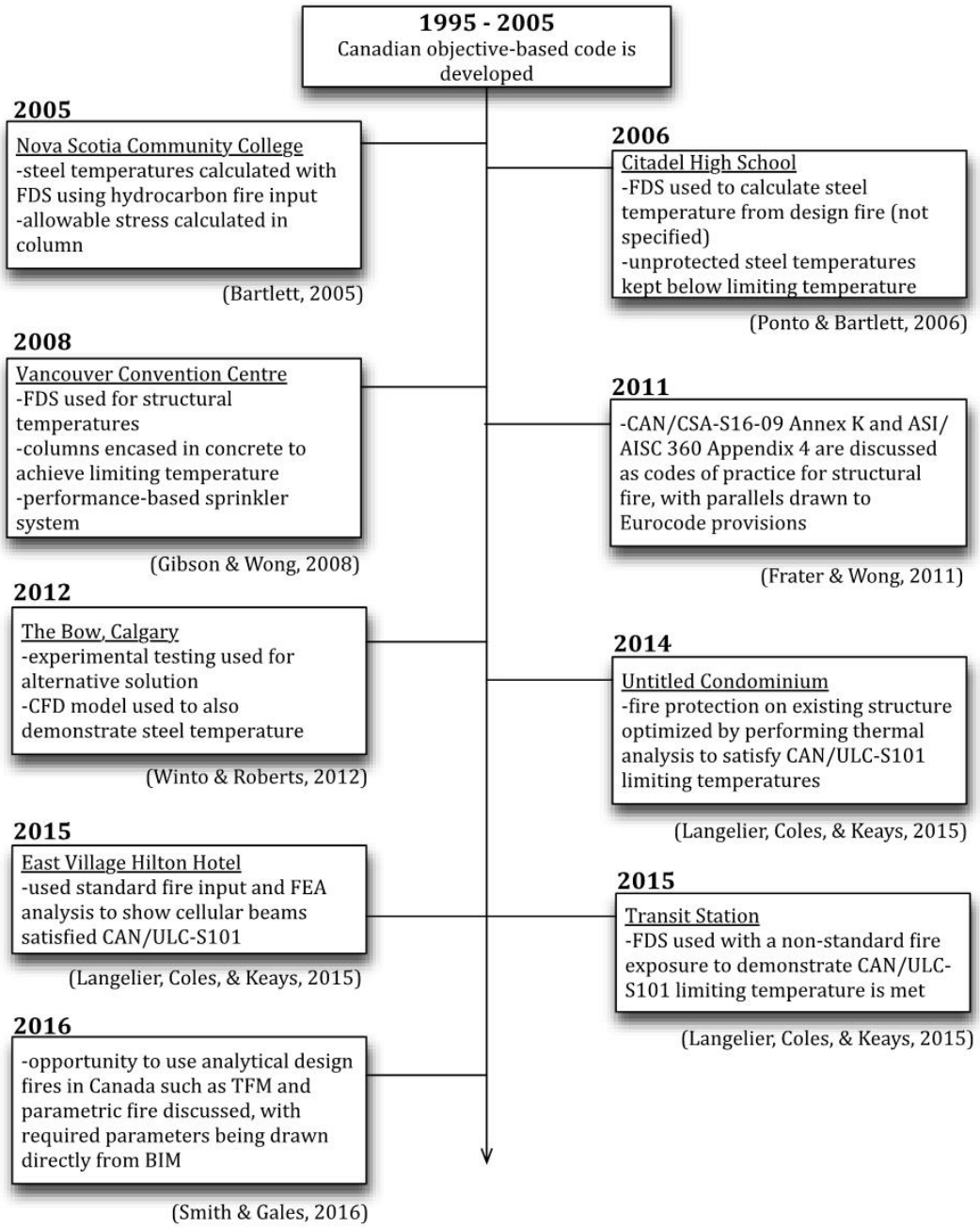

Figure 3.9: Canadian structural fire case studies published in Advantage Steel

When the objective-based code was first introduced in 2005, the case studies available in the literature immediately became more performance-based. This is especially evident in the 2005 case study of the Nova Scotia Community College. In this example, the AHJ asked that the project team follow the procedure outline in the SFPE Engineering Guide to Performance-Based Fire Protection Analysis and Design of

Buildings (Bartlett, 2005). The performance-based process that was used is shown below in Figure 3.10.

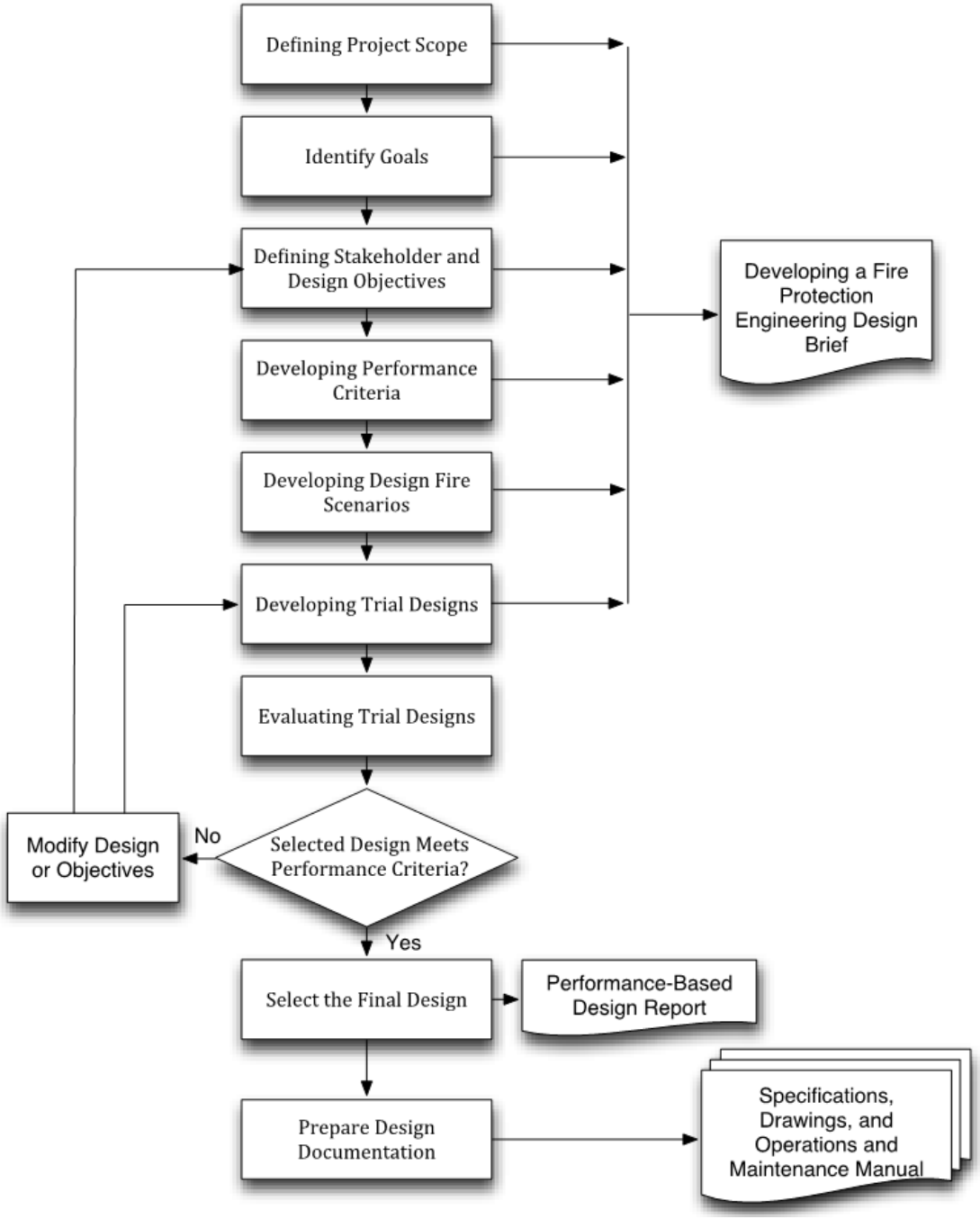

Figure 3.10: Performance-based design process; adapted from SFPE (2000)

This first case study that used the performance-based process is also the only example from Advantage Steel that appears to determine the structural capacity using

reduced properties from the elevated temperature. In that example, the columns had their capacity analyzed by comparing the stress in the column at service load to the available capacity with reduced material properties. In all of the other Canadian case studies, it appears the structure was just assessed based on the limiting temperatures provided in CAN/ULC-S101. For steel beams, this is taken as 538°C while for steel columns this is 593°C (Gewain et al., 2006). The first design was also the only one to reference the SFPE guideline for performance-based design, which seems to suggest there is no consensus yet across the country on which documents are to be followed when deviating from the acceptable solutions. In many cases, the documents to be followed and the methods employed will depend on the specific AHJ and their level of experience in the subject.

Another common theme amongst the Canadian case studies is a heavy presence of computational fluid dynamics (CFD) models to generate structural temperatures, and in particular using Fire Dynamics Simulator (FDS). This is in contrast to international case studies that use analytical correlations for structural temperatures for all but the most complex cases. The design fires were also typically the standard fire (refer to Section 2.3) to show that the proposed design met the requirements of CAN/ULC-S101 (Winto & Roberts, 2012; Langelier et al., 2015), except for earlier examples which appeared to use actual fuel loads in the compartment (Bartlett, 2005; Ponto, 2006; Gibson & Wong, 2008). Recently, Smith and Gales (2016) proposed there is an opportunity in Canada to use analytical models for design fires instead of CFD models, such as the Eurocode parametric fire or a travelling fire. See Appendix E. This approach is adopted in the alternative solution of Chapter 5.

Based on the available Canadian literature, it can be seen that the most common type of analysis is to calculate steel temperatures based on the standard fire exposure using a finite element program, and compare these temperatures with the CAN/ULC-S101 limiting temperatures. The author believes this could be a result of the way in which the objective-based code is written, where alternative solutions need to show equivalence to acceptable-solutions which themselves were subjected to standard fire exposure.

3.7 Towards Performance-Based Design in Canada

As discussed above, the current code in Canada is objective-based. This means the performance is already provided in the acceptable solutions in qualitative terms and alternative solutions may be used provided they offer the same performance as the acceptable solutions. Under the current code, designers cannot determine their own performance criteria with the stakeholders as we see internationally with other performance-based codes that were adopted. This is a key differentiation between the objective-based and the performance-based codes. The two terms cannot be used interchangeably as there are fundamental differences between the two approaches (Meacham & Custer, 1995). It can be seen through the case studies that there is a gradual shift towards performance-based design in Canada, and the objective-based codes are acquainting professionals and regulators involved with the process of calculating and demonstrating levels of performance.

Although the shift towards performance-based design is occurring, there are still barriers that need to be addressed for it to be fully adopted. Kodur (2012) outlined ten barriers to performance-based design, with a focus on North American practice. Many of these barriers continue to be challenges for the fire safety community at large, such as

developing high temperature models of new and emerging materials, creating more accurate and reliable sensor technology, making available test data for model verification, and having opportunities for full-scale testing of abandoned buildings. Other barriers seem to have had recent advances that help move current practice towards a more performance-based approach.

Recent advances in North America include defining acceptable tools and acceptance criteria for use in design. As mentioned in Advantage Steel Issue 39, Canada currently has the S16-09 Annex K that describes fire as a load case for steel structures, while AISC 360 has Appendix 4 which describes the same (Frater & Wong, 2011). These have been described as codes of reference for structural fire design and provide a reference that designers and authorities can use to inform the structural analysis procedures. Currently, the AISC Fire Protection Committee is developing a guideline that will provide industry consensus on how to perform a structural fire design. This publication is expected to greatly advance PBFD in North America. A similar process is being started with CSCE (for which the author is a member). This is very similar to the UK adoption of performance-based design, where approved documents opened the door for performance-based design to be a legitimate method of providing fire protection (DCLG, 2006).

3.7.1 University Curriculum

An area that can still use improvement in Canada and much of North America if PBFD is to see widespread implementation is the University curriculum that will develop the next generation of fire safety engineers. The author reviewed all Canadian universities actively conducting fire safety research and offering degrees in the subject

and found a lack of structural fire material being taught. The three universities that were found to offer fire safety engineering courses were the University of Waterloo, Carleton University, and University of Toronto. It was found that the only course offered at each with a focus on structural fire behaviour is a graduate level course titled "*Fire Resistance*". As part of this research, the author completed the course at each university, either for credit or audited with permission from V. Kodur, B. Weckman, H. Mostafaei and J. Gales, to assess the curriculum. The University of Toronto has ceased to offer Fire Resistance at the time of writing since the adjunct professor teaching it has relocated to FM Global. There are many strengths in the Canadian fire engineering curriculum at the universities discussed which have trained many talented architects, code consultants, and fire fighters to apply fire protection methodologies in the interest of public safety. However, to improve the structural fire curriculum to train engineers for tomorrow's design problems, we must be critical in the following discussion. An overview and comparison of the course *Fire Resistance* across the three universities is presented in Table 3.4.

Table 3.4: Comparison of Graduate course Fire Resistance in Canada, 2015

	University of Toronto	**Carleton University**	**University of Waterloo**
Course Syllabus and Reading Content	Structural Design for Fire Safety, A. Buchanan 2001		
	Post-earthquake fires Coupling of test with model NRC testing overview	Fire resistance of masonry	Investigation of 9/11 Overview of S101 standard OZONE software usage
	Course Assignments		
Application of Equations	80%	100%	90%
Design Problem	20%	0%	10%
Overall Length	4 hours (2 assignments)	5 hours (2 assignments)	10 hours (5 assignments)
	Course Projects		
Design	100%	60%	20%
Research	0%	40%	80%
Overall Length	4 hours (1 project)	8 hours (1 project)	10 hours (2 projects)
	Demographics		
Undergraduate	1	0	0
MASc/PhD (thesis)	2	7	1
MEng (courses only)	11	4	5
Special student	0	1	2
Local Student	100%	80%	75%
Distance Student	0%	20%	25%
Background majority	Structural Eng (Civil)	Structural Eng (Civil)	Mechanical Engineering

Table 3.4 was created after the author completed the course, both the lecture material and the assigned coursework, at each university and attempted to quantify how much of the course was application of equations compared to design problems. Information on student demographic was provided by the course instructors. By taking the course at the three universities, it is possible to identify the differences across the various institutions and draw conclusions about the fire safety engineering curriculum.

Referring to Table 3.4, it can be seen that each course closely (if not directly) follows the text Structural Design for Fire Safety (Buchanan, 2002) written 15 years ago. Indeed, the syllabus of each course is essentially the table of contents of that text. This particular text covers the essentials of fire dynamics, heat transfer to structural elements, and fire resistance. The text also covers analytical correlations to calculate the resistance of simple structural elements to a design fire scenario. However, developments in contemporary international practice since the initial writing of that textbook are starting to widen the gap between what is being taught and what is being done in practice. Specifically;

1. Consideration of whole floor response, including compressive membrane action and tensile membrane action;
2. Consideration of whole floor response for thermal expansion forces or global structural stability;
3. Design fires for large compartments such as the iTFM;
4. Deterministic design compared to probabilistic design; and
5. Advances in understanding material specific behaviour such as concrete spalling or load induced thermal strain (LITS)

The Canadian university curriculum is currently limited to assessing single structural elements for a fire resistance period. This is reflected in the currently available Canadian case studies that generally assess a structure on a single element basis and use the limiting temperature of that material (refer to Section 3.6). In order for Canadian practitioners to receive training on the most recent methods of assessing structures for fire, one of two options exist; they can either be taught the relevant skills and methods from an employer with sufficient competency contained in-house, or they can receive specialized training on structural fire methods through the research or project component of their graduate studies. The latter option is likely to originate from a supervisor of European training that is experienced in the international state-of-the-art in structural design and knowledgeable in the five knowledge gaps listed above. This means that currently the only formal education that exists in advanced structural fire engineering in Canada is at the graduate level and must be taught outside of the course requirements of that degree. Referring to Table 3.4, it can be seen that the graduate course *Fire Resistance* attempts to include more material than the course text contains and that this additional material is typically related to the instructor's research interests. These topics are however generally limited to one lecture and are more introductory than detailed.

Part of the reason why the material taught for structural fire behaviour is limited is because of the curriculum itself. At the two universities with full fire safety engineering programs, Carleton and the University of Waterloo, each course is found to have between two and three lectures of introductory material on fire dynamics out of ten total lectures. This is necessary since graduate students select their own courses and the instructor cannot guarantee that students in *Fire Resistance* have a background in fire dynamics.

The time available to the instructor is thus reduced so that only the basic introductory methods for each typical construction material can be covered. There is also a lack of actual design problems representative of what the students will encounter in practice, with at most 20% of the assigned problems being considered "design problems" that required consideration of appropriate input parameters, assumptions, and analysis methods.

3.7.2 Structural Behaviour

As previously mentioned, the Canadian steel design standard CAN/CSA-S16-09 contains an Annex K on Structural Design for Fire Conditions (CSA, 2009). This Annex contains a discussion on structural design as well as methods of analysis. The structural design provisions are relatively vague and qualitative, advising that the structure shall have adequate strength and deformation capacity for the duration of the fire. The Annex states this shall be done by constructing a mathematical model of the structure. The limit states that shall be checked for this advanced method of analysis include deflections, connection failures, and overall or local buckling.

In addition to the advanced method of analysis, the annex also contains simple methods of analysis that can be used to check the capacity of individual structural steel members exposed to a fire. Equations are provided for simple tension, compression, and flexural members. These methods are, however, very introductory and focus on single element response. They are only meant to orient a structural engineer with performance-based fire engineering (Frater & Wong, 2011). There appears to be a large information gap between these simple analytical methods that are written out in the annex and the advanced methods which are only alluded to. Yet, the annex does begin to get AHJ's and

practitioners comfortable with the notion of designing structures for specific design fire scenarios.

The analytical correlations given in CAN/CSA-S16 Annex K are generally replicated from the Eurocode which is far more comprehensive (CSA, 2009; CEN, 1993-2005). These specific methods are outlined in Sections 2.8.1 to Section 2.8.3. Section 2.8.4 begins to discuss methods for assessing systems of structural elements (floors) which do not received specific attention yet in CAN/CSA-S16 Annex K.

3.7.3 Material Properties

In order for a practitioner to demonstrate the performance of an alternative-solution under the objective-based code, information on material properties as a function of temperature is required. The importance of these properties, as well as the knowledge gap related to the more specific temperature phenomena of typical construction materials, was highlighted by Kodur (2012). The availability of Canadian published material property information will be outlined below, as well as guidance on where to locate the state-of-the-art internationally. This information is critical for development of performance-based structural fire designs.

3.7.3.1 Structural Steel

Steel is understood to have its structural properties degrade with elevated temperatures. The only available Canadian design guideline is the recently published Annex K in CAN/CSA-S16-09. The mechanical properties for steel at elevated temperature are the same as those specified in Eurocode 3 (CEN, 1993-2005). These properties as a function of temperature are provided in Section 2.7.1.

In terms of the thermal properties of structural steel, the only value provided is the thermal elongation. This is stated as constant at 1.45×10^{-5}/°C. A thermal analysis is required by the annex, however no properties are provided and the annex simply states "temperatures within structural members, components, and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat-transfer analysis". Further information is required for a thermal analysis to be conducted as the structural calculations in the annex require. Specifically, the additional information required for a thermal analysis includes specific heat capacity c_p, thermal conductivity k, density ρ, thermal diffusivity α, and emissivity ε. For common construction materials, these thermal properties are documented by Buchanan (2002) and Drysdale (2011) in textbooks that are recommended readings across many of the Canadian fire safety engineering programs. This should make the references and their credibility more familiar to Canadian practitioners and AHJs. As well, several internationally recognized references available to the Canada practitioner include the Eurocode (CEN, 1993-2005) and SFPE Handbook of Fire Protection Engineering (SFPE, 2016).

3.7.3.2 Concrete

The Canadian concrete design standard, A23.3-14 Design of Concrete Structures (CSA, 2014), does not contain an appendix on fire design similar to the steel standard. Rather, it states at the beginning in clauses 1.2 and 8.1.2 that "concrete structures shall satisfy the fire resistance requirements of the applicable building code". The handbook that contains the 2004 version of the concrete design standard has a fairly in-depth section on fire resistance that outlines the requirements of the National Building Code of Canada (MMAH, 2012). The methods outlined provide fire resistance ratings for various

types of assemblies (wall, slab, and column) and are functions of geometry and concrete properties. There is no mention of material degradation with temperature, actual capacity of structural members, or types of fires to consider during design. Reference is made at the end of the section to the American CRSI document, Reinforced Concrete Fire Resistance, which does contain analytical methods to determine structural capacity during fire for reinforced concrete elements. This CRSI document aligns with the analytical methods within the SFPE handbook for reinforced concrete, and could be a useful reference for the Canadian practitioner. Since none of these fire resistance documents are referenced by the NBCC, approval will need to be given by the AHJ for their use.

Since composite structures are quite common in Canadian practice, Annex K of the Canadian structural steel design standard contains some concrete information for the purpose of including it in composite beam or column calculations. Similar to structural steel, the annex provides reduction factors adapted directly from the Eurocode for the mechanical properties of concrete as a function of temperature. These mechanical properties, as a function of temperature, are provided in Section 2.7.2.

3.7.4 Fire Definition

Performance-based structural fire design relies on the practitioner to specify design fire scenarios to assess the structure for. Currently, in Canada and the rest of North America, a deterministic approach is used which develops a series of fires for the structure and the performance for each fire scenario is calculated and compared with the performance criteria that has been decided upon. Internationally, there is a trend towards a probabilistic design approach where the design fires are described with some of the

parameters as probabilistic functions, and the performance is determined in terms of risk as opposed to a deterministic pass/fail against the criteria.

Categories of design fires defined in the Canadian literature are localized fire, exterior fire, and post-flashover (compartment) fire. These are the three categories of design basis fire defined in Annex K of CAN/CSA-S16-09 (CSA, 2009). The design basis fires are described qualitatively in the annex which requires the practitioner to use other references for the actual calculations, while ensuring the qualitative intent of the annex is satisfied. Internationally, travelling fires are also seeing adoption as design fires, which have some commonality with local fires but are assumed to move throughout the compartment.

3.7.4.1 Localized fire

The Canadian steel design standard describes a localized fire as one that is insufficient to cause flashover. The temperatures are to be calculated by using a radiant heat flux that takes into account the fuel composition, arrangement of the fuel, and floor area occupied by the fuel (CSA, 2009). In general this can be done by defining a heat release rate of the burning fuel package and calculating the resulting structural temperatures using the convective and radiant heat components from the fire. Annex C of EN1991-1.2 provides analytical correlations to calculate heat fluxes on structures for one of more localized fires as a function of fire size, heat release rate, and distance (CEN, 1991-2002). This can also be done using zone models such as CFAST (Peacock et al., 2015) or OZONE (Cadorin et al., 2001), two software packages that see common usage in the Canadian fire safety engineering curriculum. As well, several of the case studies

discussed in Section 3.6 used FDS to determine structural temperatures resulting from localized fires defined by the design team.

3.7.4.2 Exterior fire

In addition to compartment fires that directly impact the structure of the compartment, in some situations the structure outside of the compartment needs to be assessed. This is due to flames projecting from windows or wall openings of a post-flashover compartment. The direct impingement of these flames on external structure needs to be considered. Exterior fires were the main design consideration in the Abu Dhabi case study discussion in Section 3.4.2. The general procedure for calculating the temperature of exterior structures exposed to an interior fire is outlined in Appendix B of EN1993-1.2 (CEN, 1993-2005) as well as in the SFPE handbook (SFPE, 2016). The current method published in the guidelines, however, has several shortcomings including an assumption of uniform heating in the structural element, a steady-state fire as opposed to a realistic fire scenario, and not accounting for the effect of balconies (Block et al., 2015). Updated formulations for exterior fires have been proposed by Kho et al. (2014) to account for these shortcomings.

3.7.4.3 Post-flashover fire

A post-flashover fire is defined as one where the heat release rate is sufficient to cause flashover (CSA, 2009). At this stage, the time-temperature response of the compartment is defined by the fuel load, the compartment geometry, the ventilation characteristics, and the thermal properties of the compartment boundaries. This definition closely aligns with the Eurocode Parametric Fire, which calculates compartment

temperatures as a function of geometry, thermal properties of the linings, ventilation, and the fuel load (CEN, 1991-2002). In the Canadian case studies that were found, the design fires typically did not include post-flashover compartment fires that used the Eurocode parametric equation or other available correlations for maximum compartment temperature. The general trend in Canada it seems is to use FDS for resulting compartment fire temperatures, while international practice is more likely to use analytical correlations for fire temperatures before turning to complex CFD models.

3.7.4.4 Travelling fire

Travelling fires are not currently mentioned in the Canadian fire literature, although they are used internationally as design fires in large open-plan spaces. They loosely fit the description of localized fires according to Annex K of CAN/CSA-S16-09, in that they take into consideration the arrangement of fuel, the fuel composition, and the floor area occupied by the fuel. The main difference is that travelling fires are assumed to move throughout the compartment and consume all of the available fuel, while a traditional localized fire is stationary.

The travelling fire methodology (TFM) began development in its current form in 2009 by Stern-Gottfried and Rein (2012a), although the principle of a localized fire moving throughout a compartment dates back to 1996 with Clifton's fire model (Clifton, 1996). At a very high level, the current TFM formulation assumes a fire of length Lf (m), expressed as a percentage of floor area, that moves through the compartment at a flame spread rate s (m/s) (Stern-Gottfried & Rein, 2012b). The temperatures in the near field where the flame exists are 1200ºC, and Alpert's correlation (Alpert, 1972) is used to calculate temperatures in the far range. The TFM then had revisions made to it by

Rackauskaite et al. (2015) and became the Improved Travelling Fire Methodology (iTFM). The main change was that the near-field correlation used the concept of a flapping angle θ to calculate a near-field temperature other than 1200ºC which was in better agreement with observed real flame temperatures. The flapping angle describes experimentally observed behaviour where the flame fans outwards with increased height and reduces temperatures may be recorded at the soffit of the structure. The iTFM near-field fire formulation is shown schematically in Figure 3.11, with the relevant equations provided in Appendix C of the thesis.

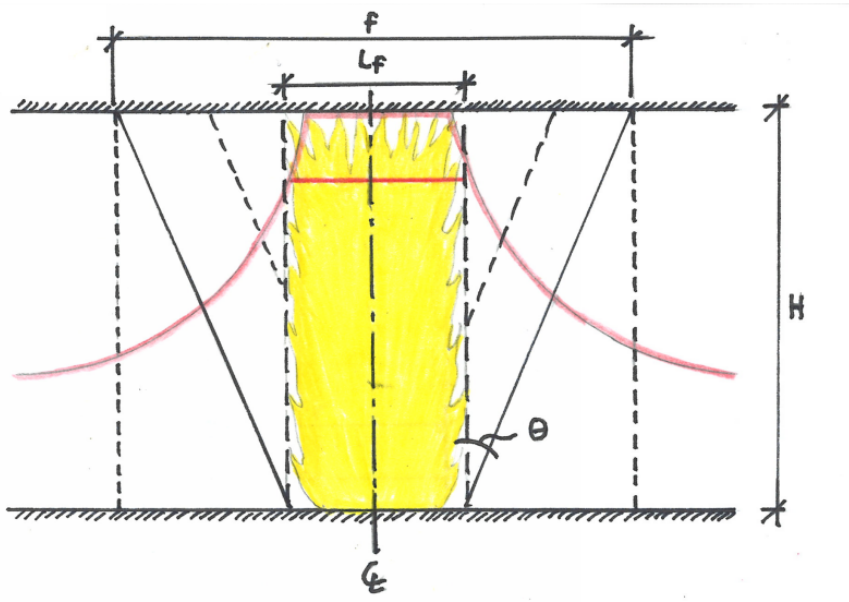

Figure 3.11: iTFM fire plume, adapted from Rackauskaite et al. (2015)

As previously mentioned, the TFM is already seeing adoption as a credible fire scenario to use in design. Typically the TFM is considered alongside other possible fires such as localized fires or post-flashover compartment fires. A current challenge still facing the model is to demonstrate it experimentally with behaviour and temperatures that match the analytical model (Gales, 2014), although several accidental fires have been shown to exhibit behaviour that supports the model (Stern-Gottfried & Rein, 2012a). A

detailed set of calculations for applying the TFM to a real building are provided in Appendix C.

3.8 Summary

This chapter has outlined the development of the objective-based building code in Canada and provided a novel timeline of documented structural fire case studies within Canada. The Canadian examples have been contrasted with the international state-of-the-art to show the disparity between the complexity and general acceptance of the methods being employed. With an understanding of the gap in current practice, the future of Canadian practice was assessed to determine if the available support mechanisms are in place to support implementation of PFBD. Generally speaking, Canadian documentation exists on material properties, design fires, and single element structural analysis. This all forms the knowledge base which is well covered in the education system. There is little available Canadian specific documentation on how to implement whole building performance-based solutions and whether or not this exceeds the limitations and intent of the "alternative solutions" clause of the Canadian building code.

The next section will look at the application of structural fire analysis methods, specifically for replicating benchmark studies of experimental tests. This demonstrates a transparent competency building of structural fire engineering which Chapter 6 will discuss as being necessary if we are to see a more widespread use of PBFD within Canada.

4 Chapter: Benchmark Modelling and Verification

In order for performance-based fire design (PBFD) to see increased usage in Canada, the practitioner needs to be able to demonstrate competency with the tools and methods available. The purpose of this chapter is to show a progression of structural analysis models that were validated against experimental data to develop and demonstrate competency. Chapter 3 demonstrated a trend in the available Canadian literature which suggests contemporary practice is starting to incorporate PBFD. As PBFD increases in usage throughout Canada, it will be important for the practitioner to transparently demonstrate competency to all stakeholders. This was identified by Woodrow et al. (2013) as necessary to develop fire safety engineering as a profession. The first model shown in this chapter will demonstrate a relatively straightforward simply-supported steel beam, which itself has been shown to be a complex problem that lacks industry consensus when being modelled a priori (Lange & Bostrom, 2015). The models will progress to a whole floor analysis from the Cardington test series which was discussed in Section 3.3.

4.1 Description of the Benchmark Modelling Software

The software package chosen for the benchmark modelling was SAFIR, a special purpose computer program for analyzing structures under ambient conditions and elevated temperatures that was developed by the University of Liège (Franssen, 2012). The program uses the Finite Element Method (FEM) to assess both two-dimensional and three-dimensional structures. The thermal and structural analysis are performed separately of one-another, with the results of the thermal analysis subsequently being used in the structural analysis to calculate thermal strains and temperature-dependent

mechanical properties. The SAFIR literature typically refers to the analysis of the structural as a mechanical analysis (Franssen, 2012), while the term structural analysis will be used herein. This is done so that structural fire engineering can be more closely aligned with the broader field of structural engineering, where fire may be one of many load cases considered to act on a building during the design stage. The general workflow developed by the author to model whole floor plates for fire using SAFIR is shown in Figure 4.1.

SAFIR was selected as an appropriate program for the analysis contained herein due to the fact that it has already been validated against experimental data for both the thermal analysis (Zaharia & Gernay, 2012) and the structural analysis (Fike, 2010). As well, being a finite element program that uses text-based input files instead of a user interface, SAFIR lends itself to integration with a broader suite of structural analysis software and BIM (Building Information Modelling) which is important for the consultancy aspect of PBFD within Canada as has been demonstrated by the author during his graduate degree (refer to Appendix E) (Smith & Gales, 2016).

The calculation of the temperatures within the compartment to use for the thermal analysis is not part of SAFIR. In several of the benchmark modelling studies, the temperature of the cross-section itself is specified as a function of time, while in more complex cases an experimentally recorded temperature is provided at the structure's surface which is then used for a thermal analysis. In practice, design fires will have to be developed to determine temperatures (see Section 3.7.4 for design fire definitions).

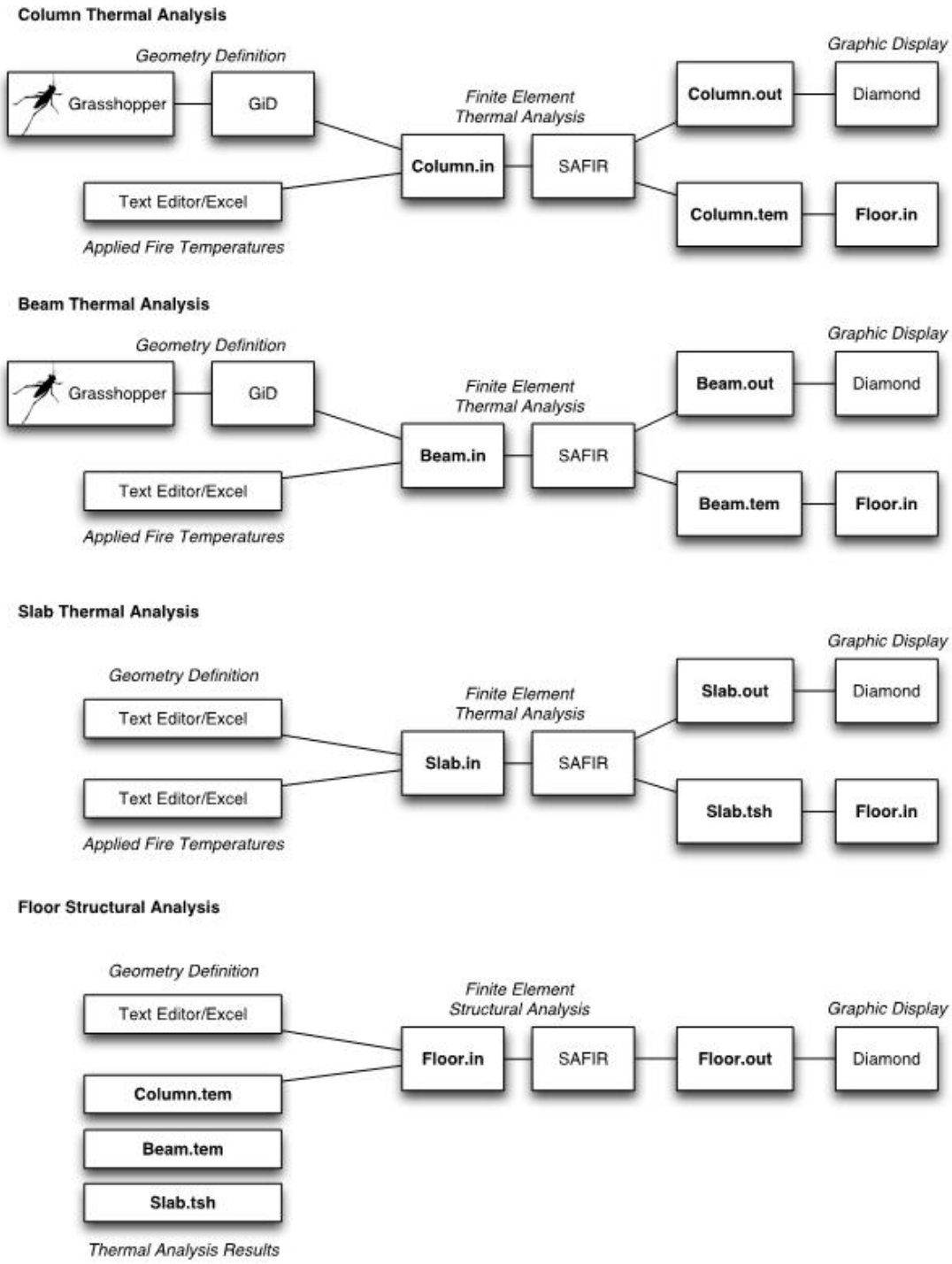

Figure 4.1: Workflow to assess an entire floor for fire in SAFIR

4.1.1 Thermal Analysis

SAFIR's thermal analysis is based on heat conduction through solid materials, described by Fourier's equation and solved according to standard finite element

procedure. The formulation of the thermal analysis is described by Franssen (2005), summarized herein.

The heat transfer equation in its three-dimensional form is given in Eq. 4.1:

$$k\left(\frac{\partial^2 T}{\partial x^2}+\frac{\partial^2 T}{\partial y^2}+\frac{\partial^2 T}{\partial z^2}\right)+q-C\rho\frac{\partial T}{\partial t}=0 \tag{4.1}$$

where

k = thermal conductivity, W/mK

T = temperature, K

x,y,z = coordinates, m

q = internally generated heat, W/m^3

C = specific heat, J/kgK

ρ = specific mass density, kg/m^3

t = time, s

The following discussion on the formulation of the thermal analysis (and subsequently the structural analysis) assumes that the reader is familiar with the basics of FEM. The reader is referred to Zienkiewicz et al. (2013).

The classical shape functions are used so the geometry of the elements can be represented as a system of coordinates of their nodes. An example of a linear rectangular element with 4 nodes is shown in Figure 4.2 below.

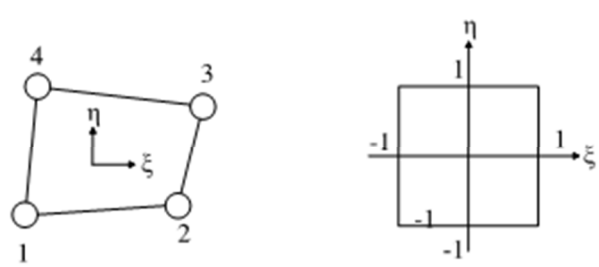

Figure 4.2: Reference rectangular elements and parametric coordinates

Eq. 4.2 results from the rectangular element of Figure 4.2:

$$N = \frac{1}{4}\left\langle \begin{matrix} (1-\xi)(1-\eta); & (1+\xi)(1-\eta) \\ (1+\xi)(1+\eta); & (1-\xi)(1+\eta) \end{matrix} \right\rangle \tag{4.2}$$

Similarly, SAFIR allows for triangular elements with three nodes as well as three-dimensional six and eight node prisms. With the shape functions presented above, the nodes of the elements can have their coordinates represented as in Eq. 4.3 below and their temperatures represented as in equation Eq. 4.4:

$$x = N_i x_i; y = N_i y_i; z = N_i Z_i \tag{4.3}$$

$$T = N_i T_i \tag{4.4}$$

To transform Fourier's heat transfer equation to differential form, Eq. 4.4 is substituted in for the temperature terms and the boundary condition of Eq. 4.5 is substituted in, where $\nabla = \langle \partial/\partial \mathrm{x}; \partial/\partial \mathrm{y}; \partial/\partial \mathrm{z} \rangle$ and q_n is the heat flux at the boundary of the element.

$$q_n = -k\nabla T_j N_j \tag{4.5}$$

As well, the heat flux across the boundary accounts for convection (Section 2.5.3) and radiation (Section 2.5.2) according to Eurocode 1 is given in Eq. 4.6:

$$q_n = h\left(T_g - T_s\right) + \sigma\varepsilon^*\left(T_g^4 - T_S^4\right) \tag{4.6}$$

where

ε^* = emissivity

σ = Stefan Boltzmann constant, $5.67\text{x}10^{-8}$ $\text{Wm}^{-2}\text{K}^{-4}$

T_g = gas temperature, K

T_S = Temperature of structure boundary, K

h = coefficient of convection, $\text{W/m}^2\text{-K}$

With the substitution of boundary condition Eq. 4.5, Eq. 4.1 can be written as:

$$\int_{element} k\{\nabla N_i\}\langle\nabla N_j\rangle T_i dV + \int_{element} C\rho N_i N_j T_i dV^{\cdot} + \int_{element} Q N_j dV$$

$$= -\int_{boundary} N_j q_n dS \tag{4.7}$$

Since SAFIR cannot account for any reactions of the material itself, the term Q for internally generated heat is taken as zero. Summing the contributions of all the elements in matrix form, the total equilibrium of heat fluxes in the structure at a point in time is provided in Eq. 4.8:

$$[K]\{T\} + [C]\{\dot{T}\} = \{g\} \tag{4.8}$$

where

[K] – matrix of conductivity,

[C] – matrix of capacity,

{T} – vector of the temperatures at the nodes,

{g} – vector accounting for the heat exchanges at the boundaries

The matrices in Eq. 4.8 are solved using the method of Gauss, with the number of integration points being specified by the user. Based on the temperature of the element at time step i, the temperature dependent thermal properties are taken into account for the subsequent time step and can vary between integration points.

SAFIR stores the results of the thermal analysis in an output file (.out) that can be viewed with the postprocessor Diamond, as well as in a temperature file (.tem) that can be referenced in a subsequent structural analysis as previously discussed.

4.1.2 Structural Analysis

SAFIR's structural analysis uses the principal of virtual work, which states that the sum of the internal stresses multiplied by the increments of strains is equal to external

loads multiplied by the increments of displacements (which may include rotations). That is to say that at each increment, the internal work equals the external work. The formulation of the structural analysis is described by Franssen (2005) and is summarized below, although the notation of the equations has been altered to be more familiar to the author and intended readers of structural background. The goal is to formulate the conditions of equilibrium of the structure into an equation of form:

$$\{p\} = [K]\{q\} \tag{4.9}$$

where

$\{p\}$ = Applied loads on the structure

$[K]$ = Stiffness matrix of the structure

$\{q\}$ = Displacements at the degrees of freedom

The goal above is to solve for q at each increment. Next, we recall that work is:

$$W = \int F \cdot ds \tag{4.10}$$

where

F = A force vector acting on an object

s = Position vector of that object

We can define the internal work and the external work as:

$$Internal\ work = \int_V \delta\epsilon^T \sigma dV \tag{4.11}$$

$$External\ work = \int_V \delta u^T b dV \tag{4.12}$$

In which the stresses and strains are defined as σ and $\delta\epsilon$, respectively. As well, the displacements are defined as δu with body forces as b. Setting internal work of Eq.4.11 equal to the external work of Eq.4.12, we get Eq.4.13 below:

$$\int_V \delta\epsilon^T \sigma dV = \int_V \delta u^T b dV \tag{4.13}$$

Several substitutions are made into Eq.4.13:

$$\sigma = \boldsymbol{D}(\epsilon - \epsilon_o) + \sigma_o \tag{4.14}$$

$$u = \boldsymbol{N}a \tag{4.15}$$

$$\epsilon = \boldsymbol{B}a \tag{4.16}$$

In the above, the initial stress and strain, σ_o and ϵ_o respectively, are assumed to be zero. The matrix *N* represents the shape functions, the matrix *B* is the nodal displacement matrix, *D* is the constitutive matrix, and *a* is the small displacements at each node. Substituting these into Eq.4.13:

$$\int_V \delta a B^T D B a dV = \int_V \delta a N^T b dV \tag{4.17}$$

Re-arranging, and removing incremental displacements δa from the integral:

$$\delta a \left(\int_V B^T D B dV a - \int_V N^T b dV \right) = 0 \tag{4.18}$$

In the above, K = $\int_V B^T D B dV$, p = $\int_V N^T b dV$, and Eq.4.9 has indeed been reproduced. The effects of elevated temperature from the prior thermal analysis of Section 4.1.1 are included in the strains of each element as well as the constitutive equations. These are evaluated at each time increment taking into account the temperature of each node which will alter the constitutive equations accordingly. By default, temperature effects follow the Eurocode formulations as discussed in Section 2.7.

Several interesting observations are made when comparing the thermal analysis discussion of Section 4.1.1 to the above structural analysis. Since FEM is used for both the thermal and structural analysis, the same general equation is derived although the terms have different physical meanings. Ignoring the capacity term within Eq.4.8, both Eq.4.8 and Eq.4.9 have the general form of applied load being equal to property change times the object's resistance to change. The load is a temperature flux in the thermal

analysis, and a more traditional applied force in the structural analysis. The resistance to change is the conductivity matrix in the thermal analysis and the stiffness matrix in the structural analysis. Lastly, the property change is the node temperatures in the thermal analysis, and node displacements in the structural analysis.

4.2 Thermal Sensitivity Analysis

A sensitivity analysis was performed on the thermal properties of the materials as well as the meshing of the finite element geometry and input parameters from the structural configuration. The scenario considered for this sensitivity analysis was a protected steel beam matching that from ULC listed assembly F906. This is a common fire-resistance rated assembly used in contemporary Canadian structural designs and is the assembly used on the protected steel beams of Chapter 5. The assembly has been shown previously in Figure 3.8. Parameters that were studied for the sensitivity study are summarized in Table 4.1.

Table 4.1: Sensitivity Study Parameters

Parameter	Range Considered	Typical Value	Comment
Finite Element Mesh			
Steel Beam	5mm – 25mm	10mm	Above 11.8mm represents one element only in the flange
Fire Protection	2.5mm – 15mm	7.5mm	Above 13mm represents one element only in the spray
Slab	10mm – 50mm	25mm	
Thermal Properties (for steel, concrete, and insulation)			
Emissivity	0.2 – 1.0	0.7	
Convective Coefficient, hot	10 W/m^2K to 100 W/m^2K	25 W/m^2K	As stated by Drysdale (2001)
Convective Coefficient, cold	1 W/m^2K to 8 W/m^2K	4 W/m^2K	
Thermal Conductivity	0.03 W/mK to 0.24 W/mK	0.12 W/m^2K	
Specific heat	300 J/kgK to 2400 J/kgK	880 J/kgK	

Parameter	Range Considered	Typical Value	Comment
Moisture Content (Insulation)	2.5 kg/m^3 to 20 kg/m^3	10 kg/m^3	
Moisture Content (concrete)	11 kg/m^3 to 92 kg/m^3	46 kg/m^3	
Physical Parameters			
Insulation Thickness	3mm to 26mm	13mm	
Fire Size	25% T to 200% T	S101 curve	Standard fire used, temperatures scaled for analysis

Note: Values from (Buchanan, 2002) unless otherwise noted

4.2.1 Finite Element Mesh

The size of the finite element mesh was varied to determine the effect of mesh on calculated temperature. The analysis was run for each mesh size, and the temperature was recorded at one hour at the locations specified in Figure 4.4 to Figure 4.6. Where no location is provided, temperature was recorded at the extreme fiber of the bottom flange. Figure 4.3 shows the range of finite element meshes considered, as well as the resulting thermal output from each for illustration.

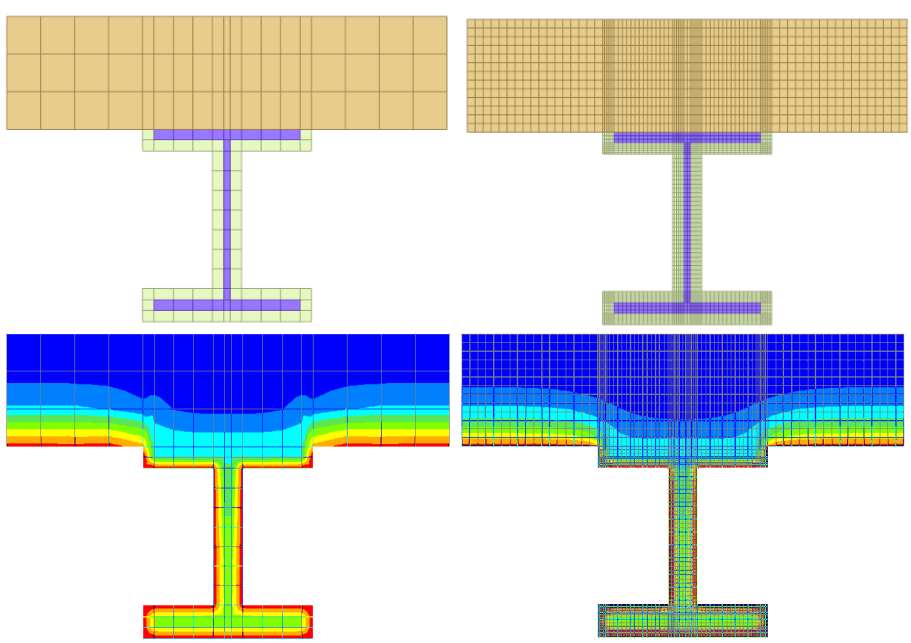

Figure 4.3: Coarsest mesh (top left) and finest mesh (top right), as well as resulting thermal outputs at one hour of S101 standard fire exposure.

The first mesh iteration was done on the number of elements used to model the spray-applied fire protection. For assembly F906, the beam requires 13 mm of protection. The mesh was varied from 2.5 mm up to 13 mm, with the results shown in Figure 4.4.

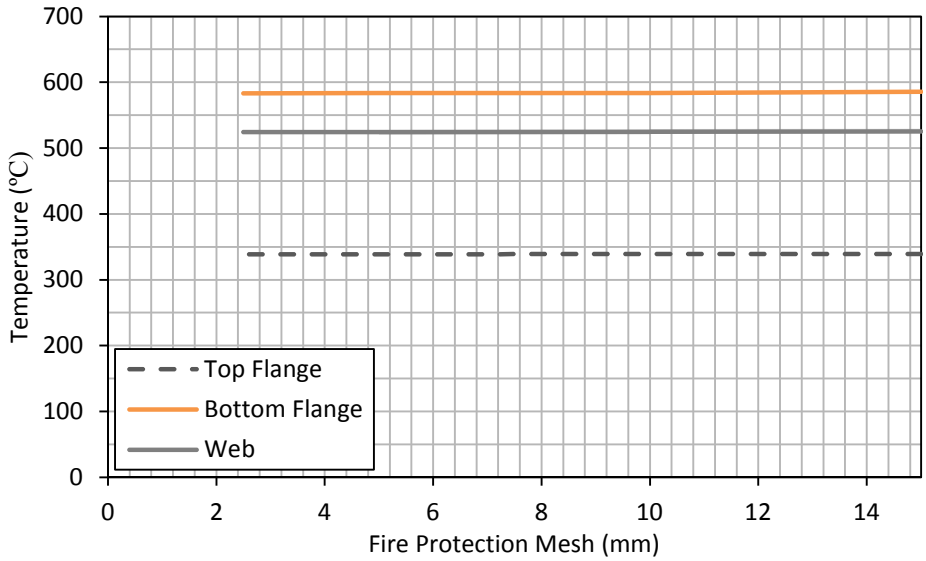

Figure 4.4: Maximum recorded temperatures in steel beam (FP mesh)

Next, the mesh size of the steel beam itself was iterated, as seen in Figure 4.5. During this analysis, the spray-applied fire protection mesh was held constant at 7.5 mm. Figure 4.4 demonstrates that the solution has converged for that size of mesh.

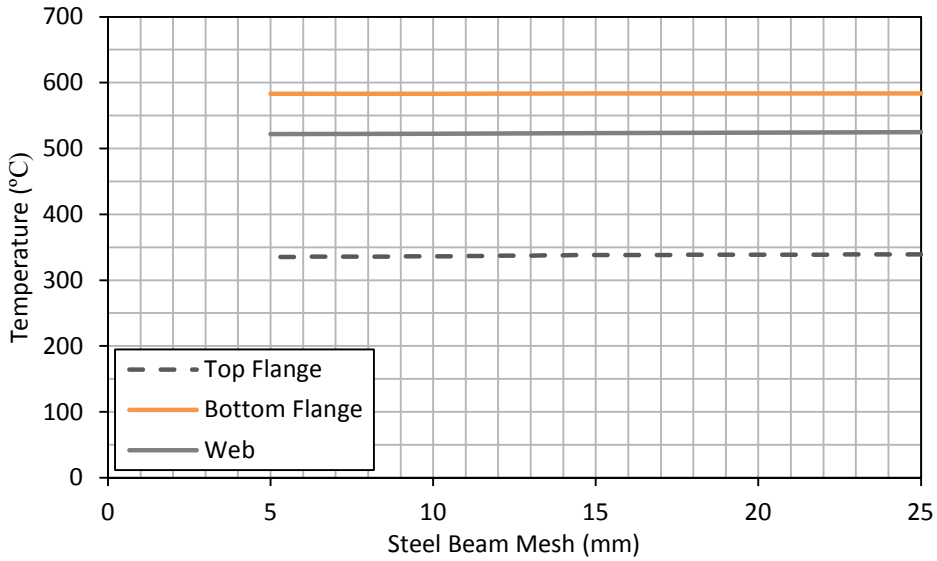

Figure 4.5: Maximum recorded temperatures in steel beam (steel beam mesh)

Lastly, the mesh size of the concrete slab was iterated as seen in Figure 4.6. During this analysis, the spray applied fire protection mesh was held constant at 7.5 mm and the steel beam mesh was held constant at 10 mm. Figure 4.4 and Figure 4.5 show that the solution has converged for this mesh size of the fire protection and steel beam.

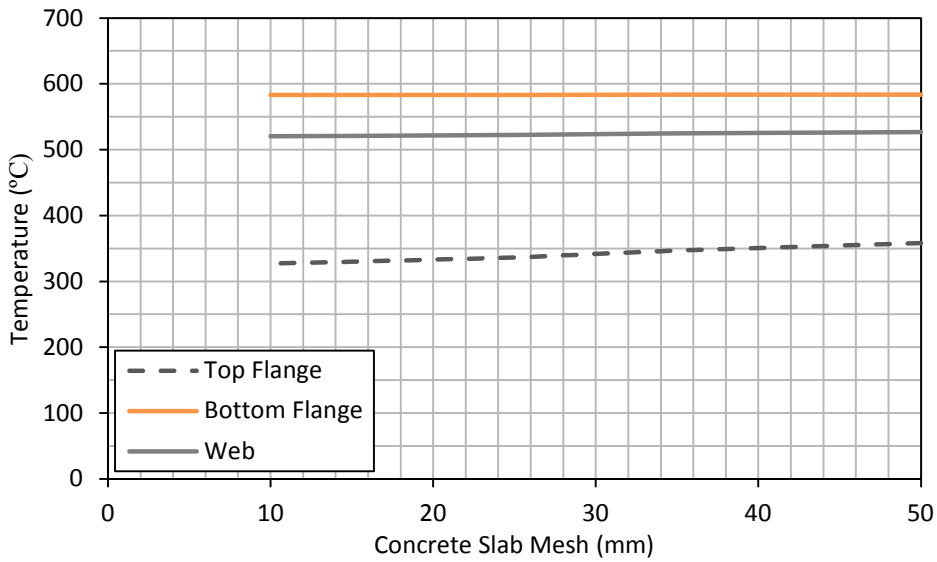

Figure 4.6: Maximum recorded temperatures in steel beam (slab mesh)

In general, the thermal analysis had converged at even the coarsest mesh where the steel beam web and flanges were modelled as one element thick and the spray-applied fire protection was modelled as one element thick as well. The only mesh which seemed to affect the thermal analysis if left too coarse was the concrete slab. With a very coarse mesh, the temperature of the top flange is overestimated. The effect of the heat sink from the concrete slab is underestimated. Since the concrete slab temperatures do not get accounted for in the structural analysis (the slab is modelled separately), it is presumably conservative to overestimate the top flange temperatures with a coarse mesh.

4.2.2 Thermal Properties: Steel

The thermal properties of the steel beam were the first to be analyzed. It should be noted that SAFIR uses the Eurocode thermal properties for structural steel, so the

proceeding section focuses on those thermal properties that vary for each fire scenario and are defined by the user, as opposed to assessing the sensitivity of the Eurocode thermal properties themselves. The thermal properties input by the user for the structural steel are emissivity, convective heat transfer coefficient of hot surfaces, and convective heat transfer coefficient of cold surfaces. None of these parameters were found to have an impact on the resulting temperatures of the steel. This is intuitive because in the situation where the steel is protected, the only method of heat transfer to the steel is through conductive heat transfer at the steel/insulation boundary, and for this SAFIR uses the Eurocode thermal properties which the user cannot change for default steel materials.

4.2.3 Thermal Properties: Concrete

Next, the thermal properties of the concrete were analyzed. The results are shown in Figure 4.7 below.

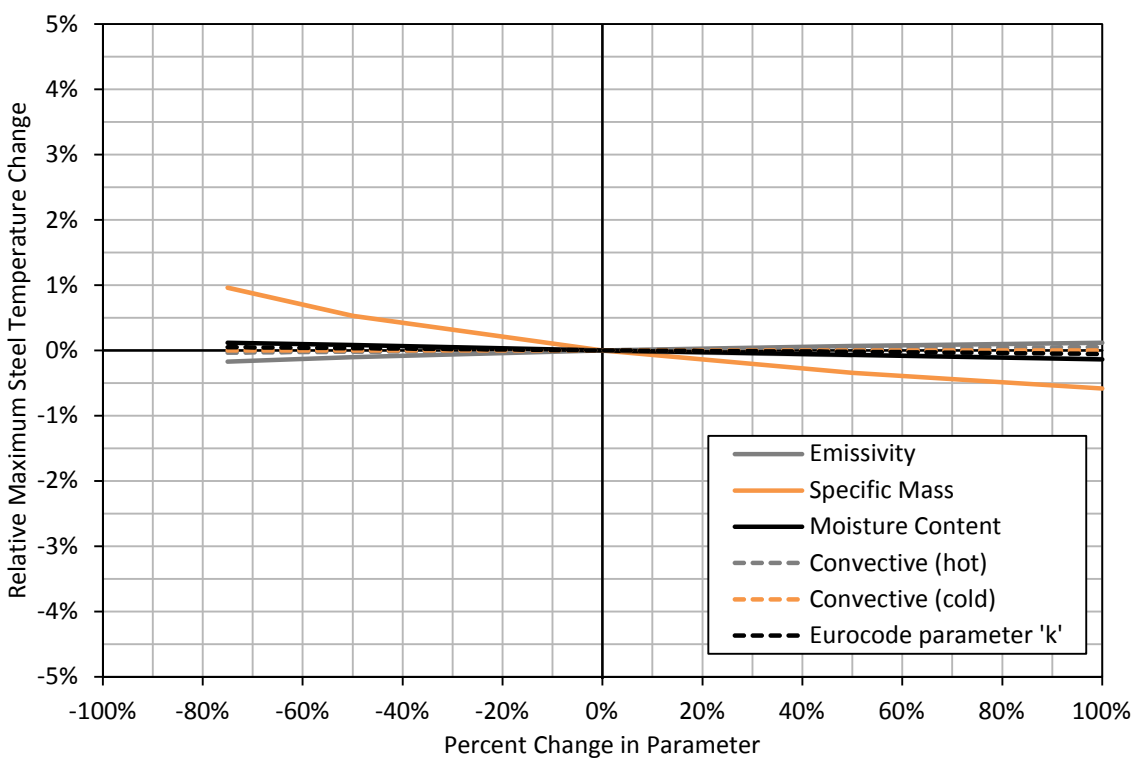

Figure 4.7: Sensitivity of maximum recorded temperature due to concrete slab thermal properties

The maximum temperatures in the bottom flange of the steel beam were most influenced by the specific mass of the concrete slab. At the bottom of the steel beam where the temperatures are highest, the influence from the cooler concrete slab is reduced. As mentioned before, the concrete slab is assessed independently in SAFIR for the subsequent structural analysis as the shell elements are independent of the beam elements.

4.2.4 Thermal Properties: Insulation

The last thermal properties assessed were those of the spray applied fire protection material, as shown in Figure 4.8.

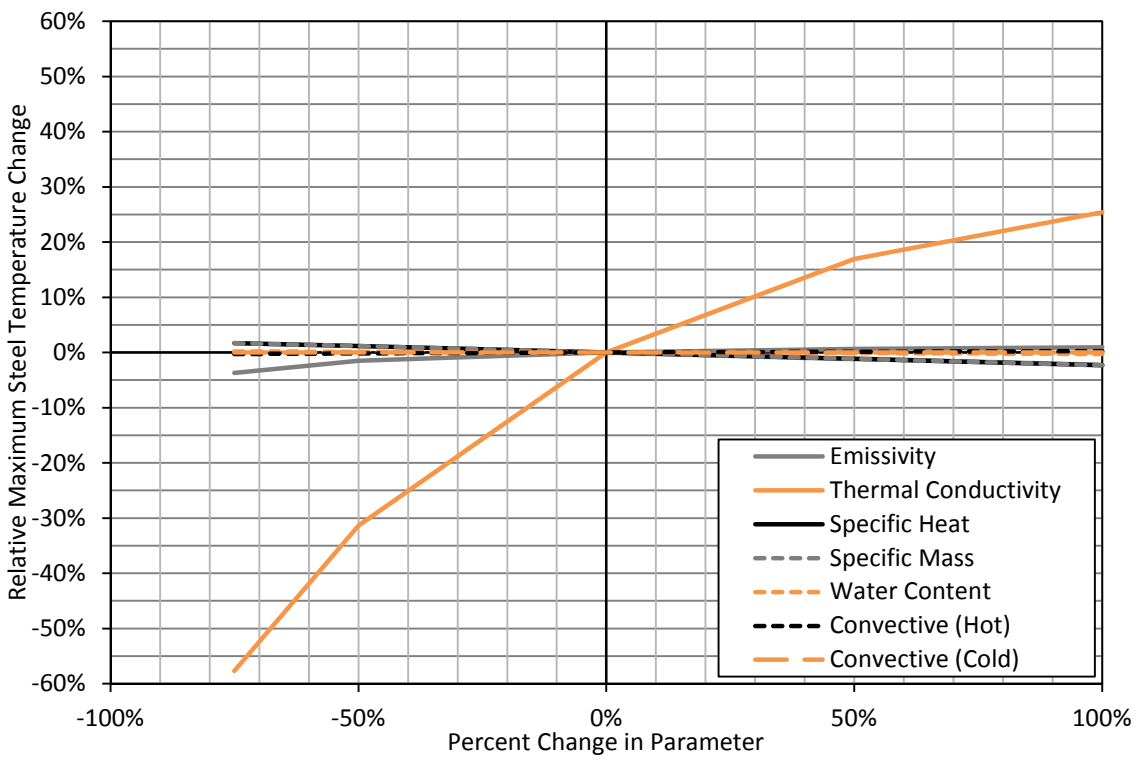

Figure 4.8: Sensitivity of maximum recorded temperature due to fire protection thermal properties

As expected, the parameter having the greatest influence on the maximum steel temperature was the thermal conductivity of the fire protection material. This parameter

directly determines how much heat is transferred through the insulation material, so it makes sense that it heavily influences the steel temperatures.

After thermal conductivity, the emissivity of the fire protection material had the next largest impact on the temperatures. For values below the baseline of 0.7, less heat is actually put into the insulation through radiation and hence the temperatures in the steel appear lower. By definition the emissivity cannot go above 1.0, so any increases in emissivity above the baseline of 0.7 only appears to have marginal increases in steel temperature.

4.2.5 Physical Properties

Lastly, the effects of the physical properties defined by the user were assessed for their impact on the maximum steel temperatures. The two parameters analyzed for this were the thickness of the fire protection material itself as well as the fire temperature as shown in Figure 4.9. Since the standard fire was used, the time-temperature curve was scaled up or down accordingly for the sensitivity analysis.

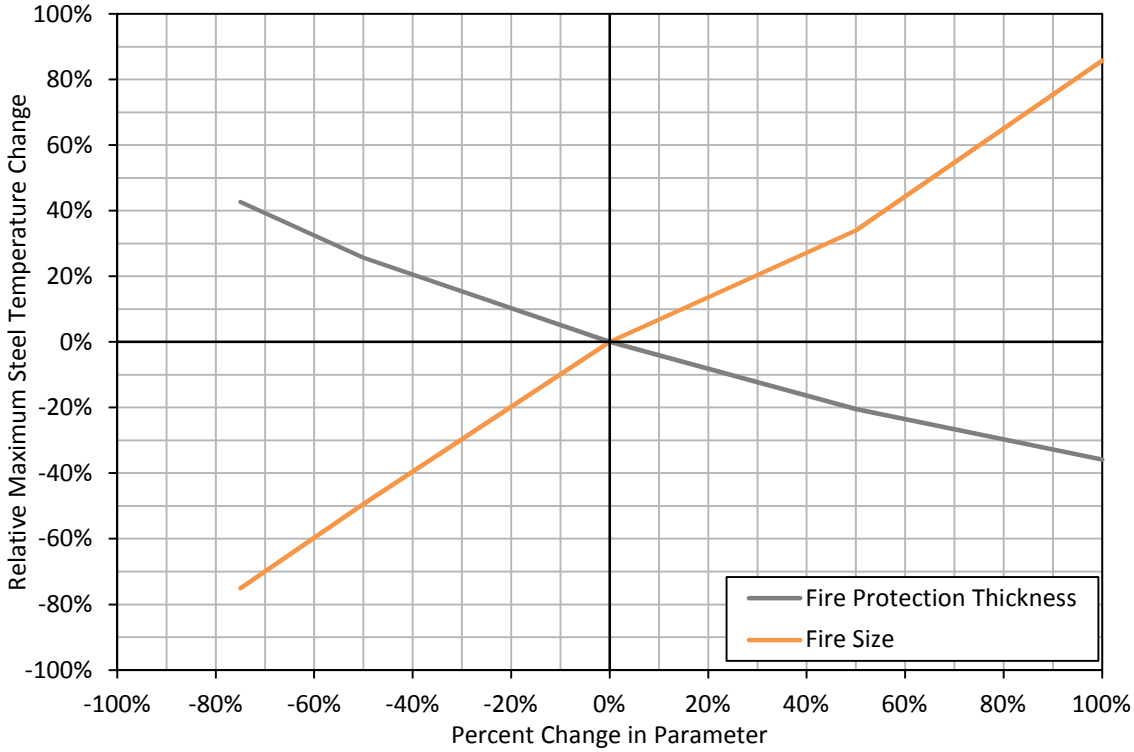

Figure 4.9: Sensitivity of maximum recorded temperature to fire protection thickness and fire size

The fire size has by far the largest effect on the maximum steel temperature. As well, it is expected that the uncertainty in the expected fire temperatures will be higher than the uncertainty in the material thermal properties which shows just how sensitive the calculated steel temperatures are to the fire temperatures calculated by the fire engineer. As for the fire protection thickness, a minimum is typically specified by the designer and it is up to quality control on-site to ensure that this minimum thickness is achieved throughout. The goal is that any deviations on-site will be for thicknesses above that which is specified, and the minimum required is maintained.

4.2.6 Validation of the Thermal Analysis

In addition to serving as a sensitivity analysis, this exercise also served to validate the thermal analysis for the F906 assembly. The solution converged on a maximum temperature of 583ºC in the steel beam, while the temperature limit according to

CAN/ULC-S101 (ULC, 2014) for a restrained steel beam is 593ºC for a 1 hour fire resistance rating. Hence, it appears the thermal analysis is in agreement with the expected thermal behaviour of the assembly even though detailed data of assembly F906 being tested is not available.

The sensitivity analysis presented herein demonstrated that the thermal analysis is not sensitive to the mesh size or the material properties defined by the user. Rather, the results of the thermal analysis are most sensitive to the fire temperatures used as input for the analysis as well as the thickness of fire protection.

4.3 Structural Steel Beam – Fixed Supports

This first validation example is a structural steel beam, with fixed supports, that was exposed to an idealized fire scenario. This validation example does not have accompanying experimental data, but rather analysis results from the program Vulcan (Vulcan, 2016). The example is taken from the COST (2014) document on Benchmark Studies: Verification of Numerical Models in Fire Engineering. All results in the following section that are referred to as "Benchmark" are adopted from the above.

4.3.1 Beam Element Model

The model was first created using simplified beam elements in SAFIR. A UB457x191x98 beam was modelled. The steel beam was assumed protected, with the bottom flange and web having a time-temperature profile equal to 70% of ISO fire and the top flange having a time-temperature profile to 60% of ISO fire. The applied temperature curves are seen in Figure 4.10.

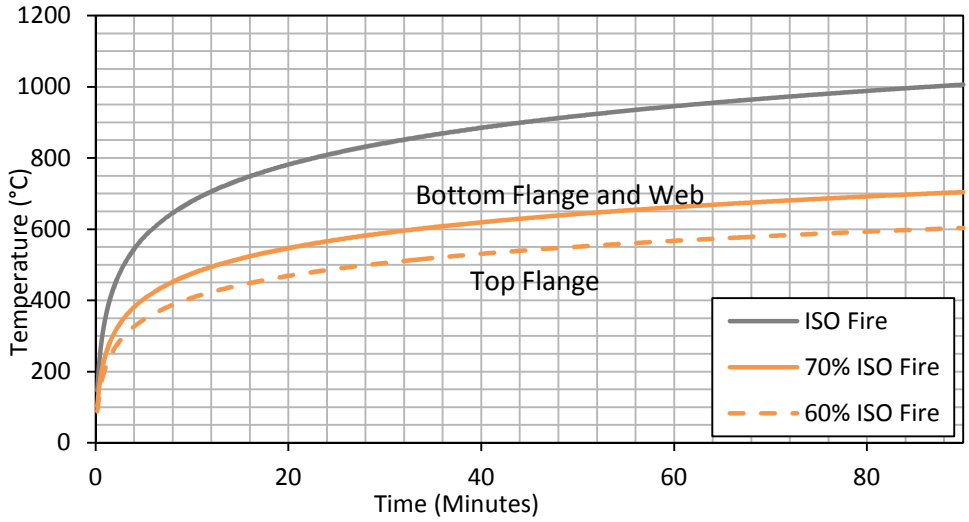

Figure 4.10: Applied time-temperature profile for protected UB457x191x98 beam

The steel beam was simplified in the analysis to not include the bevels where the web meets the flange, or at the tips of the flanges. The effect on section properties is negligible. The dimensions of the UB457x191x98 are shown in Figure 4.11.

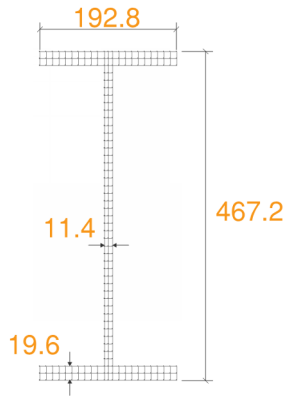

Figure 4.11: Dimensions of UB457x191x98 beam

The mesh used was 22x2 quadrilaterals for the flanges and 2x40 for the web, as validated in Section 4.2. The second moment of inertia of the section is 453.2×10^6 mm^4, compared to the tabulated value of 457.3×10^6 mm^4. This represents a 1% difference.

Unlike the software Vulcan (Vulcan, 2016) which the benchmark study was performed with, SAFIR (Franssen, 2012) is not typically used with temperatures applied directly to the entire cross-section. Rather, the temperature field is usually imported from

a prior thermal analysis that takes into account the heat flux at the perimeter of the cross-section and conduction through the cross-section. This was overcome by applying the temperature field, as shown in Figure 4.12, directly to each node of the beam. This represents the first difference between modelling the test in Vulcan and SAFIR, which is the discretization of the temperature field.

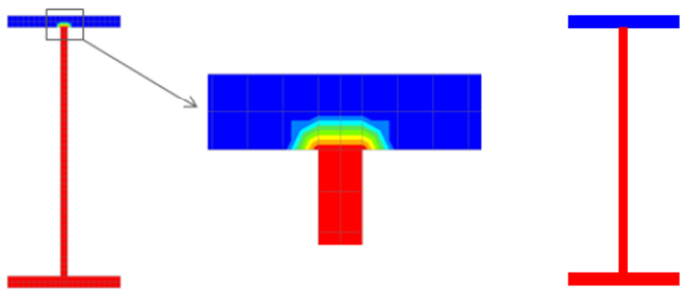

Figure 4.12: Comparison of SAFIR temperature field (left) with benchmark simplification (right)

A uniformly distributed load of 20kN/m was to be applied along the length of the beam in the direction of gravity. First, the structural response was tested under ambient conditions, where each node had 20°C applied to it for the duration of the analysis. A deflection 2.24 mm was found at the middle and 1.26 mm at L/4. These are compared with the calculated values in Eq. 4.19 and Eq. 4.20.

$$\text{Midspan deflection: } \Delta = \frac{wl^4}{384EI} = 2.24 \text{ mm} \tag{4.19}$$

$$\text{L/4 deflection: } \Delta = \frac{wx^2(l-x)^2}{24EI} = 1.26 \text{ mm} \tag{4.20}$$

Based on the above, the structural response under ambient was verified. The heated section was then used in the mechanical analysis, with the deflections shown in Figure 4.13.

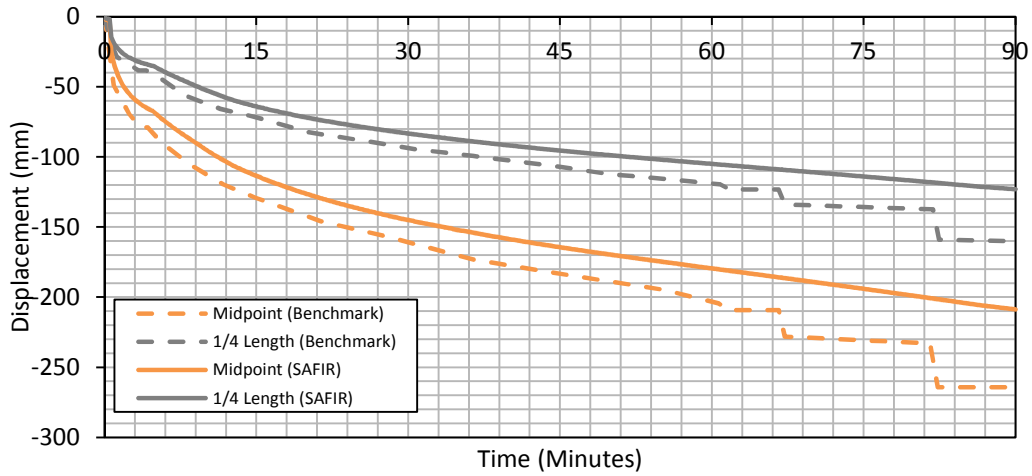

Figure 4.13: Deflection results of UB457 with beam elements and fixed supports

The agreement is not ideal for the heated case. The difference at midspan was 13% up until approximately 70 minutes, at which point it appears the benchmark case developed an internal mechanism that caused deflection to jump and stabilize, with another mechanism at 80 minutes observed. These mechanisms were not captured in the three dimensional beam-element simplification used in SAFIR nor discussed by the authors of the benchmark study. Following this, a shell element model will be created in SAFIR to assess capabilities in capturing the behaviour observed in the benchmark case.

4.3.2 Shell Element Model

The same benchmark case as in section 4.3.1 was repeated, however the model used shell elements to try and capture the benchmark analysis results better. This resulted in a far more complex model as seen in Figure 4.14 due to the quantity of nodes and elements modelled, the attention that had to be paid to the connectivity and restraint of the nodes, and the potential for local instabilities or failures in the web and flange elements. As well, the quantity of shell elements make the model far more

computationally intense compared to the relatively simple model that used beam elements.

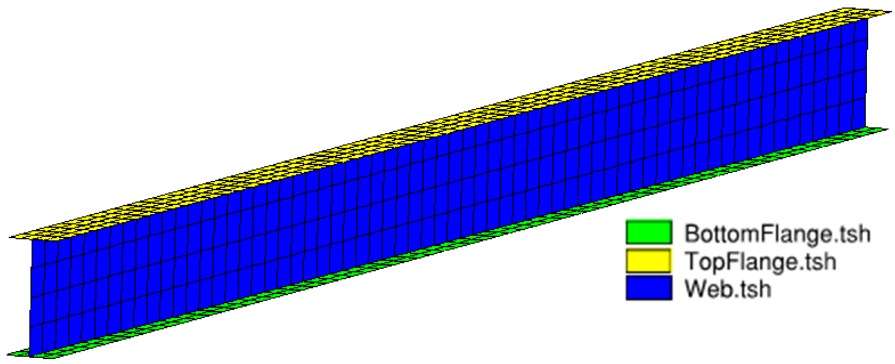

Figure 4.14: UB457 benchmark beam modelled with shell elements in SAFIR

Unlike the model that used beam elements, the restraint conditions of the shell elements require greater attention since it is not just a single node at each end of the beam. Initially, just the nodes at the end of the beam were restrained. After the first model run, it was found that the beam buckles out-of-plan almost instantly due to the compressive forces that develop from thermal strain and the fixed ends, as shown in Figure 4.15.

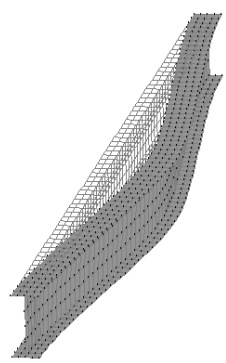

Figure 4.15: Initial analysis results for UB457 shell model

The first time step was 20s, and by then the beam had already buckled. To confirm these results:

$$T_{ISO} = T_0 + 345\log_{10}(8t + 1), \text{ where t is in minutes and T is in °C.} \quad (4.21)$$

$$\text{At 20s, } T_{ISO} = 215°C \tag{4.22}$$

Conservatively assuming the entire beam has been heated to 60% of the ISO fire:

$$0.6T_{ISO} = 128°C$$

The Euler Buckling Load for the beam is calculated as:

$$P_E = \frac{EI_Y\pi^2}{L^2} = 761 \text{ kN} \tag{4.23}$$

Lastly, the force that develops from the thermal strain is calculated as:

$$\text{Thermal strain} = \varepsilon_{TH} = \alpha\Delta T = 1.50\text{x}10^{-3} \tag{4.24}$$

$$\text{Thermal stress} = \sigma_{TH} = \varepsilon_{TH}E = 315 \text{ MPa} \tag{4.25}$$

$$\text{Force} = \sigma_{TH}A = 3938 \text{ kN} \tag{4.26}$$

It can be seen that the force that develops is extremely large, 3938 kN, while the buckling load for the beam was only 761 kN. This confirms the analysis results that the beam had buckled out-of-plane. It appears the benchmark case had the compression flange braced, although this was not explicitly stated by the authors. It may have been implied since the top flange has a lower applied temperature (60% ISO vs 70% ISO) and the presence of a concrete slab above lowers the top flange temperature. The restraint conditions for the shell element model were refined and the analysis was re-run.

Once the top flange was braced, the results were in much better agreement to the benchmark case as shown in Figure 4.16.

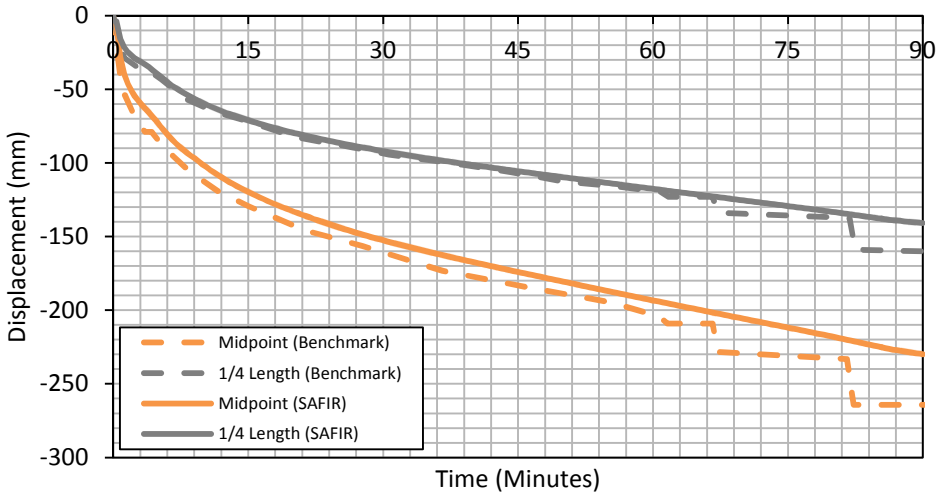

Figure 4.16: Deflection results of UB457 with shell elements and fixed supports

Although the top flange was braced, the bottom flange is also in compression. This is because the fixed support develops a negative moment. Local buckling of the flange was observed almost immediately as seen in Figure 4.17.

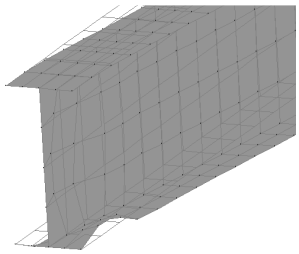

Figure 4.17: Flange buckling observed at fixed end of UB457 with shell elements

Again, the benchmark literature that was used did not mention if the bottom flange was at all braced near the support. The last iteration that was done was to brace the bottom flange for a length of 1 m from the support, to remove the flange buckling from the results. The updated restraint conditions are shown in Figure 4.18 with the updated deflections in Figure 4.19.

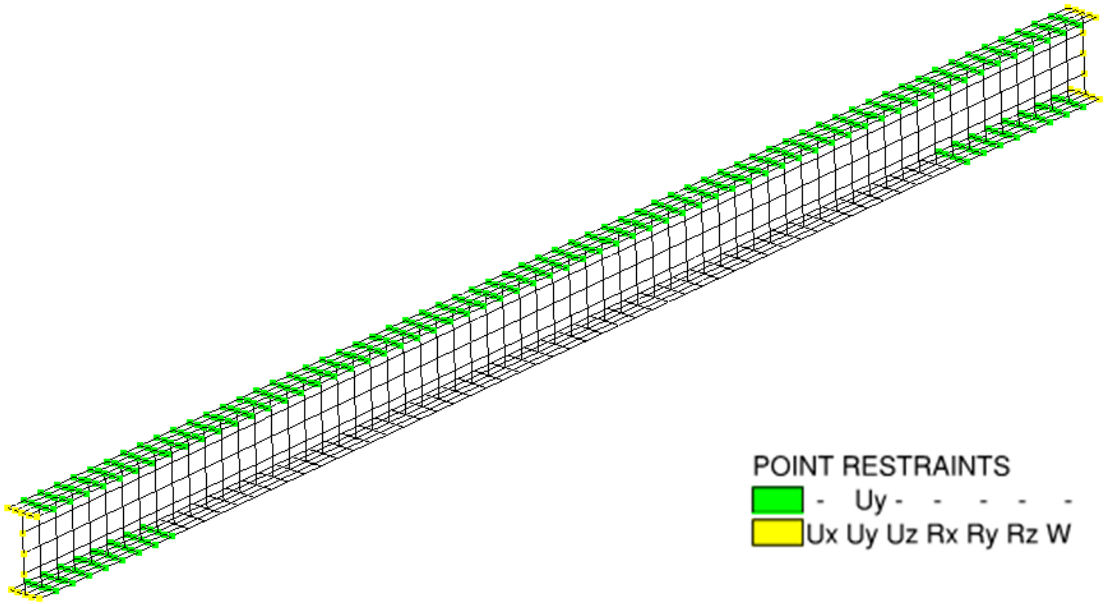

Figure 4.18: Refined support conditions for UB457 with shell elements, fixed supports

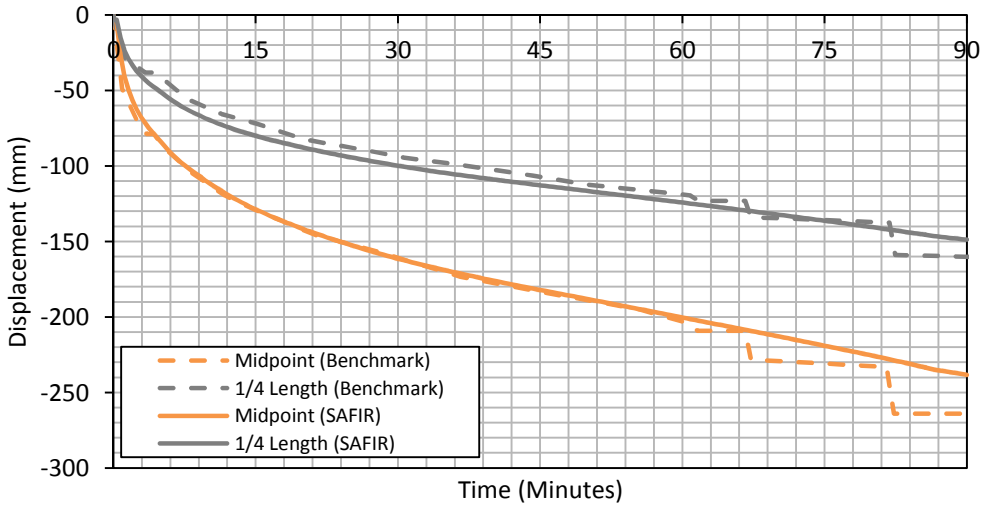

Figure 4.19: Refined deflection of UB457 with shell elements and fixed supports

The model predicted closer results to the benchmark study, yet the mechanisms around 65 minutes could not be captured. The assumptions that the top flange was braced and that the bottom flange was restrained from buckling near the supports appears to be accurate, although there is room for more detail to be provided by the literature for this specific example.

4.4 Structural Steel Beam – Pinned Supports

Similar to Section 4.3, the document Benchmark Studies: Verification of Numerical Models in Fire Engineering was referenced for a benchmark case of a steel beam with pinned supports (COST, 2014). All results in the following section that are referred to as "Benchmark" are adopted from the above. The beam geometry and heating are the same as in Section 4.3 except the supports are pins. This seemingly less complex case was modelled to see if beam elements can be used where supports are pinned, such as in a composite floor. This will make full-floor analysis less complex if beam elements can be validated as opposed to all floor members needing to be shell elements.

4.4.1 Beam Element Model

The model from Section 4.3.1 was adopted, except the supports were changed to be pinned. The results showed very good agreement from the beginning without any refinements needed to the model, as shown in Figure 4.20 below.

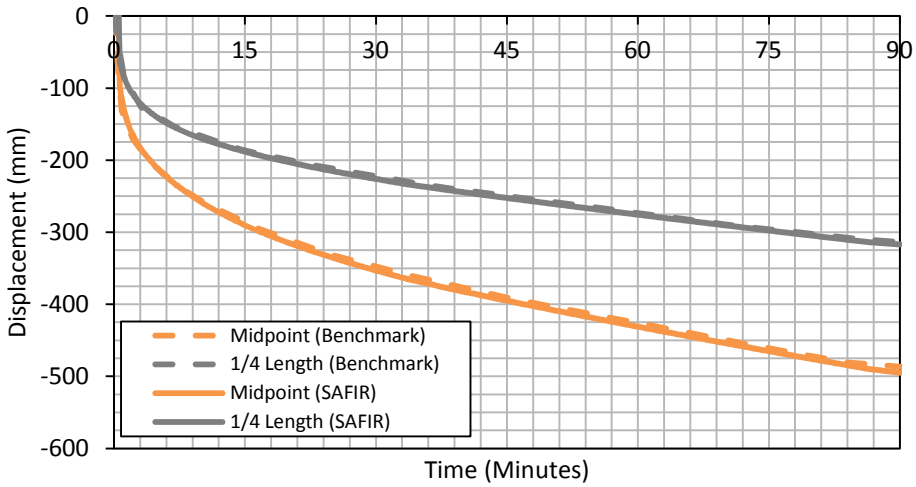

Figure 4.20: Deflection results of UB457 beam element model with pinned supports

The beam element model had an almost perfect agreement with the benchmark results. Additionally, the benchmark documentation provides the axial force in the beam.

Again, the SAFIR model shows good agreement with the benchmark example as seen in Figure 4.21.

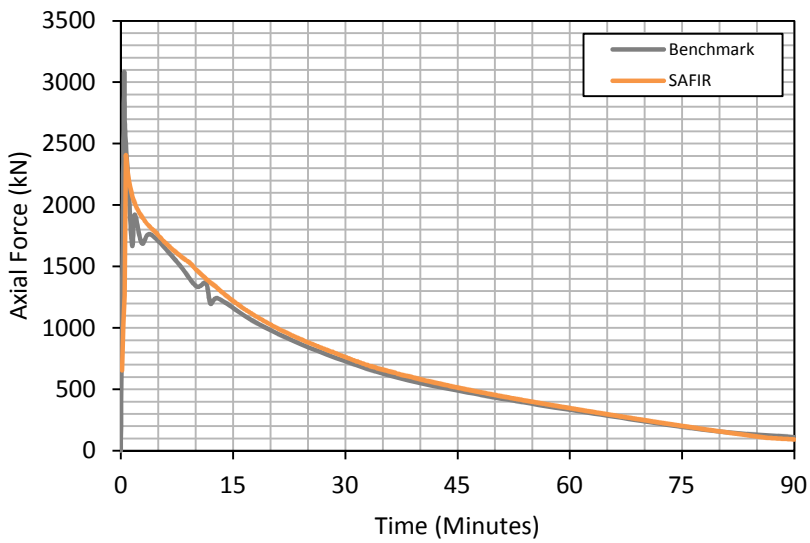

Figure 4.21: Axial force for UB457 with beam elements and pinned supports

To understand if the model with simple beam elements and pinned supports was capturing all of the failure mechanisms, a shell element model of the same scenario was developed.

4.4.2 Shell Element Model

The initial shell element model had a global buckling instability similar to Section 4.3.2 until the top flange was braced, as seen in Figure 4.22. This was due to thermal strains causing very high compressive stresses throughout.

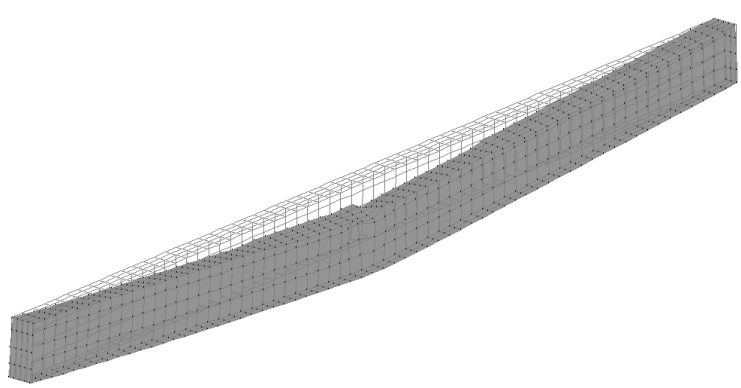

Figure 4.22: Global buckling of UB457 shell element model with pinned supports

Bracing of the top flange was not explicitly stated in the benchmark literature but was rationalized by the presence of a concrete slab which reduces the top flange temperature. The revised point restraints are shown in Figure 4.23.

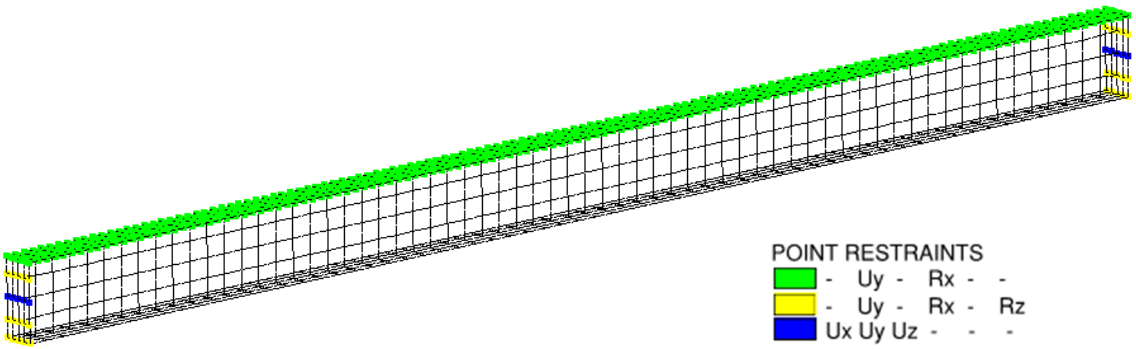

Figure 4.23: Refined support conditions for UB457 with shell elements

With the top flange braced, the beam still experienced a web buckling failure. This can be seen as an instability in the analysis, which caused it to crash at roughly 23 minutes in Figure 4.24.

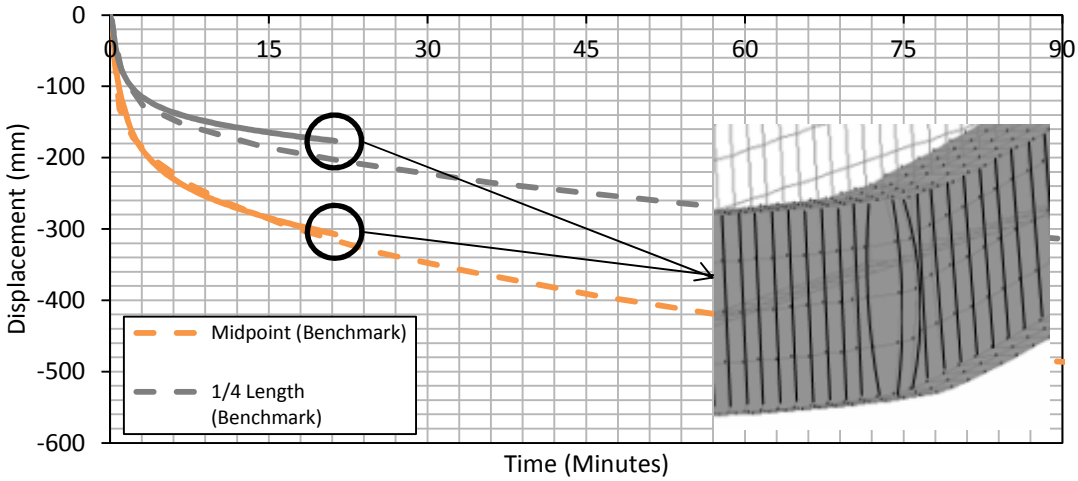

Figure 4.24: Web buckling behaviour of pinned UB457 shell element model

In order to eliminate the web buckling mechanism, which was not observed in the benchmark case, the web of the beam was restrained out-of-plane. Obviously this is no longer representative of reality and would not be done in a design scenario, however it

was done to determine if there are any other differences from the benchmark example. In Figure 4.25, it can be seen that the agreement between the SAFIR model and the benchmark case is better with the web buckling removed from the failure mechanisms.

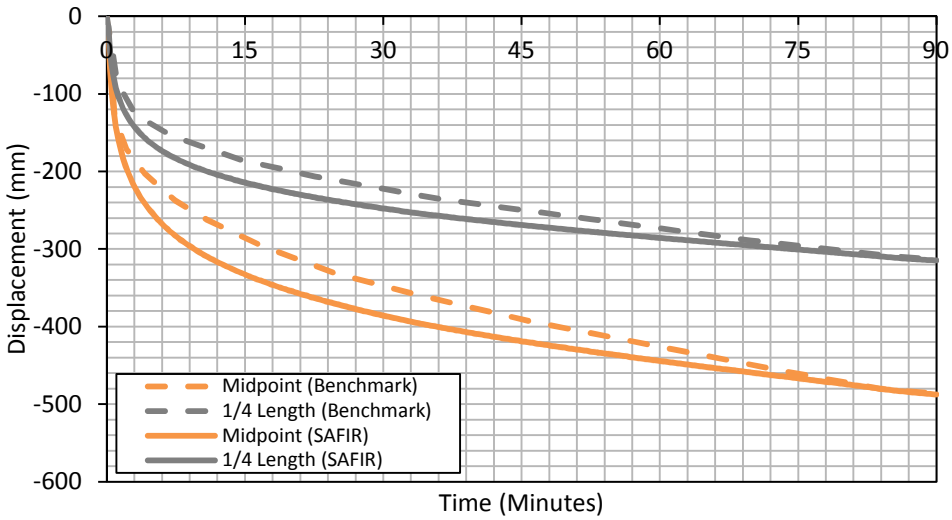

Figure 4.25: Deflection results of UB457 shell element model with pinned supports

The initial deflection of the 3-D shell element model is higher than the benchmark example that was done in Vulcan, however the final result is in good agreement in terms of deflection magnitude. This shows that the benchmark case study, which likely used beam elements, was not capturing local failure mechanisms and that restraint conditions necessary to match the benchmark results were not fully discussed in the literature.

4.4.3 Discussion on Steel Beam Modelling

The steel beam models outlined demonstrate the capabilities to use SAFIR to model benchmark verification cases, with the intent to show competency building from the simple beam element model up to a more sophisticated shell element model that captures local failure modes. Strong agreement with the published deflection responses of the benchmark cases was shown. However, it should be noted that in design applications we cannot expect to arrive at an absolute answer with our computer models. There will

always be uncertainties in the input, including the fire temperatures, the material properties, and the boundary conditions. This has been best shown by a series of round robin studies at SP Sweden where experts from across the fire engineering community were invited to model a relatively simple structural fire test. This test was very similar to the benchmark studies outlined above: a simply-supported wide flange section (see Figure 4.11 for an example) under constant load and exposed to a standard fire. The expected behaviour submitted by the various experts showed a large spread. Different experts assumed different failure criteria, used different heat transfer assumptions, and used different programs to run a finite element analysis (Lange & Bostrom, 2015). After the test was performed, the participants were then provided the actual time-temperature response and ambient material properties to repeat their analysis. Still, there was variation in the results. The mean value for calculated "failure" according to the Eurocode failure criteria was 22 minutes with a standard deviation of 4.15 minutes, while the actual experiment failed at 27 minutes (Lange & Bostrom, 2016).

Although the industry lacks consensus on the tools to use, the assumptions to make during analysis, and how to define acceptance criteria, it is still important to demonstrate competency with the analysis tools available and to be able to match published experimental results. Not all of the decisions/assumptions made during the analysis in the literature were explicitly stated for each benchmark case which did require some tweaking of the author's analysis models to match the results posteriori. In a design situation, one just has to be sure that those assumptions err on the side of conservatism to produce a safe structure and can be justified to an AHJ and peer reviewer.

4.5 Composite Floor

As mentioned in Section 3.3, the Cardington Test Series provided the first chance to experimentally observe the response of an entire floor plate to fire, with sufficient instrumentation to be able to analyze the mechanisms present and validate computer models of the behaviour. Raw data for the test was graciously made available by John Dowling on the BRE Data Lab website (BRE, 2015) for use in this research and other endeavours. The second corner test of the Cardington test series was chosen to validate the modelling approach for composite floors. Figure 3.2 shows a plan view of the Cardington building with the second corner test identified as "Test 4".

4.5.1 Floor Construction

The floor being considered is part of a larger 8 storey building. The building was designed and constructed in accordance with the British and European structural design standards. Composite floors were used throughout which were 130 mm thick lightweight concrete with A142 anti-cracking mesh throughout (Lennon, 1999). A cross-section of the floor profile is shown in Figure 4.26.

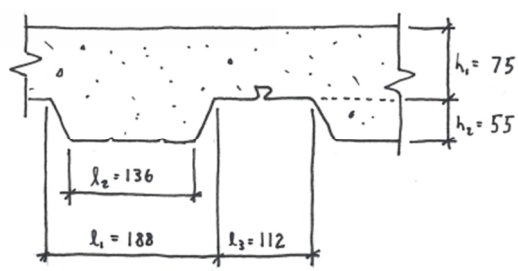

Figure 4.26: Profile of composite slab used in BRE Cardington Building

The floor layout is shown in Figure 4.27 below, and generally consisted of simply supported steel beams supported by girders in typical 9m x 6m bays. Typical connections between beams and girders were fin plates, while the girder to column connections were

partial depth end plates. To simplify the construction of the building, only a few different beam and column sizes were used as highlighted in Table 4.2.

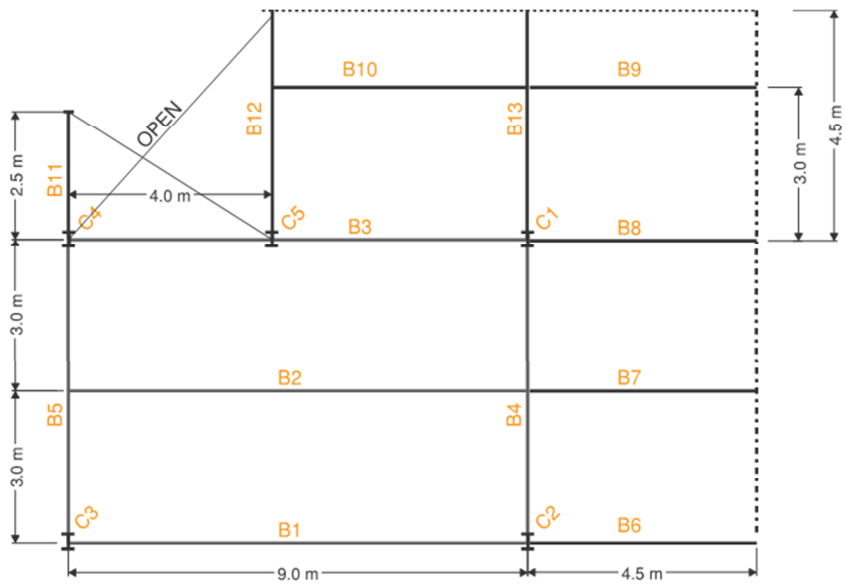

Figure 4.27: Cardington Corner Test floor geometry; adapted from Lennon (1999)

Table 4.2: Cardington Corner Test structural members

Mark	Element	Grade	Mark	Element	Grade
B1	UB356x171x51	S355	B10	UB305x165x40	S275
B2	UB305x165x40	S275	B11	UB356x171x51	S355
B3	UB305x165x40	S275	B12	UB610x229x101	S355
B4	UB356x171x51	S355	B13	UB610x229x101	S355
B5	UB356x171x51	S355	C1		
B6	UB356x171x51	S355	C2		
B7	UB305x165x40	S275	C3		
B8	UB305x165x40	S275	C4		
B9	UB305x165x40	S275	C5		

4.5.2 Thermal Analysis

Unlike the steel beam benchmark studies which had idealized temperatures applied directly to the steel cross-section, the data for the Cardington test had recorded temperatures on the perimeter of the steel members. For that reason, a thermal analysis had to be performed with SAFIR for all of the members. Each floor beam had the temperature recorded at three locations, and each location had between seven and nine

thermocouples installed on it. These typically included one on the middle of the web, and between three and five on the top and bottom flanges as seen in Figure 4.28. Where multiple thermocouples existed on a single flange, the average temperature was used. This way each section of beam just had three heat fluxes applied to it to keep the data manageable.

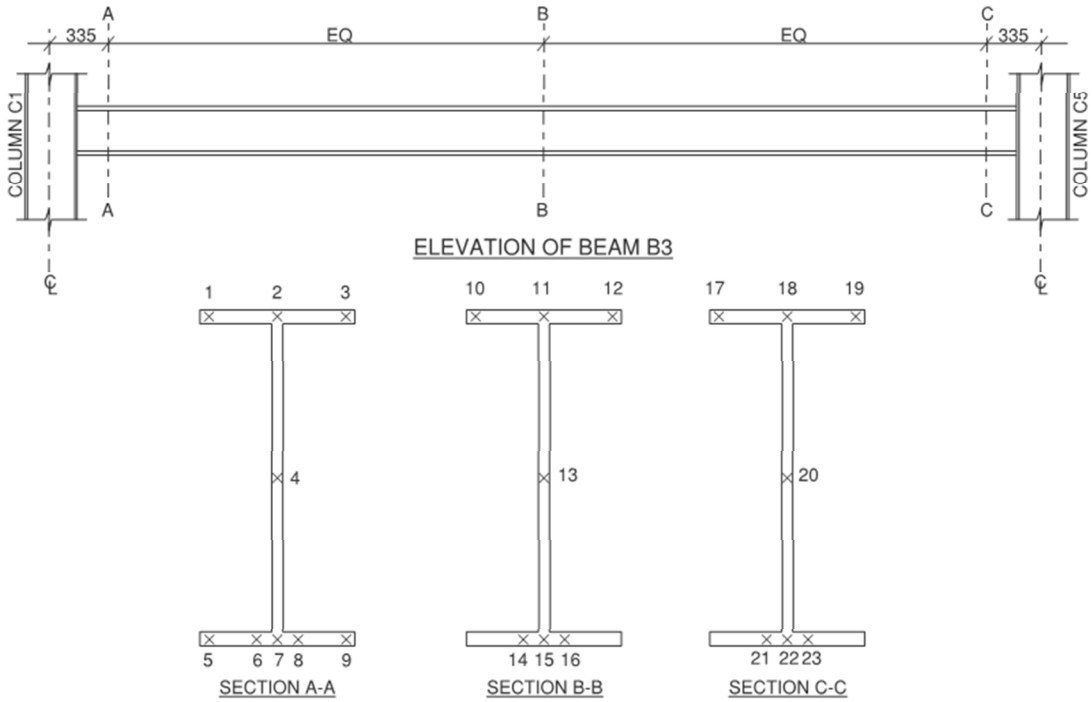

Figure 4.28: Representative thermocouple locations for steel beams; adapted from Lennon (1999)

For the thermal analysis, structural beam elements were modelled as two-dimensional cross-sections in SAFIR as validated by Section 4.4.1, with the measured time-temperature plots applied as heat fluxes at the section boundary. The quantity of input files that had to be created was quite large because of the different recorded time-temperature results, the different beam cross-sections, and the different configurations of beam and slab. The creation of the input files was streamlined using parametric modelling software Grasshopper to model the cross-sections and generate input for the

SAFIR thermal analyses. An example of a 2-D cross section for the thermal analysis is shown in Figure 4.29.

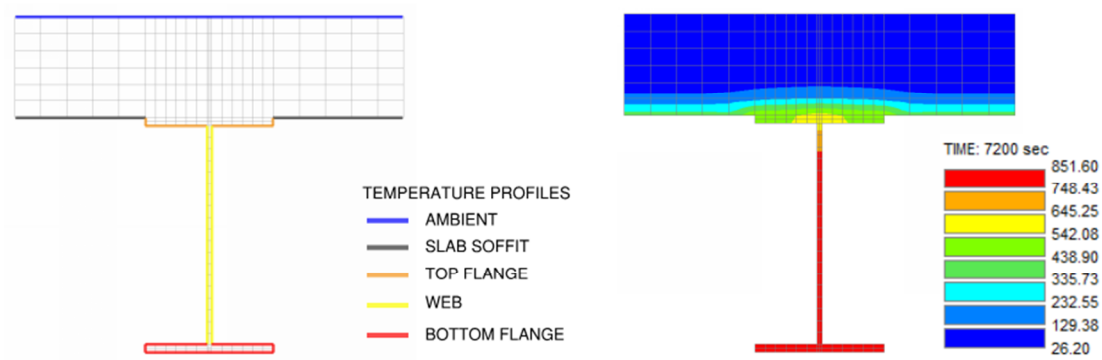

Figure 4.29: Sample thermal input for SAFIR (left) and resulting temperatures at 120 minutes (right)

Similar to the beams, the slab also required a thermal analysis to be performed prior to the structural analysis. The Cardington corner test had thermocouples installed at four locations along the soffit of slab, with each location having six thermocouples installed at the underside of the steel deck as well as within the concrete slab itself. To simplify the thermal input, the average of the two surface thermocouples was used for each location as a temperature input in the thermal model. The thermocouples within the slab were neglected as input but were used to verify the results of the thermal analysis.

Lastly, a thermal analysis was done on the columns of the corner test. Although fire protected, the columns still experience a temperature increase and this must be accounted for in the analysis. Thermocouples were installed at the top, middle, and bottom of the columns. Similar to the beams, each instrumented location had a series of thermocouples installed on the flanges and web, although for the columns it was only 5 thermocouples. Again, the results of the flange thermocouples were averaged to simplify the input.

4.5.3 Structural Analysis

The full three-dimensional floor model for the structural analysis was written for SAFIR manually with a spreadsheet and text editor. The reason for this was to correctly identify the master-slave relationships between the steel beam elements and the concrete shell elements of the floor. There was assumed to be a full shear connection between the steel beams and concrete slab. In contemporary Canadian structural design it is common to provide steel beams with 40% - 60% partial shear connection since the long spans in office buildings are generally deflection governed and the load carrying capacity can be optimized. At the time of writing, SAFIR was unable to account for partial composite action in the model with springs as verified by the developers through email. As well, there is a lack of experimental data for full-scale composite slabs with partial shear connection between the steel and concrete to validate such a model.

All shell elements contained six degrees of freedom, while all beam elements contained seven degrees of freedom. The floor was modelled to the centerlines of the adjacent ambient bays, with boundary conditions applied at the centerlines to generate continuity in the ambient bays.

In terms of material properties, the structural steel was specified as STEELEC3, with an ambient elastic modulus of 210 GPa and yield strength in accordance with Table 4.2. Reinforcing steel was specified as STEELEC2 with an elastic modulus of 210 GPa and yield strength of 400 MPa. The concrete slab was modelled as SILCOETC2D with a compressive strength of 35 MPa and a tensile strength of 1.7 MPa.

The concrete material model, SILCOETC2D, incorporates damage in the concrete and transient creep strain. Damage in the concrete is calculated assuming that the tensile

damage (κ_t) and compressive damage (κ_c) are functions of the accumulated plastic strains in the concrete (Gernay et al., 2013). The model parameter $\tilde{d}_c$ is defined by the user to relate total strain to inelastic strain, and hence damage, and is recommended to be taken as 0.3 (Gernay et al., 2013). At elevated temperature, the model must also account for free thermal strain and transient creep strain. Thermal strain follows the Eurocode uniaxial relationship and is a function of temperature and concrete aggregate (CEN, 1992-2005). Transient creep strain is included explicitly and is adapted from the Explicit Transient Creep Eurocode model, developed at the University of Liege by Gernay and Franssen (2012). Temperature dependent mechanical properties of the concrete model are also the same as those in the Eurocode (CEN, 1992-2005). Damage properties such as crack energy, dilatency, and compressive damage at peak stress are assumed constant with temperature in lieu of more experimental data to suggest otherwise (Gernay et al., 2013). Table 4.3 below summarizes the material properties used in the model for concrete.

Table 4.3: Material parameters included in the concrete model

Parameter	Description	Value	Units
v	Poisson ratio	0.2	
f'_c	Compressive strength	$30x10^6$	N/m^2
f_t	Tensile strength	$3x10^6$	N/m^2
	Strain at peak stress	0.0025	
α_g	Dilitancy parameter	0.3	
x_c	Compressive ductility	0.19	
$\tilde{d}_c$	Compressive damage at peak stress	0.3	
g_t	Tensile ductility parameter	400	N/m^2

The applied loading applied during the test was 2.63 kN/m^2, while the self-weight of the floor itself was 2.31 kN/m^2. A total of 4.94 kN/m^2 was applied as a uniform load in SAFIR, with the exact breakdown of loading shown in Table 4.4. The loads included in

the design and testing of the Cardington corner test are in-line with contemporary structural design in Canada, although exact magnitudes may differ on a case-by-case basis. Load combinations were not used for the application of applied load since these are the actual loads applied to the floor during the test as reported by British Steel (1999). In a design scenario, load combinations would need to be used as discussed in Section 2.8.

Table 4.4: Applied loading for Cardington Corner Test

Load	Value (kN/m^2)
Self-Weight	
Composite Slab	2.06
Steel framing	0.25
Superimposed Load	
Raised Floor	0.4
Services	0.25
Ceiling	0.15
Partitions	1.0
Imposed (1/3 of design load)	0.83

The structural analysis was run with the results of the thermal analysis from Section 4.5.2 used as input. The deflected shape of the composite floor under the fire scenario is shown in Figure 4.30.

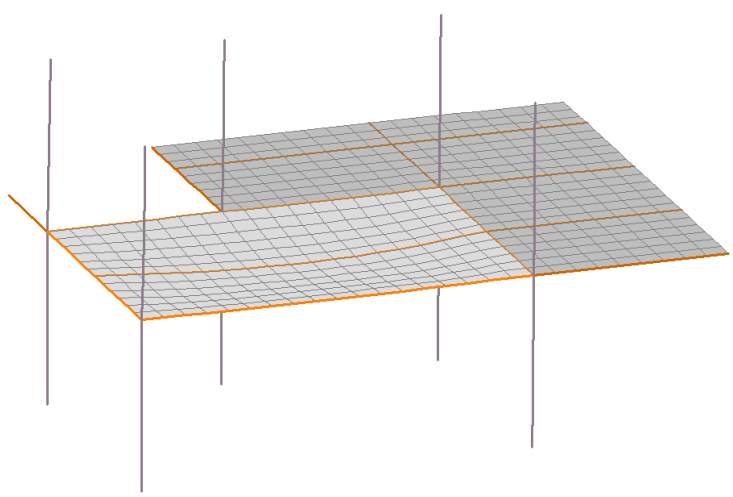

Figure 4.30: Deflected shape of Cardington Corner Test

As well, looking at the membrane forces in the model shown in Figure 4.31, it can be seen that tensile membrane action (TMA) was developed. The blue vectors denote

compressive forces in the concrete slab, while red is tension. The stress distribution is in agreement with the TMA description provided in Section 2.8.4.

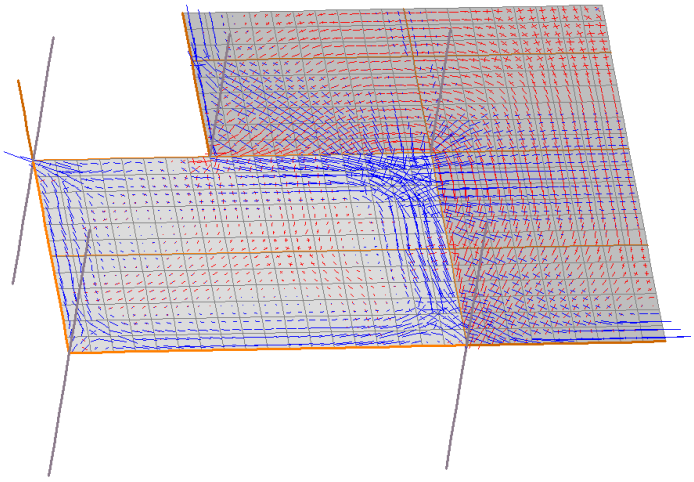

Figure 4.31: Membrane forces developed within composite slab of Cardington Corner Test

The floor had strain gauges installed throughout; however the recorded strains are not used as a basis for comparison due to the difficulty of interpreting the results. Gillie et al. (2001) refer to the difficulty of matching experimental strains to modelling strains in full-scale tests because of mechanisms not captured in the finite element model, such as concrete cracking, ductility in connections, and local or global buckling relieving strains. Instead, floor deflections are typically used to validate whole floor models as seen in the literature (Wang et al., 2013; Gillie, 2009). The deflection results of the SAFIR model are shown in Figure 4.32 for the midpoint of the slab. It can be seen that the agreement is generally good, particularly in the trends, maximum deflection, and maximum slopes. The maximum deflection and the maximum rate of deflection are important to match in the validation process since the Eurocode failure criteria for steel beams is expressed as an absolute deflection as well as a rate of deflection. Similar acceptance criteria are discussed in Section 5.6 for the design scenario although the exact magnitudes differ. The

temperatures within the compartment and measured throughout the structure were not homogenous; the simplification of which by the author may account for disagreement between the recorded and calculated deflections in Figure 4.32. The difficulty of modelling Cardington for this reason is also cited by Gillie (2009).

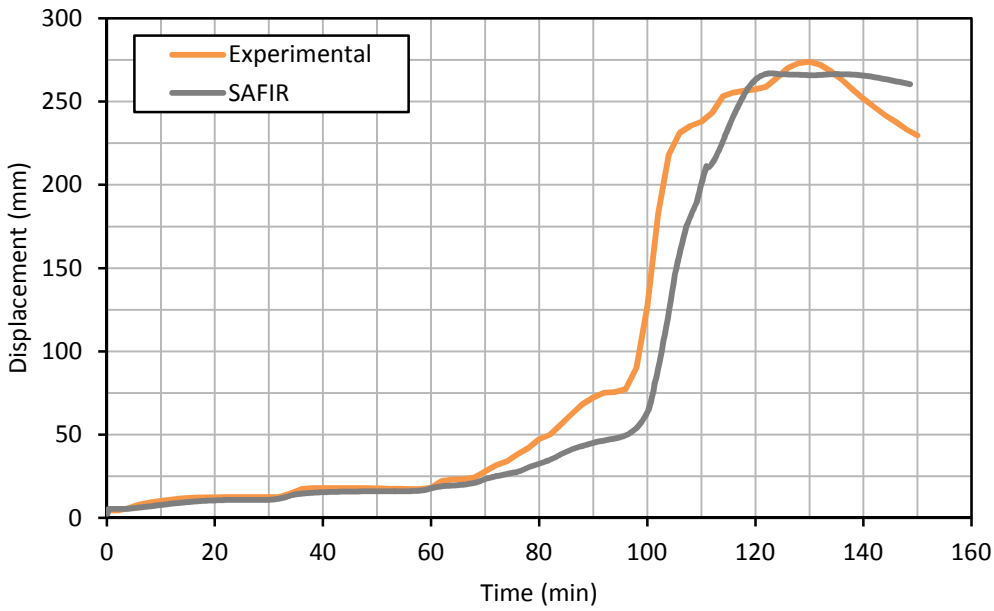

Figure 4.32: Cardington Corner Test deflection response

For the basis of comparing the SAFIR model to the experimental data, the beam and slab forces do not need to be considered in detail. It was observed in the analytical model, however, that beam B3 and B4 (refer to Figure 4.27) experienced very high axial compressive forces. Although it could not be captured in this model with the simple beam elements, this matches the experimental observations where the girders framing into the column experienced local flange and web buckling (Bisby et al., 2013).

4.5.4 Discussion on Composite Floor Modelling

This benchmark example demonstrated the capability of the software and the user to accurately model a portion of a composite floor with appropriate boundary conditions and match the results to experimental data. It should be noted however that several key

differences exist between the composite floor modelled here and what is typically seen in contemporary Canadian construction. First, the mesh used for Cardington was A142, which is 6 mm diameter smooth bars at 200 mm c/c, while Canadian construction typically specifies wire mesh which may be ribbed. A typical reinforcement for a 141 mm composite slab in Canada is 152x152 MW18.7xMW18.7, which are 4.88 mm diameter smooth bars at 152 mm c/c. Although the Cardington reinforcing mat and that used in Canadian construction give similar steel area per unit width, the difference that smooth bars and ribbed bars may have on the overall behaviour of the system is not well understood and needs to be researched further. TMA is highly dependent on the forces and strains that develop at the boundary of the slab, and the Cardington test observations showed compartmentalization failure where the reinforcing mat was missed in a slab. The difference between smooth and ribbed bars can also not be captured in SAFIR since the rebar layer is smeared and perfect connection is assumed between the concrete elements above and below this. The literature suggests that strains may be different between smooth and ribbed reinforcement since the distribution of strain will be different in the slab (Giroldo & Bailey, 2008).

Further, the Cardington test series used lightweight concrete for its construction, while contemporary Canadian construction uses normal density. Lightweight concrete would give a lower slab temperature for the duration of the fire since it has a lower density, however heat conduction through normal weight concrete is well understood and this can be accounted for during the thermal analysis. It is not expected that the mechanism of TMA is impacted by switching between lightweight and normal weight concrete, although the observed deflections may vary due to a difference in mechanical

properties between the two concretes. The literature does not mention this difference in concrete type between the Cardington test series and concrete that is used in contemporary construction, particularly Canadian construction, and is indeed a research need to understand if normal weight concrete would impact TMA.

Lastly, the question must be asked of what mechanisms are important to capture in the model for design purposes. The Cardington test series showed that globally the floor remained intact and was able to support the load with alternative load paths, even with some local failures observed such as beam flange buckling in Test 1, 3, 5, 7, local column buckling in Test 2, and connection failure in Test 1, 2, 5 (Bisby et al., 2013). The model outlined in this section only had deflections considered for the basis of comparing the model with the experimental results. In a design situation, it will also need to be determined if the connections are adequate and if compartmentalization is maintained. There does not appear to be consensus across the industry on what should be the acceptance criteria used for these vital aspects of the fire design.

In terms of the connections, there are published case studies of buildings that have been constructed where the connections are not quantitatively assessed since similar connections have been tested and shown to be ductile in fire, even though the slab is designed with tensile membrane action and does not explicitly account for that connection ductility (Wang et al., 2013). Other published case studies of similar buildings have also designed the slab with TMA but then extracted the connections and the forces from the global model to develop a sub-model of the connection model and demonstrate the capacity was acceptable (Lamont et al., 2006). This highlights just how important industry consensus is for structural fire engineering, in particular the acceptance criteria,

and is why the authors role on the acceptance criteria task group of the ASCE Fire Protection Committee is complementary to this thesis.

4.6 Summary

This chapter has demonstrated benchmark modelling to develop the competency necessary to model structures for the fire limit state. This modelling transparently increased in complexity from a simply supported steel beam up to a whole floor analysis from the Cardington test series. Chapter 5 will implement this complex modelling to develop an alternative solution for a real Canadian building and apply the draft chapter of Acceptance Criteria that has been proposed for the ASCE Guideline for Structural Fire Engineering and forthcoming ballot. This novel case study will develop design fires including the recently published improved travelling fire methodology, assess the contemporary Canadian building for its performance in fire, quantify the benefits of PBFD, and assess knowledge gaps in the Canadian literature for applying PBFD.

5 Chapter: Alternative Structural Design Solution

This chapter will propose a novel alternative solution for a contemporary composite steel building that has been designed in accordance with Canadian codes and best practices. The baseline solution follows the prescriptive solution of specifying fire-resistance rated assemblies for the structural elements. The alternative solution developed will use performance-based fire design (PBFD) to incorporate whole-floor response where the floor configuration allows it and experimental data exists to support the modelling. This is the first time to the author's knowledge that a structural fire analysis of such complexity will be applied to an actual Canadian building to assess the actual structural performance. Section 3.6 demonstrated a lack of published case studies in Canada, while the ones that are published generally discuss a simplified, idealized structural layout. Based on this alternative solution developed, the structural fire design can be assessed to determine if there is potential in Canadian building construction to employ the methods and what benefits may be realized. An analysis of the financial considerations for the design and the impact on the robustness of the building and the resilience of the business it houses will be provided.

5.1 Administrative Procedure

The process to be followed for PBFD in Canada can be adapted from the SFPE Engineering Guide to Performance-Based Fire Protection (SFPE, 2000) as demonstrated by Bartlett (2005). This is schematically shown in Figure 3.10. The first few steps of the process involve defining the scope, as well as the stakeholder and design objectives.

As well, the performance criteria need to be agreed upon ahead of time. For this design, acceptance criteria that have been proposed to the ASCE Fire Protection

Committee for adoption into the ASCE/SEI Guideline for Structural Fire Engineering will be adopted. The balloting process for that chapter has not been undertaken yet so the author cannot confirm these acceptance criteria have industry consensus yet. In fact, as demonstrated in Section 3.4, there is a lack of industry consensus on what to use for acceptance criteria which is why such a document as the ASCE/SEI Guideline for Structural Fire Engineering is in demand. Therefore the criterion demonstrated herein will represent the starting point proposal for this ballot.

The initial steps of implementing a PBFD are extremely important to ensure that all stakeholders agree on the methods and criteria to be used in the design. This will help to mitigate the risk associated with performance-based designs that the authority having jurisdiction (AHJ) may not accept the final design. In developing the rest of the alternative solution, it is assumed that the stakeholders have agreed on the performance goals, acceptance criteria, and methods of analysis to be used.

5.2 Fire Protection Strategy

The candidate structure proposed herein is a five storey mixed use building, with retail on the ground floor, and office space on floors two to five. The ground floor and below are reinforced concrete, and composite-steel construction at level two and above. Lateral forces are resisted by moment frame above grade and shear walls below.

The floor is specified to have a one hour fire-resistance rating. The prescriptive design for the building is adapted from the architectural documents of the building since it is a real Canadian structure. It has been designed to satisfy all requirements of the Ontario Building Code, 2012 (MMAH, 2012). Detailed development of the prescriptive

design of the building and the ambient structural design are beyond the scope of this thesis.

The assembly used for this is F906 (see Figure 3.8), which specifies a 13 mm minimum thickness of insulation on the steel beam. This can be adjusted using Eq. 5.1 below from the BXUVC guideline (ULC, 2016) since the 13 mm is based on a W200x42 steel beam, with the results shown in Table 5.1.

$$T_2 = T_1 \, x \, \frac{(M/D)_1 + 38.2}{(M/D)_2 + 38.2} \tag{5.1}$$

where:

T = thickness (mm) of spray-applied material

M = Mass of steel beam (kg/m)

D = heated perimeter of steel section in metres (m)

In the above equation, subscript 1 refers to the given beam size specified in the listing, while subscript 2 refers to the desired beam size. As well, the range of validity for this equation is M/D ≥ 23 and T ≥ 10 mm.

Table 5.1: Adjustments to thickness of spray-applied fire protection

Section	Mass (kg/m)	M/D Ratio	Spray Thickness (mm)
W200x42	42	47.6	13.0
W610x217	217	100	10.0
W610x174	174	81.0	10.0
W610x155	155	72.3	10.1
W610x140	140	75.3	10.0
W460x68	68	51.1	12.5
W310x52	52	47.5	13.0
W310x33	33	36.3	15.0

The F906 assembly will be applied throughout the floor plate. For vertical fire resistance ratings, the façade has a one hour fire-resistance rating and a concrete block wall with a two hour fire-resistance rating exists at the north edge of the floor to separate

the mechanical space, stairwell, and elevator from the main office occupancy as shown in Figure 5.1.

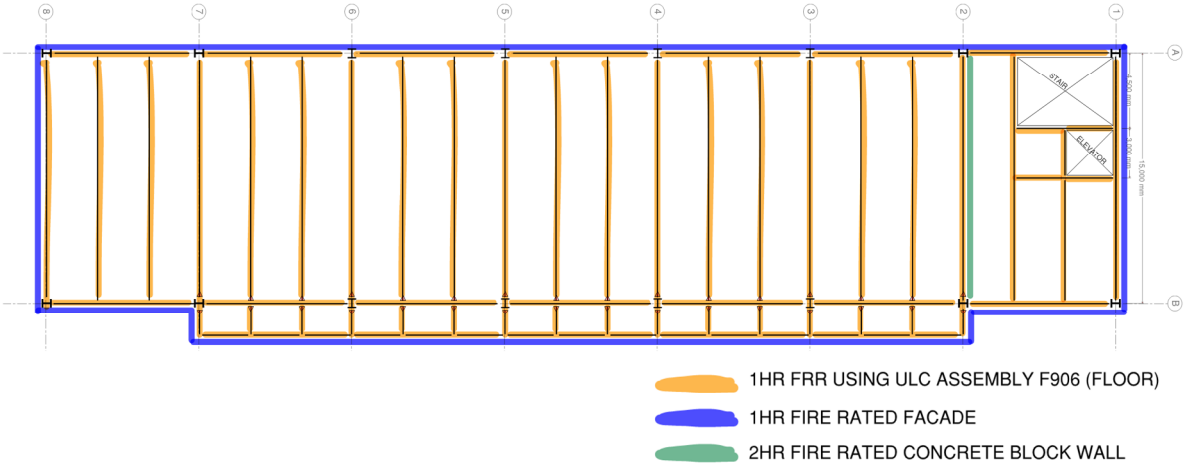

Figure 5.1: Prescriptive and default fire protection strategy for case study building

The portion of the floor shown in Figure 5.1 is symmetrical with the other half of the building, both of which are connected with bridges that are pin connected and allow for differential movement between the halves of the building. The lateral system, not shown, consists of moment frames. Along the length of the floor, there is only one moment frame at the far end near the openings to avoid accumulation of thermal expansion forces between two stiff lateral elements.

5.3 Development of an Alternative Solution

The proposed alternative solution will implement international best-practice to demonstrate and quantify the benefits of PBFD in Canadian construction. The general layout of the floor was first assessed qualitatively using lessons learned from consultancy experience with performance-based fire engineering in the United Kingdom for composite steel construction (Flint et al., 2013). This identified where bays could be assessed with TMA and where alternative load paths were required. As well, the structural fire protection was optimized by removing the fire protection on secondary

beams where TMA would allow. In Canada this approach has not rationally been considered as discussed in the literature review of Section 3.6, and therefore represents a novel opportunity for practitioners to consider if it can be done transparently and competently using available Canadian literature.

The floor plan for the alternative solution is shown in Figure 5.2. The fire protection was removed from the secondary steel beams of the interior bays, however the cantilevered floor beams and end bays are still protected with spray-applied fire resistive material using assembly F906, similar to the prescriptive solution.

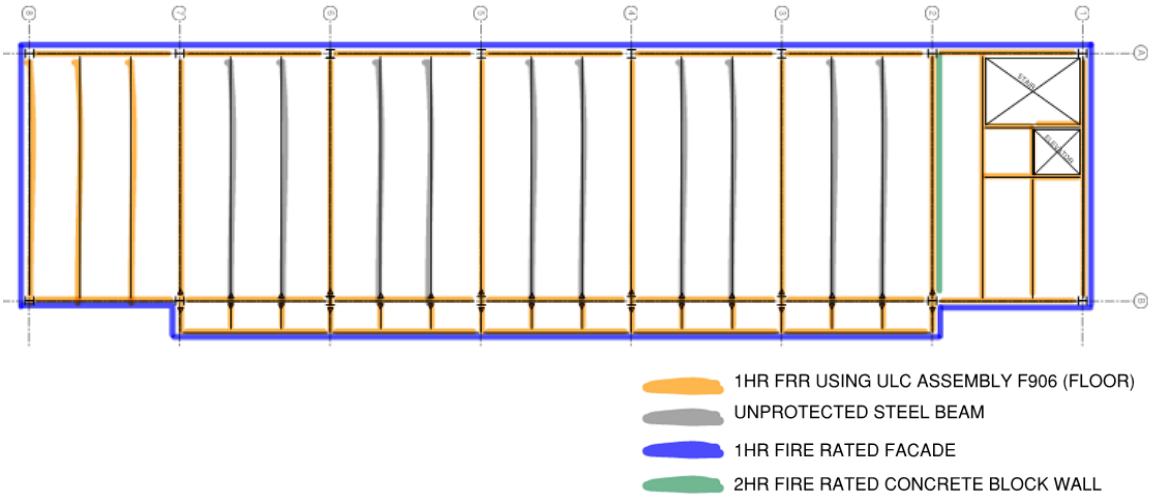

Figure 5.2: Alternative solution fire protection strategy for case study building

The openings and framing configuration of the end bay with the mechanical room was not conducive to TMA, which is the load carrying mechanism used to justify removal of the secondary fire protection. As well, the framing of the cantilever portion of the floor was adjusted for the alternative solution as shown in Figure 5.3. The reason for this is that with the secondary steel beams not fire protected, the back span of the interior cantilevers is assumed to have no capacity at the fire limit state. The edge beam was

made continuous in the alternative solution so that an alternative load path is provided at the fire limit state.

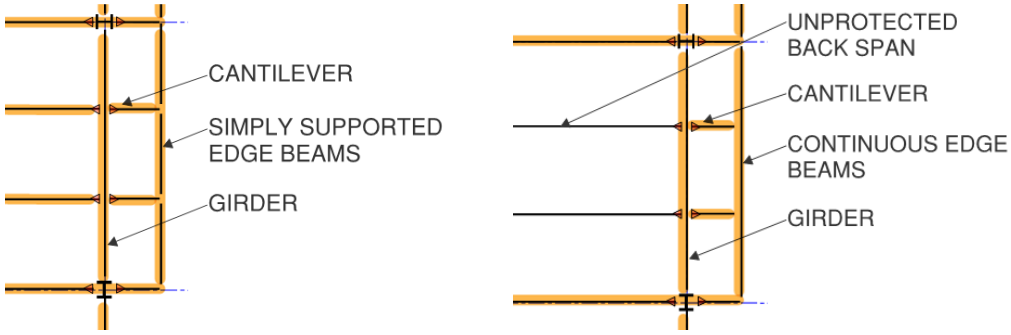

Figure 5.3: Cantilever framing revision between prescriptive (left) and alternative (right)

5.4 Material Properties

The material properties used for the alternative solutions follow the Eurocode formulations for elevated temperature effects. This applies to both the thermal expansion of the structure and the degradation effects of high temperature. The material properties are provided for steel in Section 2.7.1 and concrete in Section 2.7.2. Material properties during the structural analysis are evaluated at every time step based on the temperature of the beam or shell element as outlined in Section 4.1.

For the ambient design of the structure, structural steel conformed to CAN/CSA-G40.20 Grade 350W (min yield strength 350 MPa) while the steel deck conformed to ASTM A653M Grade A (min yield strength 270 MPa) and the welded stud shear connectors were ASTM A108 Grade 1010 (min yield strength 400 MPa). The welded wire fabric was CSA G30 series (min yield strength 386 MPa) and the concrete itself conformed to CSA-A23.1 with a minimum compressive strength f'c of 30 MPa.

5.5 Design Fires

The design fires comprise a range of realistic, worst-case, structurally significant fires that can occur in the compartment. They range in size from a compartment fire

isolated to a 9 m x 15 m bay, up to a travelling fire which heats the entire length of the floor (63 m) over a period of time. This large open office space lends itself to the TFM methodology (Stern-Gottfried & Rein, 2012b). All design fires considered have precedent since peer-reviewed case studies have used them in their design. As well, the standard fire is considered as a design fire simply because it is familiar to the governing bodies likely to review the structural fire design in Canada. Having the standard fire as a design fire amongst the other realistic fire scenarios will also provide a point of reference in terms of quantifying and relating the performance of the alternative solution to the baseline. Over time, it is expected that the AHJ will become more familiar with the process of defining design fires for a building and the standard fire could be omitted from consideration and remain as simply a comparative tool for fire-rated assemblies used with the prescriptive approach to fire safety engineering.

5.5.1 Fuel Load

The fuel load of the compartment is a key variable to determine the time-temperature curves that the structure will be subjected to. The Eurocode approach based on occupancy classes will be used to calculate the fuel load, which is referenced in the SFPE Handbook of Fire Protection Engineering (SFPE, 2016). The equations for the method itself are provided in the Eurocode (CEN, 1991-2002). Refer to Appendix A for the detailed calculations of the fuel load.

Based on an office occupancy, it was found that the fuel load to be considered for the design is 642 MJ/m^2. This is a similar magnitude to previous case studies which have assessed open-plan, contemporary office spaces (Stern-Gottfried & Rein, 2012b).

5.5.2 Standard Fire

The standard fire time-temperature curve is taken from CAN/ULC-S101. The standard provides the time-temperature curve both in a tabular format as well as an equation (ULC, 2014) as shown in Eq.5.2.

$$T_g = 20 + 750(1 - e^{-0.49t}) + 20\text{t} \tag{5.2}$$

where

T_g= furnace temperature, in degrees C

t = time, in minutes

The resulting time-temperature curve is shown in Figure 5.4.

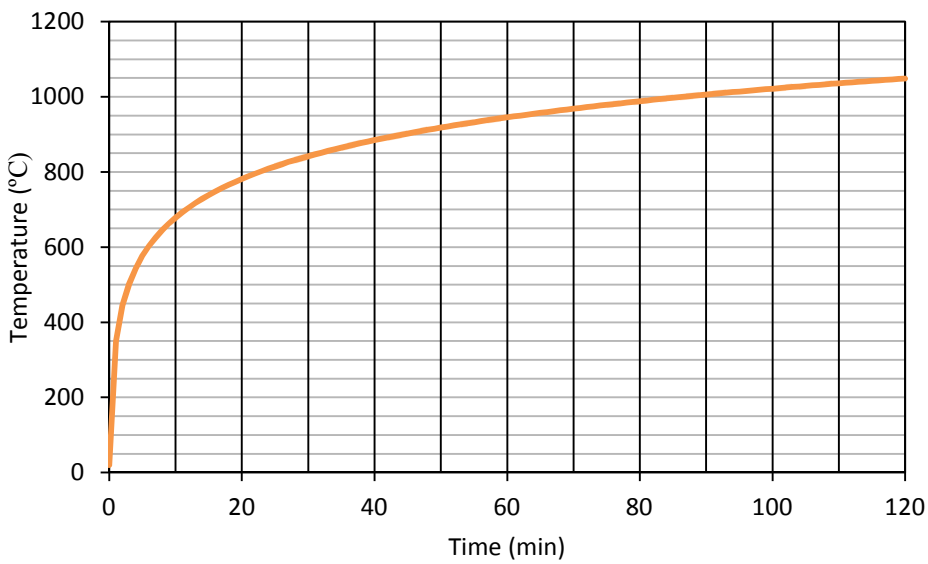

Figure 5.4: CAN/ULC-S101 standard fire curve

The standard fire was assumed to act throughout the office occupancy of the compartment. The end bay with mechanical space, the stair core, and the elevator shaft were assumed to be unheated because of the fire rated separation located at the gridline. The heated portion of the floor is shown in Figure 5.5 below, and is denoted throughout DF_S (design fire, standard) in the proceeding results and discussion sections.

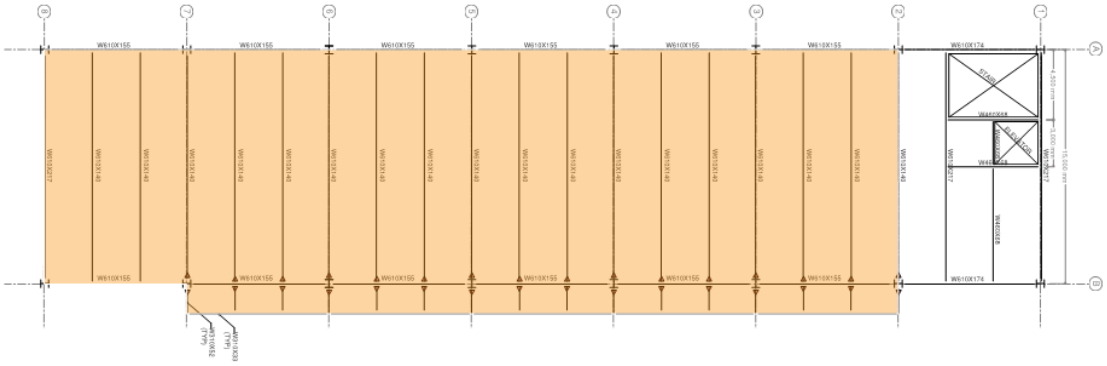

Figure 5.5: Portion of floor heated by standard fire curve

5.5.3 Eurocode Parametric Fire

The architectural plans for the building show potential compartmentalization of the space, roughly aligning with grids. This results in a typical 9 m x 15 m compartment. These spaces are not formally compartmentalized with fire rated partitions. Assuming these partitions do provide some compartmentalization in the early stages of the fire as hot gases fill the space, the author deems it conservative to assume that the compartments contain the fire until the fuel is consumed (burn-out). This ensures flashover is reached in the compartment and a severe fire environment is maintained on the structure.

The Eurocode contains an appendix with the formulation of parametric time-temperature curves. The detailed calculation of the time-temperature curves for the case study compartment are shown in Appendix B, however the highlights of the calculations will be discussed here. First, the fire temperatures and duration are dependent on the ventilation of the compartment. Referring to the architectural elevations of the building, it was assumed that this can range from 40% of the wall area to 100% of the area of a single wall. An area of 20% was also tested, but found unrealistic since the resulting compartment temperatures would begin to shatter the windows. Hence, 100% and 40% ventilation of a single wall are taken as the upper and lower limits of the ventilation

parameter. As well, the fuel load is taken from Sections 5.5.1. The series of parametric time-temperature curves considered as design fires is shown in Figure 5.6 below.

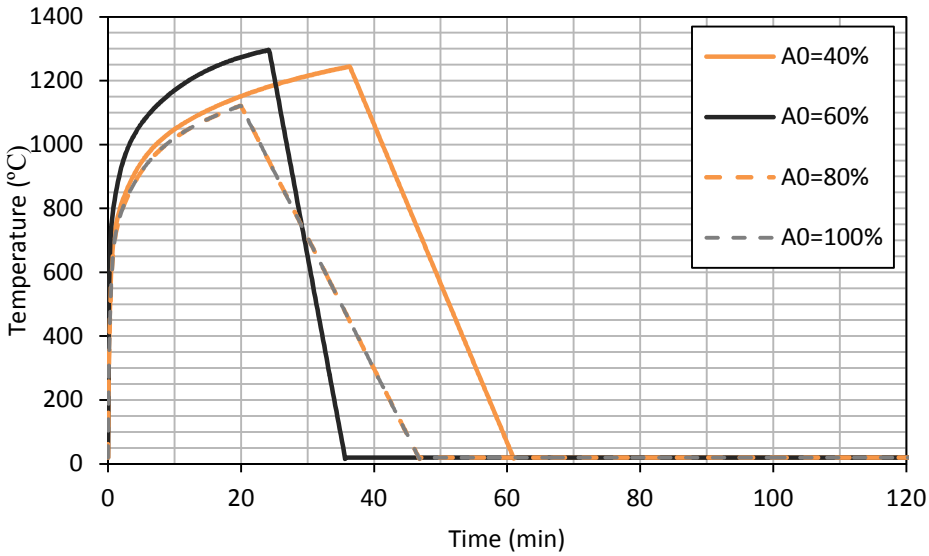

Figure 5.6: Parametric time-temperature curves

The parametric fire was considered to act in two locations as shown in Figure 5.7, and is referred to as DF_P1 (left case) and DF_P2 (right case) in the proceeding results and discussion.

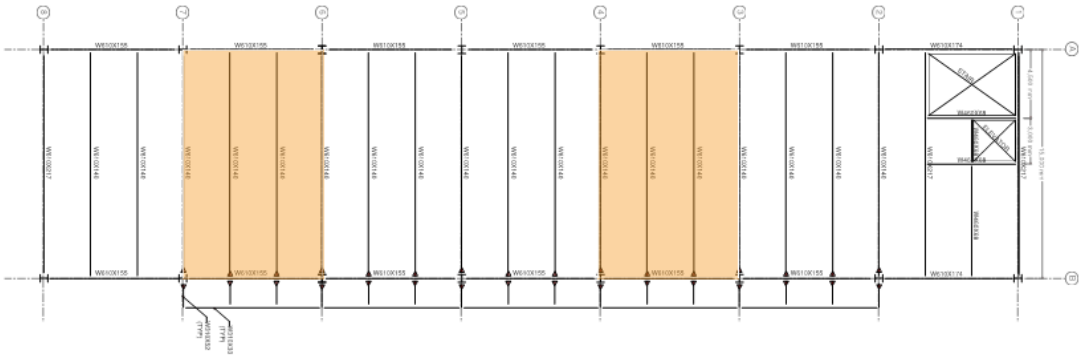

Figure 5.7: Portion of floor heated by parametric fires

5.5.4 Travelling Fire

As discussed in Section 5.5.3, the office space has non-fire rated partitions that could potentially compartmentalize offices. There is also the possibility that the partitions do not contain a fire, or are potentially removed by the tenant at a later date. For that

reason, another design fire scenario considered is a travelling fire that moves from one end of the floor to the other. The latest formulation of the travelling fire framework will be used for these calculations and is called the iTFM, or improved travelling fire methodology (Rackauskaite et al., 2015). The iTFM builds on the foundational aspects of the TFM described by Stern-Gottfried and Rein (2012a; 2012b). The detailed calculations for the time-temperature curves are provided in Appendix C. This is the first published case study of a realistic composite steel structure being analyzed using the iTFM to the knowledge of the author.

In the iTFM, the user selects a fire size which is given as a ratio of the floor area. The literature states that a fire size of 5% to 15% of the floor area is generally the most severe structurally (Stern-Gottfried & Rein, 2012b). As well, the iTFM requires the user select a "flapping angle", a parameter in the calculation of the near-field temperatures of the fire. The effect of flapping angle on temperature is shown in Figure 5.8.

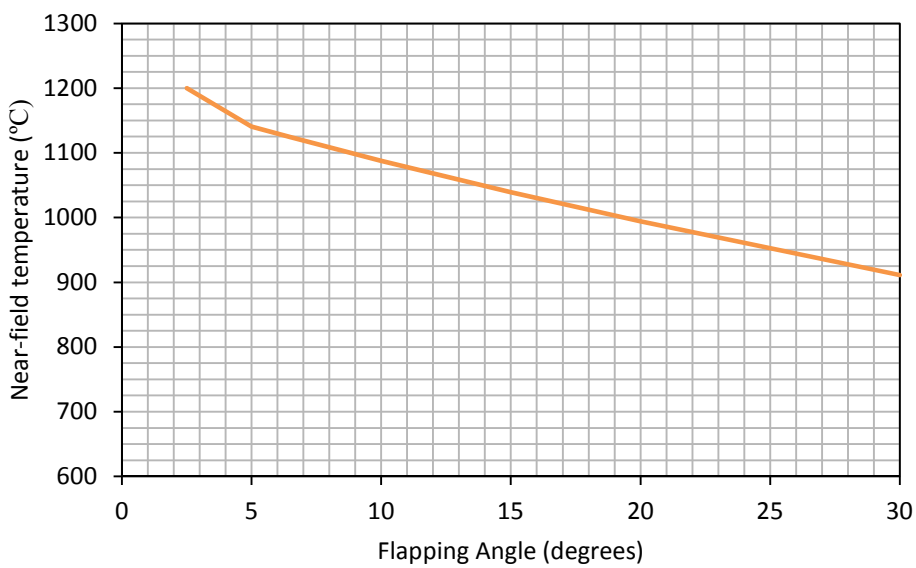

Figure 5.8: Effect of flapping angle on near-field temperature, iTFM

Figure 5.8 shows that the higher the flapping angle, the lower the predicted near-field temperatures since the fire plume is presumably spread over a larger area. The recommended value of the flapping angle is given as 6.5º based on experimental observation (Rackauskaite et al., 2015). For a fire size of 10% with a fuel load of 642 MJ/m^2, this gives a maximum near-field temperature of 1123ºC. The resulting time-temperature curves along the length of the floor (divided into 1.5 m segments) are shown in Figure 5.9. The figure also contains two specific locations (x=15 m and x=45 m) highlighted to show the general trend of each time-temperature curve.

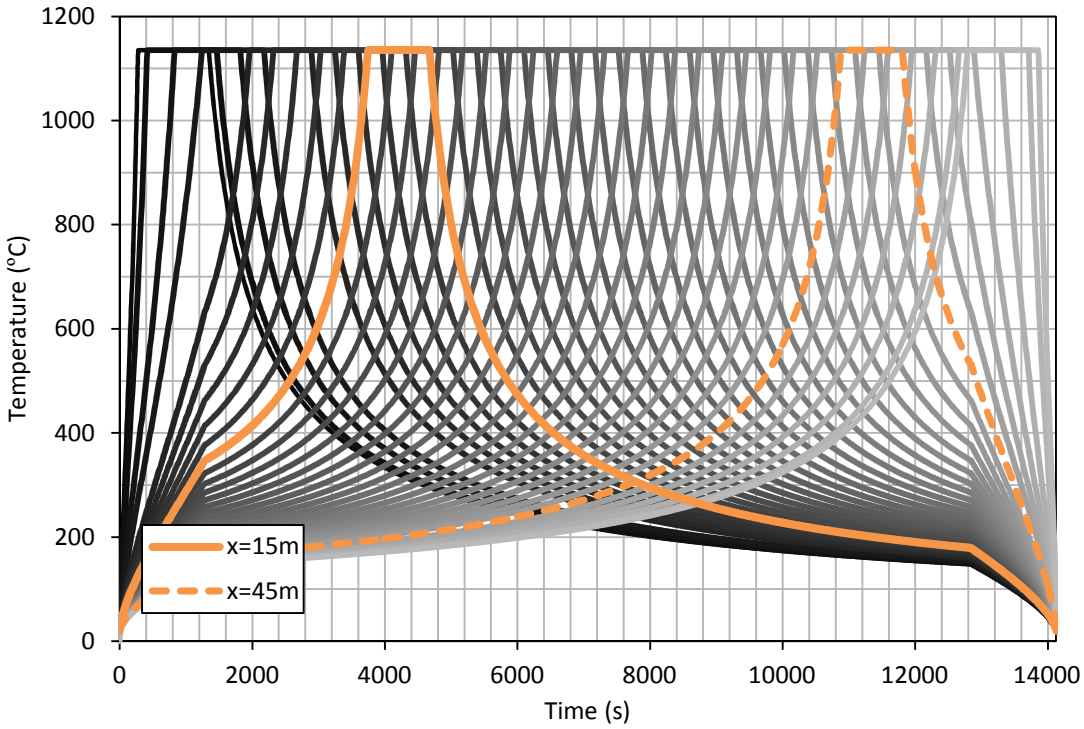

Figure 5.9: Time-temperature curves along length of floor due to travelling fire

The origin and path of the travelling fire is up to the user to decide, however in general the most severe fires start at one end and travel linearly. The last bay in this scenario receives the longest duration of pre-heating while it is heated by the far-field temperatures of the travelling fire, until eventually the near-field arrives (Stern-Gottfried

& Rein, 2012b). This point can be illustrated when comparing the time-temperature curve at 15 m to that at 45 m in Figure 5.9 above. The path selected for the travelling fire was from the left to the right, as seen in Figure 5.10. The unprotected steel beams and slab at the far end of the floor thus receive preheating as the fire travels through the first five bays, while if the fire travelled the other way the first unprotected bay would only receive preheating during the time it takes to travel four bays.

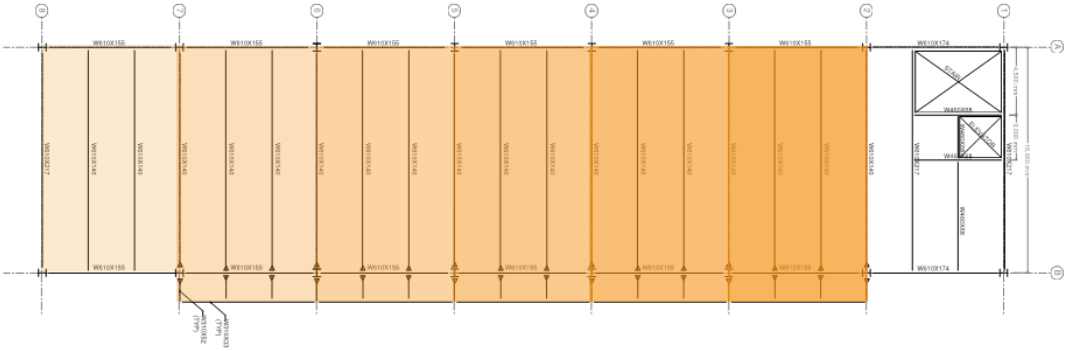

Figure 5.10: Travelling fire heated areas, and path (light to dark)

5.6 Acceptance Criteria

Current fire safety design in Canada generally follows the prescriptive approach set forth by the NBCC, which itself references the CAN/ULC S101 test standard for fire rated assemblies. The fire resistance of structural assemblies in that test subject to acceptance criteria in three categories: stability, integrity, and insulation (ULC, 2014). The acceptance criteria for the PBFD will be categorized similarly for easier comparison with current fire protection terminology.

5.6.1 Stability

The stability criterion refers to the ability of the structure to support itself without local or global collapse during and after the fire event. In the prescriptive approach, this is maintained by ensuring temperature limits in the steel. More broadly, CAN/ULC-S101 states “the test specimen shall have sustained the applied load throughout the fire

endurance period" (ULC, 2014). As well, the functional statement related to structural stability in the NBCC is "to retard failure or collapse due to the effects of fire" (MMAH, 2012). Also, to maintain stability and ensure local collapses do not occur, the connections of the steel beams shall have sufficient ductility provided to prevent failure. In the literature, it is suggested that the idealization of a rigid connection with no ductility to dissipate thermal forces is overestimating the connection forces likely to develop, hence the rationalization that providing ductility for the connections is adequate (Wang et al., 2013). The acceptance criteria for the stability criterion are:

1. Deflection of floor not to exceed total of $(L_c^2)/400d$, where L_c is the clear span of the floor and d is the depth of the steel beam being considered;
2. Rate of deflection of floor not to exceed $(L_c^2)/9000d$ per minute; and
3. Demonstrate sufficient ductility in the connections, or design connections to resist the forces developed

The deflection criteria (1) and (2) in the above list are adapted from both Canadian (ULC, 2014) and American (ASTM, 2016) standard fire testing as indications of failure. These criteria were first proposed in 1959 as part of ASTM standard fire testing and were a simplification of failure for testing in lieu of a detailed analysis considering end restraint, loading, and construction (Ryan & Robertson, 1959). The temperature criteria used in standard fire testing is not used as acceptance criteria in PBFD since it has been shown that not all steel assemblies fail at those temperatures as implied by the standard (Bono, 1962). The deflection criteria, however, did predict failure in 43 out of 50 tests (Bono, 1962) which could suggest why international practice is still to consider deflection of the floor as acceptance criteria (Wang et al., 2013). Acceptance

criteria (3) is based on testing at the University of Sheffield which demonstrated the forces due to thermal expansion in a model with rigid connections do not actually develop if ductility is provided in the real connection (Yu et al., 2009). Meanwhile, some international case studies actually develop finite element models of the connections to demonstrate failure is not reached for the expected forces and movements (Lamont et al., 2006). Crucially, in the Canadian case studies from Section 3.6, connections were not considered. Connections will need to see increased consideration moving forward as part of PBFD in Canada based on the trends that were shown in international practice.

5.6.2 Integrity

Limited information is available on predicting the integrity of composite floor slabs. The test standard CAN/ULC-S101 meets this requirement qualitatively by requiring that no hot gases or flames are seen penetrating through the slab (ULC, 2014). To ensure that the integrity of a composite floor slab is maintained, it is suggested that the deflection limits from stability criterion be used (Section 5.6.1) since this directly relates to the curvature of the floor (Wang et al., 2013). Additional reinforcing mesh in the concrete slab to develop tensile membrane action will also ensure integrity is maintained. In the Cardington test series, the only integrity failure through the slab was observed when an additional layer of reinforcing mesh was missed in the construction (Bisby et al., 2013).

5.6.3 Insulation

The insulation requirement refers to the temperatures on the unexposed sides of structural elements. This ensures combustibles in an adjacent compartment do not ignite. Since the design fires are deviating from the standard fire, it is not immediately clear if

the insulation will be maintained on the unexposed face just because the prescriptive requirements are met. The thermal analysis of the slab elements will determine if the temperature limits are met. As stated in CAN/ULC-S101 (ULC, 2014):

1. Average temperature increase on unexposed surface shall not exceed 140ºC; and
2. Maximum temperature increase on unexposed surface shall not exceed 180ºC

The first limit will govern because of the uniformity of the thermal analysis.

5.7 Structural Fire Analysis

5.7.1 Steel Beams

Steel beams were modelled using the beam element in SAFIR, defined by two end nodes with seven degrees of freedom and a midpoint with one degree of freedom. As well, a fourth node with no degrees of freedom defines the orientation of the beam element in 3-D. Each beam element is comprised of fibres along the length which each have their own temperature history from the thermal analysis and have temperature-dependent steel properties during the subsequent structural analysis. Element type is the same as used in Chapter 4 Sections 4.3.1, 4.4.1, and 4.5. The quantity of fibers ranged from 168 for the column cross-section up to 870 for the largest W610x217 as validated in Section 4.2. In the thermal analysis, the concrete slab was given concrete material properties to accurately capture the heat sink effect, however in the structural analysis the insulation material property was applied to the slab in the beam elements since that material has no mechanical properties. As discussed below, the concrete slab was modelled separately as a shell element and was only included in the beam elements for the effect on the thermal analysis. The mesh size for the structural model was 0.5 m for both shell elements and beam elements, which was shown to give good agreement to

experimental results in Section 4.5 for a real building of similar span, construction, and member size.

In total, 101 different beam elements were included in the analysis for both the steel beams and steel columns, each with a specific time-temperature curve for the design fires considered. Figure 5.11 shows an example of the time-temperature curve being applied to a cross-section for the thermal analysis as well as the material assignment for the cross-section. The term "user fire" refers to any time-temperature curve defined by the user, which is specific to the design fire being considered. The term F20 refers to an initial ambient temperature of 20ºC.

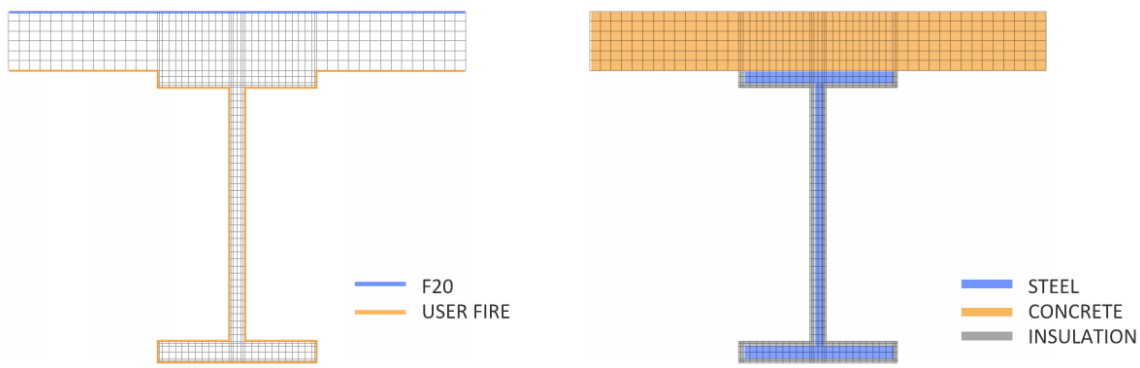

Figure 5.11: Temperature curve and material assignment for W610x217 floor beam

5.7.2 Steel Columns

The steel columns were modelled very similarly to the steel beams in Section 5.7.1. Steel columns remained fire protected since little experimental data exists on the behaviour of unprotected steel columns that are part of a global structural system. Steel columns above the floor were modelled as ambient members, while the steel columns below the floor had a thermal analysis performed which matched the adjacent compartment temperature. For the travelling fire load case, this assumption may be

invalid since experimental observations by Rush et al. (2016) suggests that the compartment temperatures are not uniform along the height during a travelling fire.

5.7.3 Composite Slab

The composite slab was modelled as four sided shell elements, defined by four nodes at the corners with six degrees of freedom each. The shell element contained four integration points on the surface and three integration points along the height. Four rebar layers were included in the shell element. Two of these represented the welded wire fabric at midheight of the slab, while the other two were the metal deck at the base of the slab. Each rebar layer can only resist axial actions in the plan of the shell and acts only in the direction defined by the user (defined by an angle). In this case, each rebar layer was defined at both 0 degrees and 90 degrees. The initial reinforcing mesh was chosen as welded wire fabric 152x152 MW18.7xMW18.7 (123 mm^2/m) as per CAN/CSA-S16-09 reinforcing requirements for composite slabs. This reinforcing mesh was increased near the support girders to achieve model stability during the structural analysis as well as to limit deflections at the fire limit state. The physical mesh size of the shell elements was chosen as 0.5 m x 0.5 m which was shown to give accurate results for the Cardington validation model of similar scale and complexity. For the thermal analysis, the effective thermal thickness of the composite slab (shown in Figure 5.12) was determined from the Eurocode (CEN, 1994-2005) using Eq. 5.3.

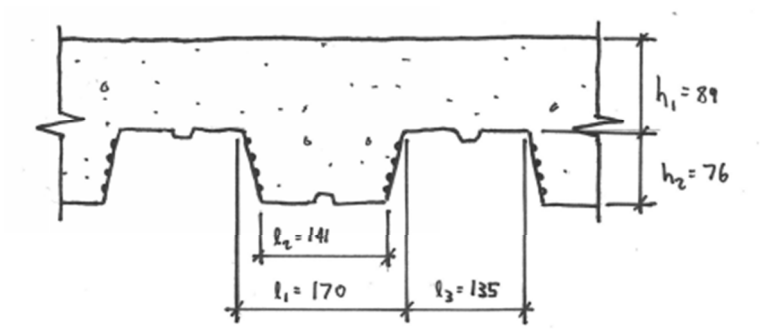

Figure 5.12: Profiled slab floor assembly, 89 concrete on 76 deck (Canam P-2432)

$$h_{eff} = h_1 + 0.5h_2\left(\frac{l_1+l_2}{l_1+l_3}\right) = 128 \text{ mm} \quad (5.3)$$

5.7.4 Loading

The load combination for the fire load case is specified by CAN/CSA-S16-09, Annex K as shown in Eq.5.4.

$$D+T_S + (\alpha L \text{ or } 0.25S) \quad (5.4)$$

The applied loads to be used for the fire design are summarized in Table 5.2. The ambient design of the structure is not presented herein but used the same loads with different load factors applied as per standard practice.

Table 5.2: Summary of applied loads for structural fire analysis

DL	3.10 0.5 **3.6kPa**	89 concrete on 76 deck Self-weight of steel
SDL	1.0 0.7 **1.7kPa**	Partitions Suspended ceiling, M&E
αL	0.5(3.8) **1.9kPa**	Office occupancy with potential for assembly spaces

The total factored load to include during the structural fire analysis is calculated as 7.20 kPa. Loads on the structure from the stair core and elevator, both with concrete block wall around them, were not included since they are in the ambient portion of the structure that is separate from the heated portions considered.

5.7.5 Thermal Analysis

The first step in the structural fire analysis is to perform a thermal analysis on all members, for all design fires. This includes both the structural steel beams and the concrete slab modelled as shell elements. Section 4.2 demonstrated that the parameter most effecting the thermal analysis is the design fire itself, as opposed to any thermal properties of the materials The results of the thermal analysis are shown in Figure 5.13 below for a sample beam, both protected and unprotected. Because of the high thermal conductivity of steel, the unprotected steel beam temperatures essentially match the temperature of the fire. The duration of the travelling fire is much longer than the standard fire or parametric fire, both of which end at 60 minutes.

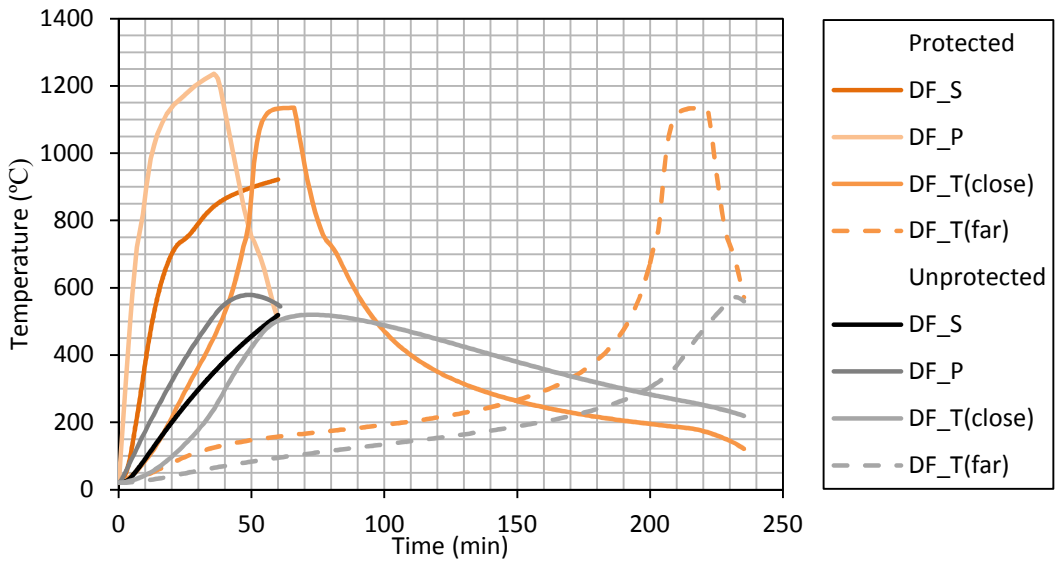

Figure 5.13: Results of thermal analysis for W610x140 beam

As well, the shell elements used for the composite slab had a thermal analysis performed. In the case of the shell elements, the thermal analysis is one-dimensional through the depth of the shell element. The effective thickness of the concrete slab for the thermal analysis was 128 mm as shown in Section 5.7.3.

5.8 Results of the Structural Analysis

The results of the structural fire analysis will be presented herein, with a focus on deflections at the midspan of the bays and the connection forces of the beams. The naming convention for beams and bays is shown in Figure 5.14. Discussion and implications are provided in Section 5.9.

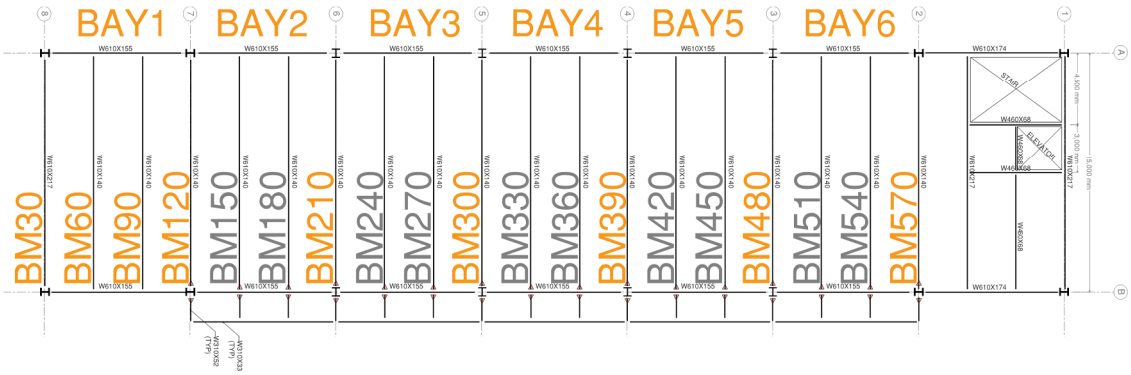

Figure 5.14: Naming convention for discussion of results in Section 5.9.

5.8.1 Calculated Deflections

As discussed in Sections 5.6.1 and 5.6.2, the midspan deflection is a key indicator of the stability and integrity of the structure. Figure 5.15 to Figure 5.18 shows the expected slab deflections for the range of design fire scenarios considered, for both the prescriptive and alternate solutions. Throughout, negative deflection denotes downwards and positive is upwards. The parametric fire has a distinct decay phase which has been labelled, however it is difficult to define a general “cooling phase” since there is still a portion of the decay phase when compartment temperatures exceed structural temperatures and are heating the structure. Once the compartment temperature drops below all structural temperatures, it can be said that the cooling phase is present.

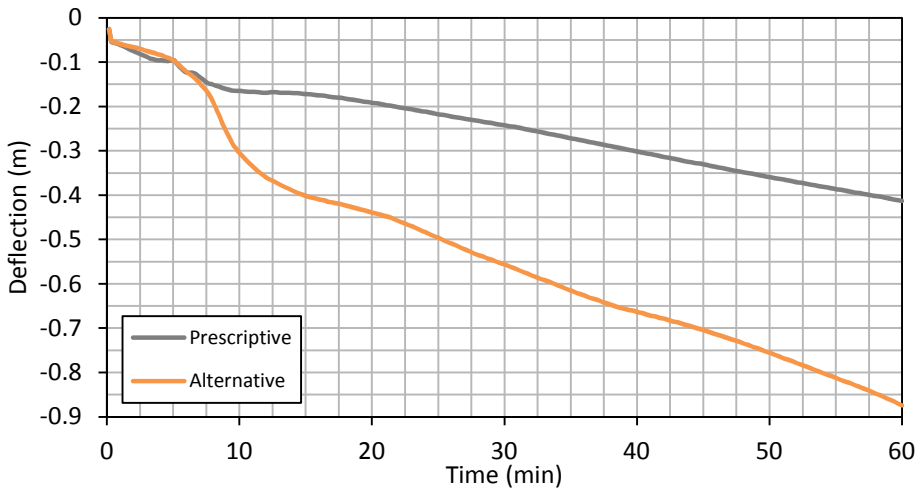

Figure 5.15: Midspan deflection for both solutions, fire scenario DF_S

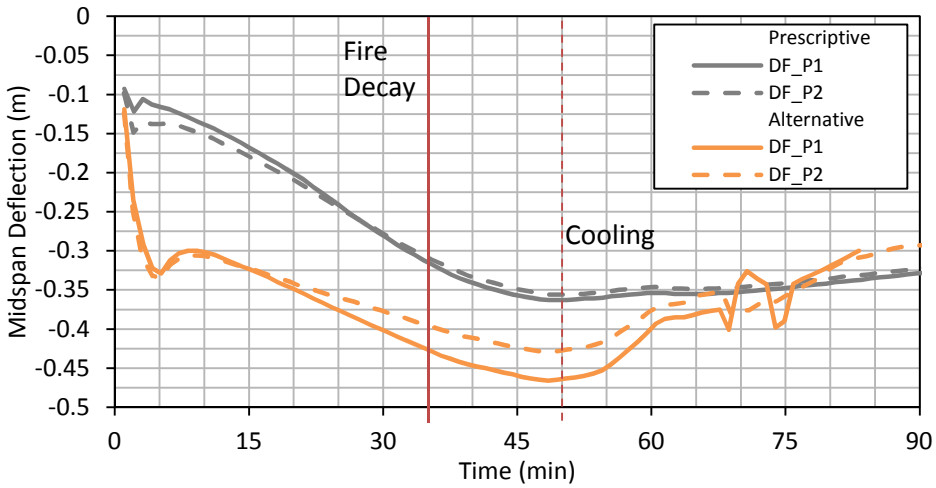

Figure 5.16: Midspan deflection for both solutions, fire scenarios DF_P1 and DF_P2

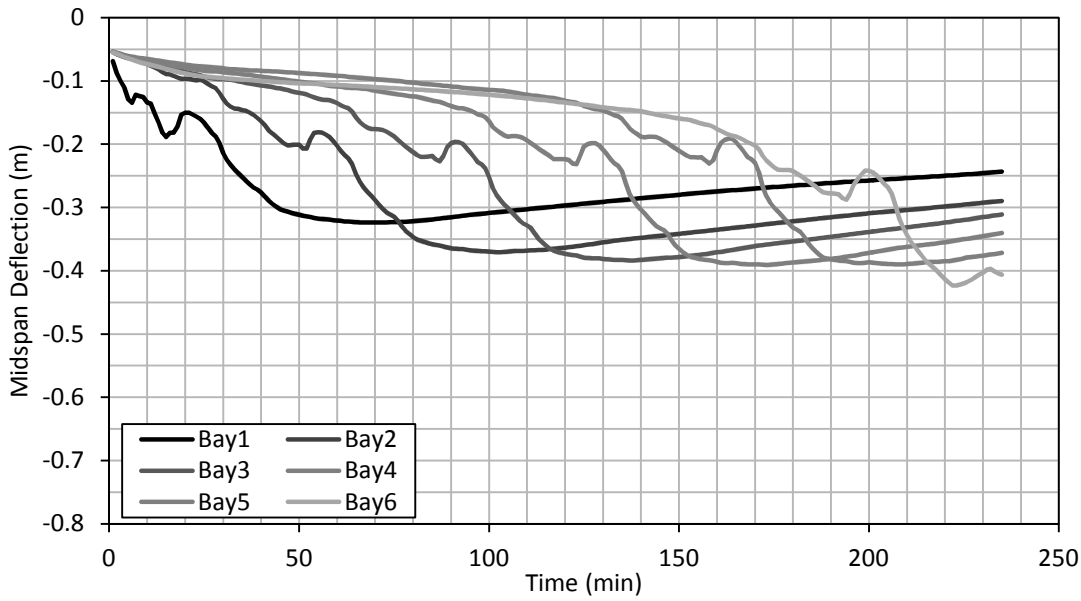

Figure 5.17: Midspan deflection for prescriptive solution, fire scenarios DF_T

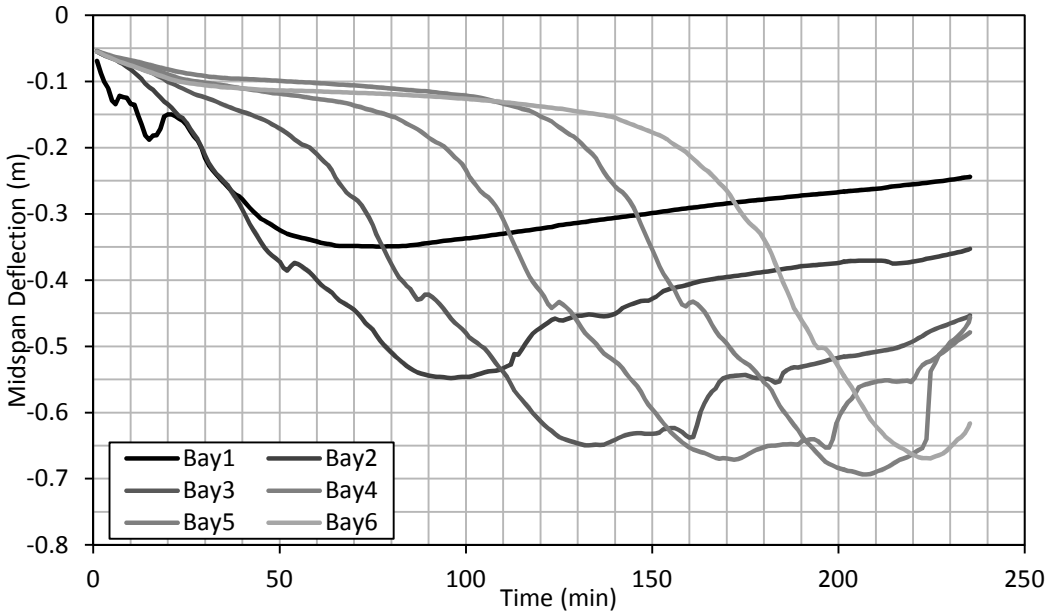

Figure 5.18: Midspan deflection for alternative solution, fire scenarios DF_T

To visualize the general trend of deflections for the travelling fire scenario, the deflected shape of the structure is shown in Figure 5.19 at various times of the analysis. Initially, the far left side of the floor plate begins to deflect since this is where the travelling fire originates, and the location of maximum deflection can be seen to travel to

the right with the fire itself. The magnitude of deflections experienced later in the fire is greater than the early deflections due to the effect of preheating the structure. Both the prescriptive and alternative solution experienced the same trends in deflection behaviour, although specific magnitudes varied.

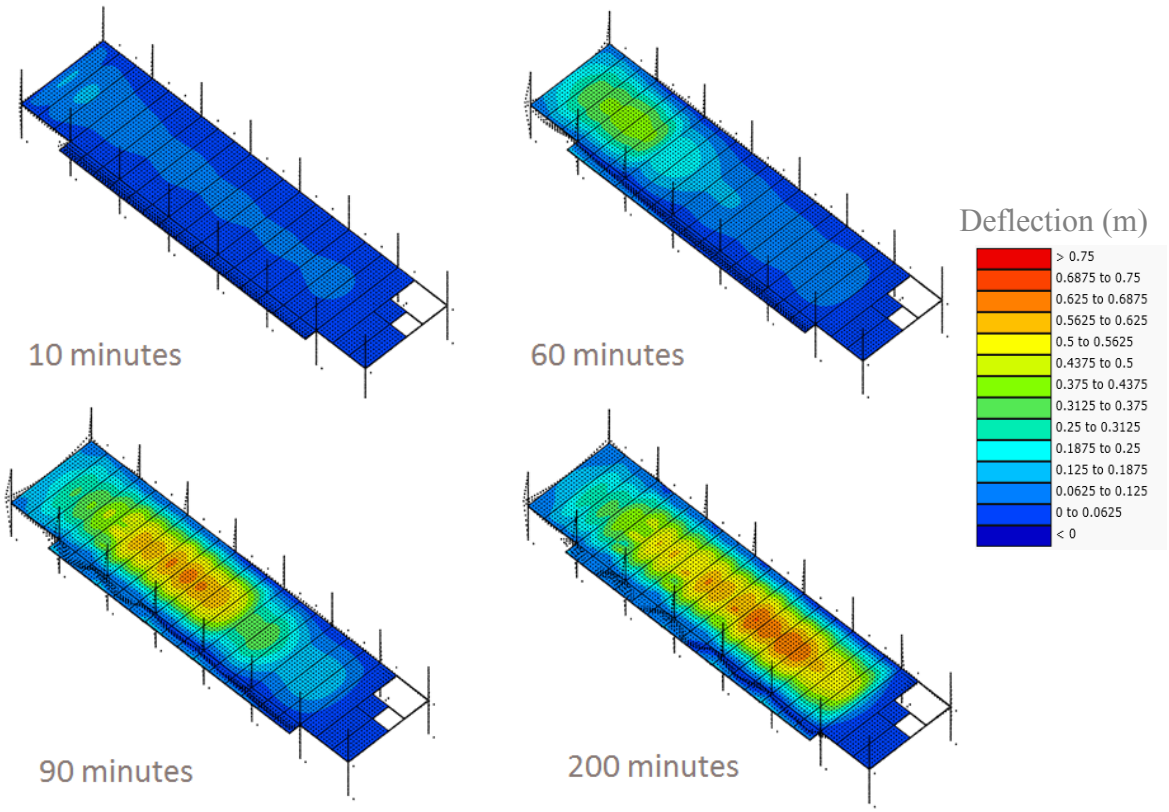

Figure 5.19: Deflected shape of floor due to travelling fire

5.8.2 Calculated Connection Forces

For each fire scenario, the connection forces in the beams are quantified. The connections in the analytical model are perfectly rigid, meaning they are not able to deform and relieve thermal strains. A negative connection force implies the connection is in compression, while positive is tension.

First, the connection forces for the standard fire scenario are shown in Figure 5.20 for the prescriptive solution, and Figure 5.21 for the alternative solution.

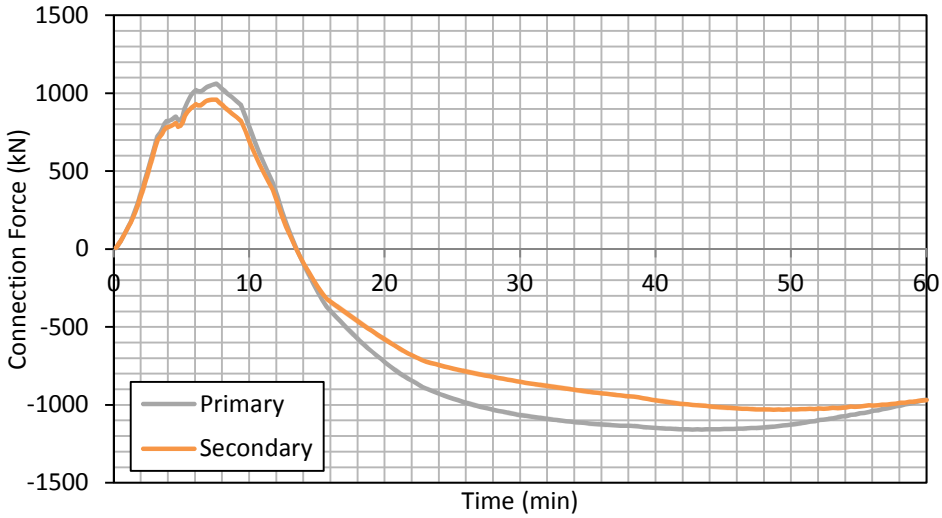

Figure 5.20: Connection forces for primary and secondary beams in prescriptive design, DF_S

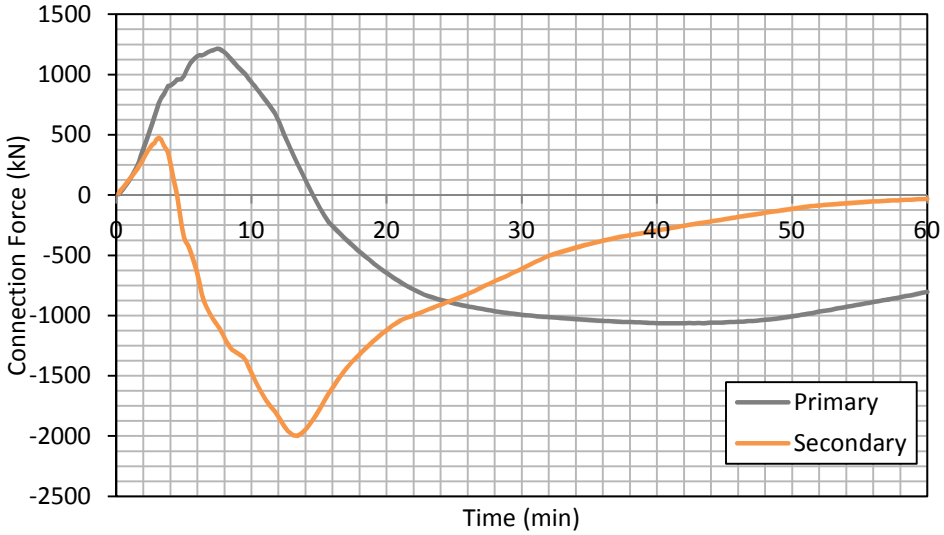

Figure 5.21: Connection forces for primary and secondary beams in alternative design, DF_S

Next, the connection forces for the parametric design fires are shown, with the prescriptive solution in Figure 5.22 and the alternative solution in Figure 5.23. The point at which the fire begins to decay is illustrated, as is the cooling phase of the fire. As mentioned in Section 5.8.1, the cooling phase is not clearly defined for real fires so the author has illustrated the time at which the average

compartment temperature drops below the structural temperatures. In reality, cooling begins over a range of time for different parts of the structure.

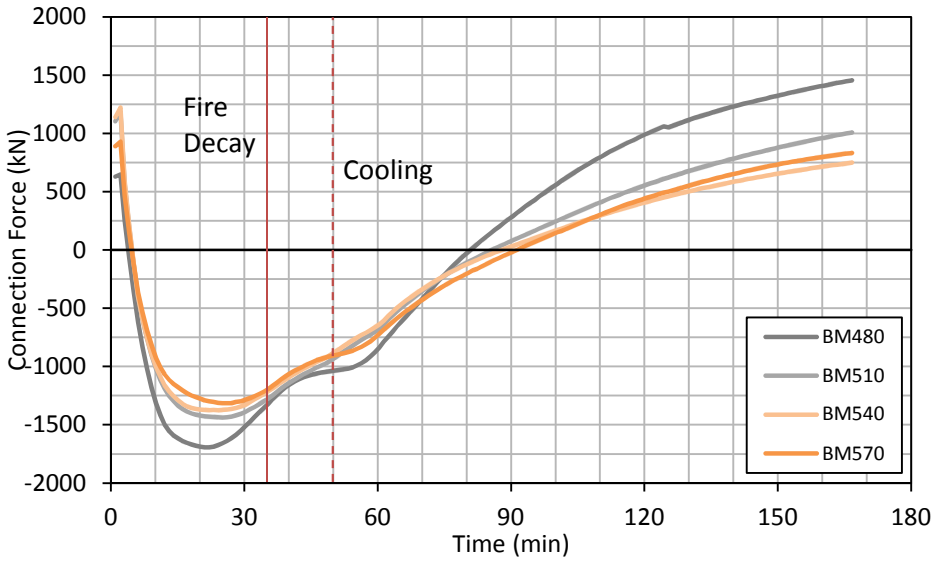

Figure 5.22: Governing connection forces for prescriptive solution, DF_P1 and DF_P2

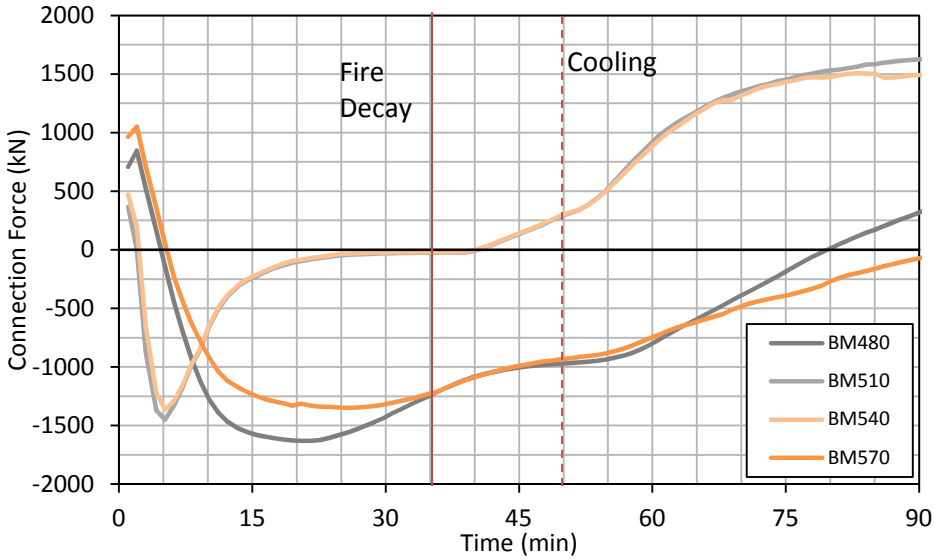

Figure 5.23: Governing connection forces for alternative solution, DF_P1 and DF_P2

The connection forces for the travelling fire are shown next, however they are provided in a series of figures since parts of the structure receive heating at different times and hence differently. The prescriptive solution is shown first, with the primary beams and secondary beams shown independently. The reader is referred to Figure 5.14

for the naming convention used. The primary beams connection forces are shown in Figure 5.24 and the secondary beam connection forces are shown in Figure 5.25. The alternative solution follows, with the primary and secondary beam connection forces shown in Figure 5.26 and Figure 5.27 respectively.

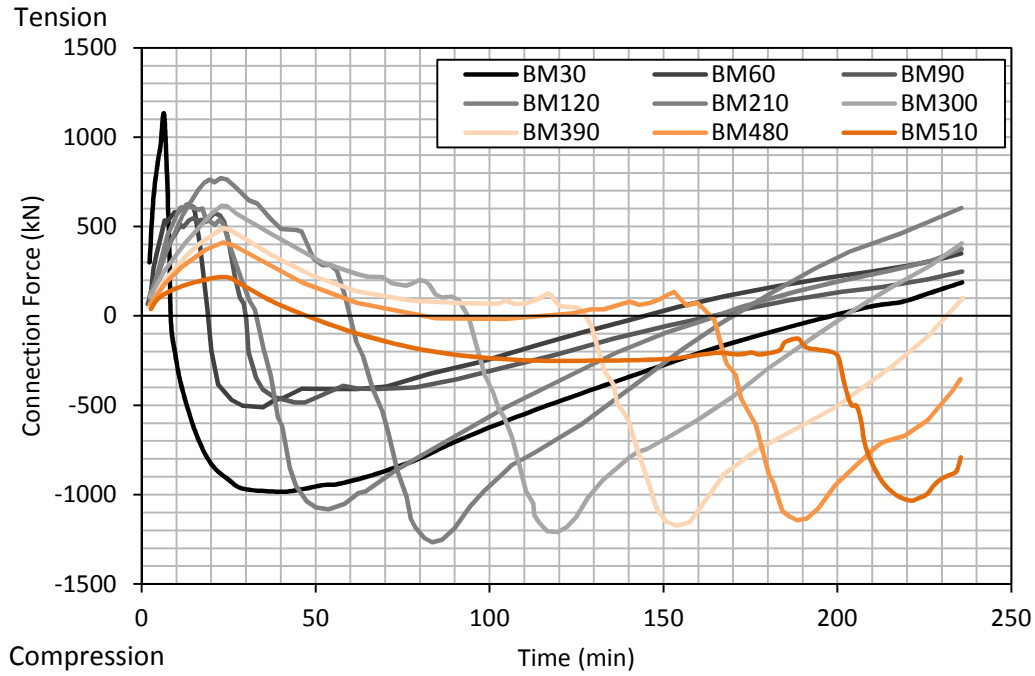

Figure 5.24: Primary steel beam connection forces for prescriptive solution, DF_T

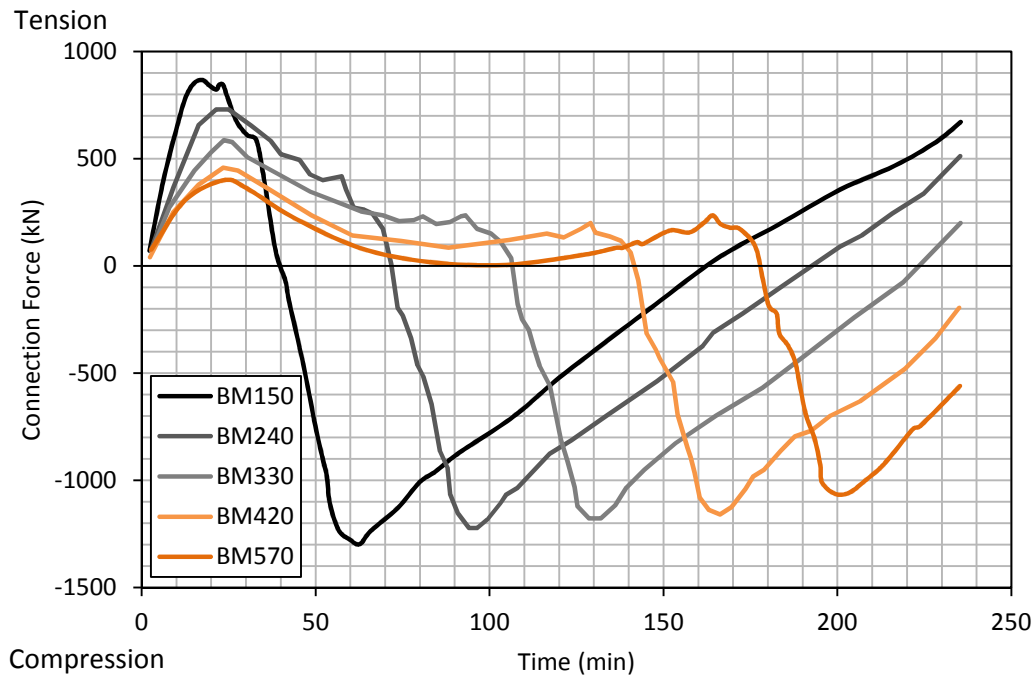

Figure 5.25: Secondary steel beam connection forces for prescriptive solution, DF_T

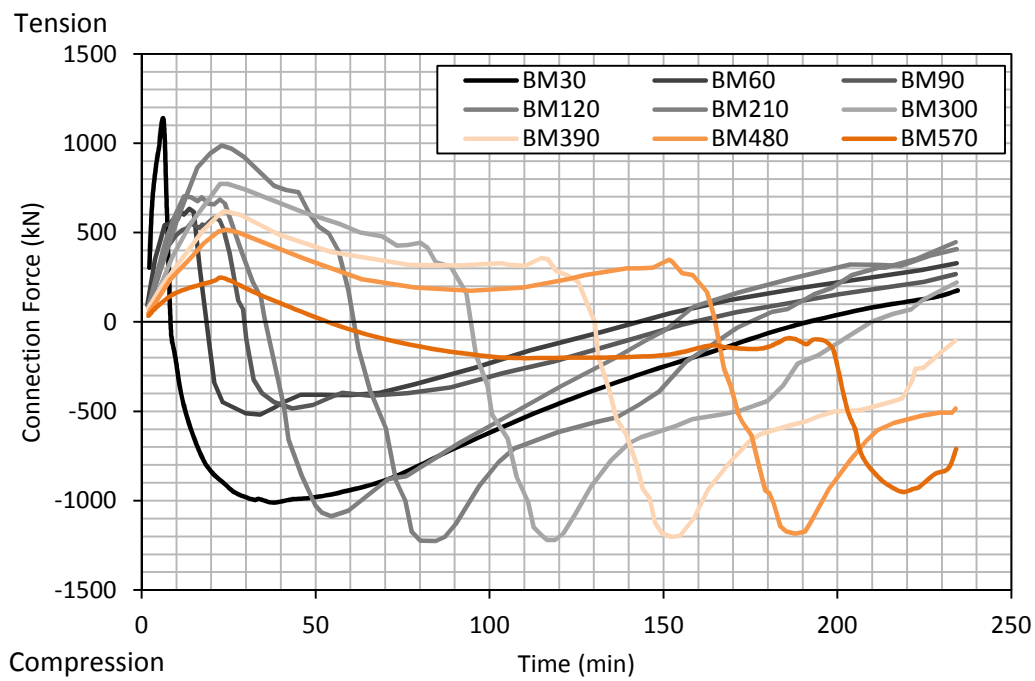

Figure 5.26: Primary steel beam connection forces for alternative solution, DF_T

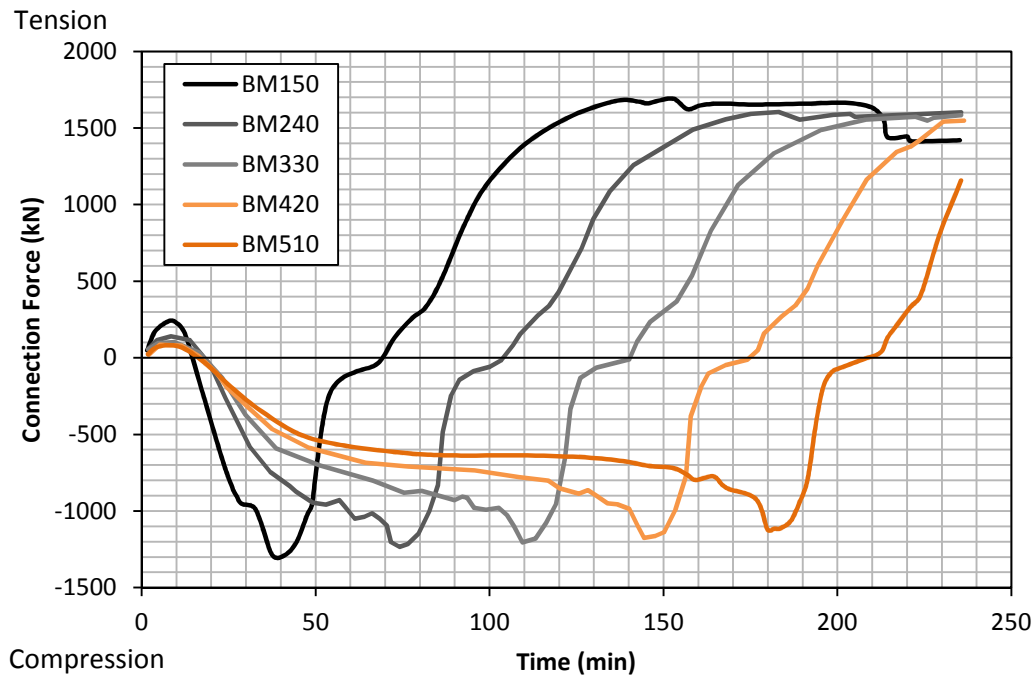

Figure 5.27: Secondary steel beam connection forces for alternative solution, DF_T

5.9 Discussion and Implications

The previous two sections have shown the expected deflections and connection forces for the prescriptive solution and the proposed alternative solution for a range of design fire scenarios. Table 5.3 below summarizes these results.

Table 5.3: Summary of design results

	Prescriptive Solution	**Alternative Solution**
Peak Deflection		
Standard Fire (DF_S)	413 mm	860 mm
Parametric Fire (DF_P)	363 mm	470 mm
Travelling Fire (DF_T)	419 mm	693 mm
Peak Connection Force		
Standard Fire (DF_S)	+1070 kN / -1500 kN	+1200 kN / -2000 kN
Parametric Fire (DF_P)	+1150 kN / -1650 kN	+1620 kN / -1740 kN
Travelling Fire (DF_T)	+900 kN / -1220 kN	+1690 kN / -1220 kN

5.9.1 Deflection

The maximum deflections in Figure 5.15 to Figure 5.18 show that the acceptance criteria are satisfied. The maximum deflection allowed was 911 mm for a 617 mm deep

beam spanning 15 m, and the maximum calculated deflection was 860 mm for the alternative solution with standard fire exposure. Interestingly, for the parametric design fire, the deflection continues to increase once the fire begins to decay since the compartment temperatures still exceed the structural temperatures and continue to heat the structure. Deflections eventually begin to recover once compartment temperatures drop below the structural temperatures and the structures begins to cool. Large plastic deformations still remain as seen in Figure 5.16.

The standard fire seemed especially onerous on the alternative solution with the secondary beams exposed, however the slab did not experience runaway deflection indicative of failure. The other deflection criteria related to stability and integrity of the floor was the rate of deflection, with a maximum of 40.5 mm/min being allowed. According to CAN/ULC-S101, this rate of deflection is not applicable until a deflection of L/30 is met (ULC, 2014). In this case that is 500 mm. The reason for this is that mechanisms can form during the early response that give a large rate of deflection, but then a stable condition is found which continues to support the load. In this sense, runaway deflection is not expected until the floor already has significant deflection. In Figure 5.15 between eight and ten minutes, the deflection increases dramatically to 70 mm/min, before again stabilizing at 10 mm/min to 20 mm/min. Since the deflection prior to ten minutes was only 280 mm, it did not meet the failure criteria proposed. This would seem to suggest that the acceptance criteria for rate of deflection will not govern if alternate paths are provided as runaway deflection will not occur. This demonstrates the resilience that tensile membrane action, and indeed alternate load paths in general, provide to the structure.

Referring to Figure 5.17 and Figure 5.18, it can be seen that the travelling fire has a larger impact on bays furthest from the point of origin. As the fire travels, the far-field region preheats the bays downstream. This results in higher structural temperatures by the time the fire reaches these bays and a larger deflection is experienced. This behaviour was noted by Stern-Gottfried & Rein (2012b). The maximum deflection observed by the travelling fire was 419 mm for the prescriptive solution, and 693 mm for the alternative solution.

An interesting trend is seen in Figure 5.17 and Figure 5.18 for the travelling fire. As the slab deflects downwards, it briefly changes direction and begins recovering deformation (roughly 10 mm to 20 mm), before changing direction again and continuing to deflect at roughly the same rate as before. Comparing the deflection of a particular bay to the axial force of the secondary beams in that bay, this mechanism can be understood. This is shown in Figure 5.28.

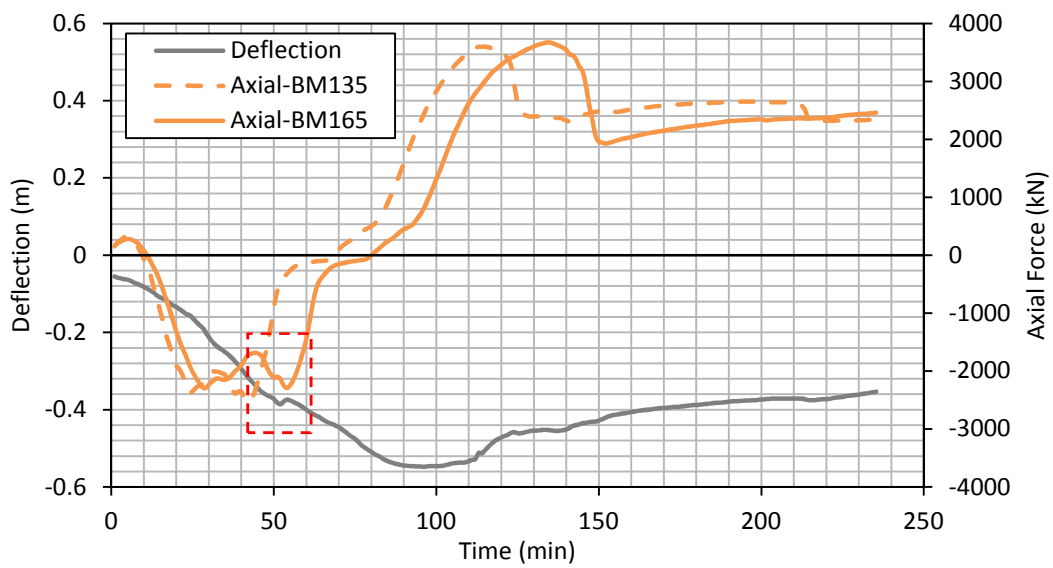

Figure 5.28: Deflection vs axial force in beams for DF_T, Bay 2

As the deformation begins to curve back upwards, the secondary beams are at a transition point in their behaviour. They have reached their peak compressive force as

they experience thermal expansion but are restrained by the slab. They continue to increase in temperature as the near-field region passes by, with the elastic modulus continuing to drop. This shifts the neutral axis of the cross-section upwards and, combined with the increasing temperature in the concrete slab, begins to curve the slab upwards as the compressive axial force above the neutral axis exceeds that below it. The response at this time is still mainly flexural. However, shortly after, the axial capacity of the steel beams is exceeded and the response shifts from flexural to membrane. At this time the beams begin a transition to very high tensile forces since tensile membrane action is beginning to support the slab. The deflection of the slab continues downwards on a similar slope as before this short transition phase. Figure 5.17 shows that protecting the secondary members increases the duration and magnitude of the transition phase.

Looking at the expected plastic deformation remaining after the fire in Figure 5.16 to Figure 5.18, the parametric fire had similar values for both the prescriptive and the alternative solution, on the order of 350 mm. The travelling fire had plastic deformation between 250 mm and 400 mm depending on the bay for the prescriptive solution, with this value increasing to 250 mm to 500 mm for the alternative solution.

It can be concluded that the effect of preheating from the travelling fire is much more pronounced in the alternative solution as it relates to plastic deformation of the slab. Removing secondary fire protection did not seem to affect plastic deformation due to a parametric fire, possibly due to the relatively short duration of the fire itself.

5.9.2 Connection Forces

One of the first and most obvious takeaways from the connection forces shown in Figure 5.20 to Figure 5.23 is that all design fires and all protection strategies do indeed

develop large forces in the connections. In the current prescriptive approach taken by the Canadian building codes, these connection forces are not quantified nor even addressed. For the standard fire scenario with the prescriptive design, it can be seen that the connection forces initially increase to roughly 1100 kN in tension. This is because the slab is not protected and increases in temperature quicker than the steel beams, so the net force in the beam is tension. Eventually, the temperatures work their way through the insulation and begin heating the steel. At this time, connection forces switch to compression and increase to 1200 kN. In the case with the secondary steel unprotected, this pattern is reversed. The secondary beams initially start in compression since they heat up dramatically quicker than in the protected case, before eventually going into tension as the behaviour switches to tensile membrane action.

For the parametric design fires, the importance of the cooling phase is demonstrated. If the analysis ends when the time-temperature curve of the fire returns to 20ºC, the connection forces are seen to only be 1200 kN in tension. This is similar in magnitude to the standard fire analysis. However, the structure is still hot. If the analysis is extended to account for the cooling, we see that the connection forces continue to increase as the structure cools. This cooling is also not uniform since concrete retains heat much better than steel. During this cooling phase, the connection forces increase another 33% to 1620 kN as seen in Figure 5.21.

The travelling fire not only has a cooling phase for each bay, it also has differential heating and cooling along the length of the floor. In Figure 5.24 and Figure 5.25, the same trend as before is observed for the alternative solution where the protected steel beams begin in tension and transition to compression, while the unprotected steel

initially goes into compression before developing very high tensile forces. In the case of the tensile forces, they don't dissipate with cooling of the slab since the plastic deformation of the slab means the dominant load path is through membrane action, not flexure. The floor seems unable to recover flexural response due to the plastic deformations. The travelling fire had the most severe connection forces, up to 1690 kN in tension. The tensile connection forces are more concerning from a design and performance perspective than the compression forces since they are more likely to result in failure and cannot be taken through bearing of the beam itself against the girder.

With the connection forces quantified, one must ask how these are treated in a design scenario. A double angle connection has inherent ductility under fire conditions, allowing the connection to rotate and deflect before failure (Yu et al., 2009). The literature suggests that in a structural fire design, the actual connection force does not need to be designed for in all cases, provided that ductility is provided similar to a structural design for seismic loads (Wang et al., 2013). This has been demonstrated experimentally and analytically. In terms of providing this ductility, the double angle connection is the most ductile, while a simple yet common shear tab connection is the least ductile and hence most brittle under fire conditions (Flint et al., 2013). One issue with the double angle connection is the additional material and labour cost. Research suggests that a single angle connection has some of the same ductility benefits of the double angle connection, likely resulting from the prying action of the perpendicular angle leg and the inherent ductility of bolted connections in shear (Selamet & Garlock, 2011). For the alternative solution, it is proposed to use angle connections throughout in lieu of shear tabs. The moment connections along the edge of the floor will also use angle

connections, and it is assumed that the relatively small moments of the cantilevers can be developed through the bolts of the angles as opposed to needing a positive connection at the flanges. Figure 5.29 shows the shear tab and angle connections discussed herein.

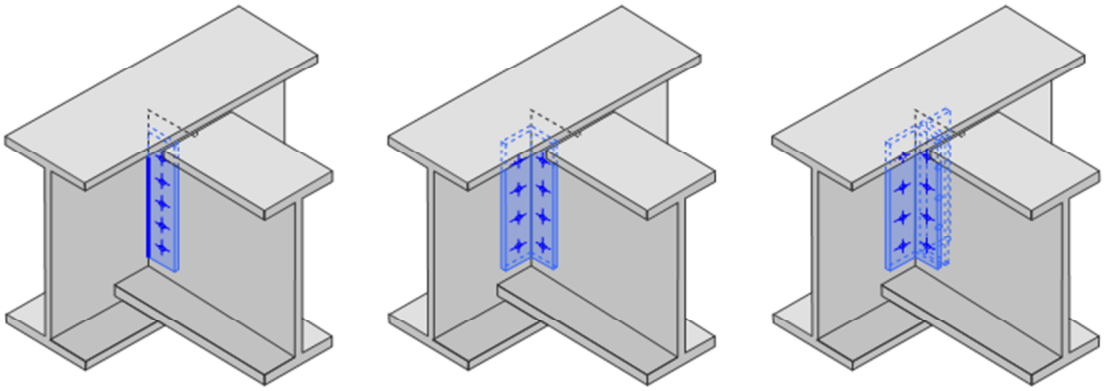

Figure 5.29: Typical shear tab (left), single angle (middle), and double angle (right) connection types

5.9.3 Cost

The main variable in determining the cost savings (or premium) for incorporating the performance-based fire design is the connection cost. Compounding factors for this are that the engineer designing the floor typically is not responsible for the detailed connection design, and the architect may have visual preferences for any exposed connections. Two options have been assessed for the impact on price: in the first, shear tabs are substituted for double angle connections (Table 5.4) and in the second the same shear tabs are substituted for single angle connections (Table 5.5). The unit rates are typical of Toronto construction prices and are based on bid documents the author has been privy to as part of their employment. Alternate unit rates can be substituted based on current market conditions but it is expected the general trend of the financial conclusions afterwards will hold true.

Table 5.4: Cost summary, double angle connections throughout

	Prescriptive	Alternative	Unit Rate	Savings
SFRM	976 m²	725 m²	$50/m²	$12,550
Welded Wire Fabric	63 m² x 2.11 kg/m² x 6 =798 kg	63 m² x 2.91 kg/m² x 6 =1100 kg	$2000/T	-$604
Connection	2.68 kg ea. x 10 x 6 =161kg	6.1 kg ea. x 10 x 6 =366 kg	$5000/T	-$1,025
	4 M22 bolts x 10 x 6 =240 bolts	12 3/4Ø bolts x 10 x 6 =720 bolts	$10/bolt	-$4,800
	6mm fillet weld x 10 x 6 =60 welds	-	$6/12" weld	$360
			Total	**$6,481**

Table 5.5: Cost summary, single angle connections throughout

	Prescriptive	Alternative	Unit Rate	Savings
SFRM	976 m²	725 m²	$50/m²	$12,550
Welded Wire Fabric	63 m² x 2.11 kg/m² x 6 =798 kg	63 m² x 2.91 kg/m² x 6 =1100 kg	$2000/T	-$604
Connection	2.68kg ea. x 10 x 6 =161 kg	3.29 kg ea. x 10 x 6 =197 kg	$5000/T	-$180
	4 M22 bolts x 10 x 6 =240 bolts	8 3/4Ø bolts x 10 x 6 =480 bolts	$10/bolt	-$2,400
	6mm fillet weld x 10 x 6 =60 welds	-	$6/12" weld	$360
			Total	**$9,726**

In this particular case, utilizing performance-based fire engineering to design the structure for fire saved between roughly $6500 and $9700. The total floor area is 1035 m^2, for a savings of at least $6.36/$m^2$. This value will obviously change depending on the market conditions where the design is done and the "baseline" solution that any resulting design is compared against. Simple shear tabs were assumed throughout initially which meant the savings were minimized.

Although the floor area was 1035 m^2, the area directly impacted and altered by the structural fire design was just 675 m^2 since tensile membrane action was relied on for

rationalized fire protection optimization. This means 35% of the floor area did not have conditions conducive to using tensile membrane action because of aspect ratios or support conditions. This also suggests that further testing of full-scale structures for realistic fires can provide the observations and data necessary to extend PBFD methods to that remaining 35%, or that reconfiguration of the structure at the beginning of the design stage can increase the opportunities for optimizing fire protection.

The savings for using a PBFD will increase as the area of floor lending itself to experimentally validated methods increases. The savings would likely be marginal for using an element-by-element design approach based on limiting temperatures since the resulting structural temperatures were quite severe for the parametric and travelling fire scenarios. The decision to use PBFD may also depend on if the expected savings and increase in robustness are justified for the consulting cost. The decision to use PBFD can have a large impact at the onset of a project since the very layout of the building will determine where opportunities exist for applying methods that have been validated experimentally. For a small floor area and a building of low or regular importance, using PBFD methods may not always be justified.

5.9.4 Resilience

A large opportunity for utilizing PBFD is increasing the inherent robustness and resilience of the structure for fires. Referring to Section 5.8.1, it was shown that the prescriptive solution and the alternative solution had similar plastic deformations after the fire depending on the fire scenario. In some cases this plastic deformation was on the order of 250 mm. This amount of deflection is not suitable for immediate occupancy after the fire, even if the floor is safe to support the loads. Regardless of if secondary fire

protection is provided or not, it is likely that the floor will need immediate replacement following the fire before the businesses on the adjacent floors can re-occupy. There is not much experimental data available that describes the conditions of floors after realistic full-scale fire tests, which makes it hard to determine the scope of repair required.

There is likely a portion of design fires not considered herein that do not govern the structural response but do give different results between the prescriptive and alternative solutions. One can envision a localized fire below a protected steel beam that does not cause structural damage, yet that same fire under an unprotected steel beam may cause local failure of the flange due to localized and immediate heating. This could theoretically need repair. Small scale fires such as this are not feasible to include in deterministic design since we want to capture the worst case design fire, although they could be included as a possible scenario in a probabilistic approach. In this way, probabilistic approaches are better suited for dealing with a range of fires and quantifying the overall risk to fire damage and interruption in general, and not just the risk of failure for defined design fires.

Another benefit of utilizing PBFD is that for tenants that care about business continuity planning, they can begin to understand the expected performance of their building for a range of possible realistic fires. They can understand that a structurally significant fire may result in the entire floor needing to be replaced due to large plastic deformations, and assign a probability and a cost to this. By understanding the scope of expected fire damage ahead of time before actual fires happen, a tenant is better able to plan for a fire and ensure their business continues to operate even if it can't occupy this particular area for a period of days or weeks after a fire. In today's business environment,

it is critical that operations return to their minimum operating capacity by 8am the following day after an event to ensure they stay in business (Hay, 2016). For most severe fires, it will become clear through PBFD that this is not possible and that the organization will need to have alternate accommodations pre-arranged in their business continuity plan to increase their own resilience to fire events.

5.10 Summary

This chapter has demonstrated the opportunity for a proposed alternative solution to have fire protection removed from the secondary steel beams. This is accommodated through a structural fire analysis, which showed the connections need to be detailed as more ductile, at a cost premium, and also the secondary layer of reinforcing mesh must increase from an area of 123 mm^2/m to 170 mm^2/m. This brings the total reinforcement for the slab up to 293 mm^2/m. For a worst case design fire, the slab will deflect up to 693 mm. Following the fire, assuming burnout, it is expected that the slab will have on the order of 250 mm plastic deformation. The slab will require extensive repair as a result of this permanent deformation. Future study is required to better quantify this damage state that requires repair. By applying PBFD to this case study building, it is also expected to save \$6.36/m^2 in material cost.

With the efficacy of PBFD demonstrated for a contemporary Canadian structure and the benefits quantified, the next section will develop and propose a framework for growing the implementation of PBFD across Canada. Specifically, assessing structures for the fire limit state and providing more resilient buildings for the benefit of the public. After developing a framework, next steps for implementation will be proposed which international experience have shown to be effective.

6 Chapter: Framework

This chapter proposes a framework for evolving and implementing performance-based fire design (PBFD) within Canada. The benefits of doing so have been previously discussed relative to international practice in Section 3.2 and within Canada in Section 3.6, as well as demonstrated in the alternative solution of Chapter 5. Another checklist of design steps is neither necessary nor novel. Rather, we, as an engineering community, need to acknowledge high level goals for the structural fire engineering discipline within Canada. This will be important for performance-based approaches to be given credibility during the design phase of a project when they can have the most significant impact on the outcome of that project. As well, it's important to realize that structural fire engineers cannot flourish on their own. They must receive the same respect and support mechanisms that the more mature profession of structural engineering receives for example. Referring to Section 3.3, we can see this includes a body of competent and respected peers, an education framework that provides the training necessary and responds to industry demands and research developments, and competent review during design and implementation from both regulatory bodies and peer reviewers where the complexity warrants it. This chapter is intended to be an open discussion from the author and the reader is encouraged to participate. It will be discussed at the end of the chapter that a proposed next step for implementing PBFD is an open discussion with the appropriate parties present as this was shown internationally to encourage progress.

6.1 Education

The current state of education for fire safety engineering (FSE), and in particular performance-based structural fire engineering, was discussed in Section 3.7.1. The three

main fire safety engineering institutions in Canada each offer just one structural fire engineering course in their curriculum. Each course follows a text that does not contain recent research findings or methods used in contemporary practice, and most notably was written over fifteen years ago (Buchanan, 2002). Woodrow et al. (2013) provide a detailed critique of the fire safety engineering curriculum using outcomes from a fire safety leadership seminar series that took place in 2011 (with other seminar series following in 2012, and 2013) at the University of Edinburgh supported by The Lloyd's Register Educational Trust (LRET). Conclusions of that paper have informed the education aspect of the proposed framework proposed herein. An underlying theme of that paper is that FSE is an immature discipline and must develop towards a competency based discipline with a focus on exposing students to real-world design problems.

The author's opinion, based on observations of international trends and education approaches, is that the current education system is not preparing future fire safety engineers to solve the structural fire problems of tomorrow if PBFD in Canada is indeed progressing towards the complexity of international practice. This trend was demonstrated in Section 3.6. Currently, in order to receive training in advanced structural fire engineering methods, as it is unavailable in the curriculums, a student or practitioner must enter into a 'research-based' graduate degree to receive specialized one-on-one guidance from a mentor or supervisor. This is more akin to the training an apprentice would receive in a trade as opposed to a curriculum that supports a mature profession.

An exemplar international program that shows where Canada can progress towards is the FSE program at the University of Edinburgh. Here, the curriculum has been refined to train future engineers in PBFD with specific skillsets and characteristics

that the local industry requires, mainly from renowned international consulting firms such as Arup. In this program, one quarter of final year civil engineering students are exposed to fire safety and fire dynamics courses. They then have a range of graduate degree specializations available within the context of FSE, and academia-industry collaboration is the norm (Richardson, 2003). By engaging students during their undergraduate degree to build a baseline knowledge of fire safety engineering, this allows for their time during a graduate degree to be maximized and redundancy eliminated. Likewise, engaging students that are practicing engineers can ensure they enter the field with a complimentary skillset and an understanding of competency that comes from a mature engineering discipline (Spinardi, 2016). The Edinburgh example demonstrates that the structural fire course currently offered by Canadian institutions needs to be expanded to include more advanced methods that industry will begin to, if it is not already, demanding. Performance-based design may be enabled if structural behaviour in fire is taught from first principles instead of analytical methods for determining a fire resistance period for a well-defined standard fire.

The method in which that student is taught is just as important as filling the knowledge gap with an updated university curriculum. As mentioned in Section 3.7.1, structural fire engineering in Canada is taught mostly by applying a series of equations as opposed to open-ended design problems representative of real situations. Woodrow et al. (2013) state that fire safety engineering graduates must be able to identify, define, and solve their own problems as opposed to applying set methods to well bounded problems. At the very least, it is recommended that the structural fire course *Fire Resistance* be expanded to incorporate and demonstrate more design type problems representative of

what will be encountered in practice. This is a part of the European structural fire education mentioned earlier which the author also assessed. Here, exemplar case studies such as Heron Tower or the proposed Pinnacle Tower are discussed with an overview of how the structure fits into the overall fire safety strategy. These buildings used state of the art PBFD methodologies which are taught to the students.

A progression in the engineering curriculum is required for practitioners to competently develop complex PBFD solutions for buildings if we are to treat fire safety engineering as a profession, with some initial first steps being in the author's opinion:

- Expand the undergraduate course offerings in fire to include all fundamentals of fire in dynamics, structures, and human behavior. This instills the fundamental knowledge of each and can enable holistic application of them to an overall fire strategy;
- Expand fire resistance to include whole-building considerations and design methods;
- Teach first principles of structural behaviour, not equations for fire resistance;
- Require pre-requisites within the graduate level fire engineering courses so that redundancy can be removed (take fire resistance after fire dynamics for example); and
- Increase collaboration with industry to identify any skills gap with structural fire engineering graduates and to create a feedback loop in the curriculum.

6.2 Competency

The question of competency is an important one in PBFD, and was a central theme in Torero's critique of FSE being either a trade, occupation, or profession (Torero, 2012). In a profession, it is stated that the individuals are regulated as opposed to the activity itself and those with licensure are thus assumed to have the competency and ethical obligations necessary to offer that service. This is contrasted with an occupation

which is self-regulated instead of state-regulated, and a trade where the activity itself is regulated. For PBFD to be offered as a service, it is argued that it must be done from a profession. The author agrees with this assessment, and the competency portion of the proposed framework is derived from Torero's (2012) assertion that we must make FSE a profession.

When performance-based approaches were first introduced in New Zealand, it was estimated that 30% of the designs were done by poorly qualified engineers (Buchanan, 1999). This issue was amplified by the fact that authorities themselves were not always competent in the methods being used to assess fire performance and would rely on other consultants for peer review. Spinardi (2016) argues there will always be an "expertise asymmetry" between regulators and practitioners because regulators cannot reasonably keep up with the advanced and complex methods increasingly being implemented by designers. As well, the decreasing fire fatalities over time give little incentive to provide more funding for regulation over fire safety. This requires that fire safety engineering move towards a profession if there is an expertise asymmetry since the authorities have no choice in that case but to regulate the individuals and determine what constitutes the experience, education, and ethics necessary to offer those services to the public. The observed issues with peer review and regulatory consistency were also discussed in Section 3.2.1. The fire safety engineering community was said by Woodrow et al. (2013) to have an issue with poor individual awareness of competency because of the size of the profession, a lack of accreditation procedures, a historical over-reliance on prescriptive methods, and perhaps the education system which supports all the above.

Currently, it is unknown what level of competency exists across the profession in Canada. Published Canadian fire design case studies to date do not contain detailed information and are more aimed at marketing specialized services than promoting knowledge sharing and instigating discussion as shown in Section 3.6. Chapter 5 of this thesis aimed to address that by transparently showing a proposed structural fire design which can be openly critiqued, discussed, and learned from by the structural fire engineering community within Canada. A more transparent dialogue needs to occur between practitioners, educators, and authorities to understand what is possible with the available knowledge in Canada (as well as what can be used abroad here) and where knowledge gaps exist. These knowledge gaps will likely represent future opportunities for fire safety engineering in Canada which the public will benefit from if implemented responsibly. Performance-based design methods will demand practitioners understand the first principles behind their designs and be able to demonstrate this to authorities. For structural fire engineering in particular, practitioners with a background in structural engineering will have a greater awareness of how to identify competency since they come from a mature profession.

As well, authorities must be trained in the methods being proposed to them so that they can intelligently provide input throughout the PBFD process and review the design as discussed in Section 3.2.1. Authorities should always have some baseline understanding of what is being proposed so they can have an intelligent dialogue, ask the right questions, and seek outside peer review where warranted, even if an "expertise asymmetry" exists. Adding independent peer-review to this process increases the level of competency even more as it creates an environment of intellectual discussion and

constructive feedback in the FSE community. Given the lack of accreditation in Canada for FSE, competency will need to be defined by the profession itself, and will exist amongst qualified peers in the profession. The reality of fire safety engineering is that fires are rare and we only recognize the implications of inadequate competency when the designs are ultimately tested by real events, if at all (Spinardi et al., 2016).

6.3 Implementation

In the discussion of competency in Section 6.2, one aspect that is missing is the fact that this competency needs to be transparently demonstrated to be authentic and to be understood and benchmarked by peers. The objective-based code in Canada has introduced the notion of designing alternative solutions and demonstrating their equivalence to acceptable solutions. This is guiding us towards performance-based solutions (Bergeron, 2008). True performance-based design however requires the stakeholders to agree on their performance goals and acceptance criteria of the design instead of simply benchmarking back to acceptable solutions, and this requires competent engineers to then analytically demonstrate that the performance is achieved.

The difference in approach and in the complexity of methods used is stark when comparing the Canadian practice in Section 3.6 to the international case studies in 3.4. The author believes that to grow structural fire engineering usage in Canada, the designs must be incremental in nature so that competency is gradually built towards whole-building methods that are more holistic with a broader fire protection strategy. Each design that goes through the approval process will require the designer, peer reviewer, and AHJ to have at-least that level of competency as discussed in Section 3.2.1. There may be differences in specific skillsets and abilities, but all parties must have an

understanding of what is being proposed. By gradually implementing PBFD methods we can increase confidence in the regulatory community and thus create more opportunities for innovative analyses and design (Meacham, 2014).

The Canadian case studies to date (summarized in Section 3.6) have not actually assessed the structural performance for fire. They have determined structural temperatures using fire dynamics models but have stopped short by simply comparing that temperature to a limiting temperature in steel. The author believes the building code is responsible for this approach because of the emphasis it places on demonstrating equivalency of rated assemblies even when practitioners do implement alternative solutions. However, there is an opportunity to begin working towards actually assessing the structural performance as demonstrated in Chapter 5. Fire is a load case on structures that can and should be considered if the expertise exists similar to how buildings are designed for rare but potentially catastrophic events such as earthquakes and malicious attacks. This is the approach being implemented in ASCE 7, that if one deviates from the prescriptive methods for fire protection of structures they need to perform a structural analysis of that building at the fire limit state and cannot simply demonstrate a limiting temperature is met (ASCE, 2016). The ASCE Fire Protection Committee of which the author is a member is developing the industry-accepted design guidance that will facilitate this process and see publication as early as next year. This can enable structural fire engineering to begin to see gradual implementation in contemporary Canadian construction.

6.4 Proposed Framework

Consolidating Section 6.1 to Section 6.3, the proposed framework for developing performance-based fire design responsibly in Canada, with an emphasis on structural fire design, is:

1. Competency must be demonstrated in both the designer and the AHJ
2. Regulation and peer review must be thorough and consistent
3. Holistic fire design must be taught in education
4. Implementation shall be incremental and transparent

The framework is purposely broad, as it speaks to the intent behind performance-based fire design and what the end goals should be as opposed to specific ways to implement it, all while ensuring that the practice evolves responsibly with more noble goals than simply reducing fire protection. This framework was developed specifically for composite steel construction since there is a large gap between current Canadian practice and contemporary international design that market pressures will soon want to close since the benefits have been demonstrated (Section 3.4 and Chapter 5).

6.5 Path Forward

With a framework proposed, the natural question is how to begin implementing this since the framework is qualitative. The first step to growing PBFD is to acknowledge that structures, in particular composite steel structures, can be exposed to a fire limit state that we as engineers are able to design for. We do not need to rely on prescriptive guidance as was demonstrated in Chapter 5 which can be overly restrictive and not representative of reality. Even before any analysis is done though we have a general understanding of how structures will behave for fire and can inform the design team of these (Flint et al., 2013).

This means considering how the building will expand/contract, what types of connections are vulnerable to failure during cooling, how columns will react, if bays are conducive to tensile membrane action or not, etc. Once that discussion of a fire limit state in structural design starts, we can begin to implement the methods and actually design structures for fire. Proposed implementation steps based upon research herein are:

- Educate AHJ's, and in the interim require third party peer reviews by peers with the competency that the design requires;
- Educate building owners, property managers, architects;
- Encourage industry-academia collaboration through graduate studies, student co-op work, or collaborate research partnerships;
- Begin to implement structural fire designs transparently and publish the methods and results in case studies to grow the profession in Canada. There is a dearth of published case studies as discussed in Sections 3.2.1 and 3.6 with the required level of detail to invite critique and improvement;
- Ask how we define competency in a profession that has no formal accreditation. Competency and performance in a profession are best assessed by other peers within that profession, which is why we must strive for transparency (Woodrow et al., 2013). Structural fire engineering, if we treat fire as a load case, is a logical progression of the roles and responsibilities of a structural engineer. In this sense, competency is well understood and the accreditation framework is in place. However, the structural fire engineer will likely require input from a more broadly trained fire protection engineer, a skillset that is not as evolved in terms of being a profession;

- Seek to develop the fire engineering curriculum to solve design problems, not to train individuals to apply a prescriptive code. As PBFD increases in use, a demand will be created for specialized degree programs and a feedback loop will be created with industry. This has already been demonstrated in the United Kingdom between successful consultancies and the leading academic institutions with demonstrated accomplishments and research thrusts. Two notable examples incorporated in the alternative solution of Chapter 5 were work done by Stern-Gottfried & Rein (2012a; 2012b) and Rackauskaite et al. (2015).

Compiling all of the above, a logical next step is to facilitate a workshop that includes practitioners, academics, and authorities since many of the steps simply require increased communication and collaboration. This has been successfully done in the past in the United Kingdom with a seminar series aimed at explaining the fire safety methods incorporated in Plantation Place South prior to its approval and eventual construction (Corus, 2004). A similar leadership seminar series took place in 2011, 2012, and 2013 at the University of Edinburgh supported by The Lloyd's Register Educational Trust (LRET) with a general focus on FSE and a more detailed discussion on the continued development of PBFD and how fire safety professionals are being trained to provide those services (Woodrow et al., 2013). The seminar also led to discussions about the role of regulators in an environment that is increasingly performance-based and what the societal impacts of this shift may be (Spinardi et al., 2016). This seminar included industry, academic, regulatory, and fire service representatives. Feedback from these seminars was that independent dialogue began to occur around the issue of fire safety and that the outcomes of such would be beneficial to the fire safety community at large

(Bisby, 2011). Lastly, a 2014 workshop was hosted by the National Institute of Standards and Technology (NIST) and the International Council for Research and Innovation in Building and Construction (CIB) to develop a roadmap for fire resistance R&D initiatives with an acknowledgement that PBFD was beginning to be developed and implemented in the United States. Again, this workshop consisted of industry, academic, regulatory, and fire service representatives. The outcomes of this were a roadmap of North American fire research to fill existing knowledge gaps and implementation plans for each specific material which is now being acted upon by the author and various international fire experts (NIST, 2015).

A similar workshop to those discussed above is recommended as a first step to begin working towards the framework proposed in Section 6.4. Even if the only outcome of this workshop is a discussion around the issue of fire safety in contemporary structures and the appropriateness of developing structural PBFD solutions in Canada, it has been shown that sometimes all it takes is this open communication to create momentum and promote innovation. Section 3.2.1 has highlighted that PBFD cannot be pushed by a lone individual or organization in order for it be implemented properly. We have a unique situation in Canada to build on the international lessons learned from countries that implemented PBFD before us and to develop our own PBFD best practices to construct buildings that are safer, more economical, and more resilient.

7 Chapter: Conclusions and Recommendations

7.1 Summary

Despite the fact that contemporary Canadian structures are designed for fire safety in a prescriptive fashion, it is clear that the industry is headed towards a more performance-based approach. The alternative solutions clause of the National Building Code of Canada was implemented in 2005 to allow for innovation and to put the industry on a path towards performance-based design, but the code itself is still heavily rooted in the acceptable solutions with their own inherent level of performance. The comprehensive literature review of Chapter 3 identified the individuals, events, and ideologies that developed the Canadian building code into the objective-based building code that it is today. The literature review also contrasted contemporary Canadian fire design case studies with those that have been implemented internationally, with an emphasis on structural fire design. This highlighted just how large the gap is between the complexity and technicality of Canadian fire design compared to international practice.

With such a gap existing, the research herein looked at how Canadian practice can evolve to enable more advanced structural fire designs. The research was motivated by a desire to understand how this can be done responsibly with public safety and high performance buildings in mind; not simply trying to reduce costs in fire protection. The literature review identified a lack of transparency in the Canadian case studies being published and a general lack of engineering and scientific discussion about the methods being used. For this reason, benchmark modelling was performed with published experimental data to transparently demonstrate competency being built such that a whole structural floor could be assessed for the fire limit state. This provided the model

validation necessary to then propose a novel alternative solution in Chapter 5 that began to implement the advanced methods already being used internationally into a real Canadian structure. This is the first known case study to implement the improved travelling fire methodology in a real building configuration. Additionally, the author has included proposed acceptance criteria from the ASCE/SEI Guideline for Structural Fire Engineering, of which he is a member. Aside from demonstrating that the alternative solution could meet the acceptance criteria, the model was able to demonstrate quantifiable benefits that were enabled by the performance-based fire design (PBFD), and was also novel in identifying mechanisms in the behaviour of the composite steel floor.

Finally, the research sought to develop a framework for implementing PBFD in Canada in a responsible manner. This framework included aspects related to the education, competency, and implementation of PBFD that are specific to Canada but informed from international experience. The literature review of Chapter 3 informed the framework while the alternative solution of Chapter 5 provided a valuable Canadian case study to demonstrate the efficacy of the framework and of PBFD in general. A proposed next step for progressing PBFD in Canada is to facilitate a seminar of industry, academic, regulatory, and fire service professionals to begin the discussions and work together to identify further barriers.

7.2 Conclusions

The primary conclusions of the research presented herein can be summarized as:

- Canadian structural fire engineering, and fire safety engineering on a more broad level, are fundamentally trades. There exists no specific accreditation and the end-product itself is regulated through prescriptive guidance. Canada is not unique in this

regard. However, we must work towards developing fire safety as a profession. Steps proposed herein included adjustments to the university curriculum to better train engineers to solve design problems and competently implement solutions, and increase industry-academia collaborations. Also, a profession demands a greater understanding and transparency around the competency of the practitioners.

- The alternative solution of Chapter 5 demonstrated the efficiency of PBFD within Canada. The model incorporated contemporary Canadian construction methods, referenced Canadian and North American design guidance where possible, and was shown to meet acceptance criteria that will be proposed and balloted shortly through industry consensus in the ASCE/SEI Guideline for Structural Fire Engineering. It will be available for industry-wide reference thereafter. The connection forces, expected behaviour, and plastic deformations were all quantified to inform the stakeholders of the expected performance during fire. This is not possible with prescriptive guidance. Secondly, by implementing PBFD, the alternative solution was able to reduce the construction cost by \$6.36/m^2 using current Canadian cost estimates.
- Implementing PBFD successfully is reliant on having experimental data available to validate the models and identify mechanisms in the structure, and on the fire design strategy being discussed early in the design stage. This was shown in Chapter 5 when only 65% of the floor area of a real building configuration lent itself to tensile membrane action (TMA) as demonstrated and validated through available experimental data. The other 35% of the floor represents possible opportunities for experiments to validate sub-models of the complex framing, or for reconfiguration during early schematic design to enable mechanisms such as TMA.

- Resilience of the structure was identified as a key outcome of PBFD. For a given fire scenario, stakeholders can understand the expected plastic deformations and connection demands in the building which informs the repair strategy and business continuity plan. The type of fire was also found to impact reparability since different levels of plastic deformation were found to remain after exposure to a standard fire, parametric fire, or travelling fire. Resilience is becoming a popular topic in an engineering and business culture that is increasingly risk aware, and research needs were identified related to resilience as outlined below.
- A proposed framework for responsibly increasing the use of PBFD for structural fire design in Canada was identified as:
    - Competency must be demonstrated in both the designer and the AHJ
    - Regulation and peer review must be thorough and consistent
    - Holistic fire design must be taught in education
    - Implementation shall be incremental and transparent

7.3 Research Needs

As the author developed the alternative solution of Chapter 5, the literature was reviewed for relevant parameters and modelling assumptions. During that process it became clear that several aspects of modelling structures in fire still need further research and at best we can make conservative, informed assumptions in our modeling approaches. These research needs are listed below with the relevant section of the research presented herein that attempted to address them.

1. Identify and describe fire scenarios for the fire load case (Section 5.5);

2. Define failure with respect to structural stability and integrity (Section 5.6.1) and cost to repair/restore the structure (Section 5.9.4);
3. Understand how natural fires contribute to whole building response (Section 5.8.1 and Section 5.8.2, on the results of the travelling fire); and
4. Develop a consistent framework for PBFD and fire as a load case for structural design (Section 6.4)

These research needs have also been raised by NIST (2015) following a series of R&D workshops specific to fire research that were held in Europe and North America, as well as three white papers that were developed through industry consensus of experts in the field to cover the three predominant construction materials (steel, concrete, and timber). Research need (1) was partially met with the development of the design fires for the alternative solution, as well as a publication by the author introducing fire as a load case to the Canadian practitioner (Smith & Gales, 2016). Research need (2) was partially addressed with the introduction of the proposed acceptance criteria to be balloted within the ASCE Fire Protection Committee, while cost and scope to repair a structure after a fire remains a research need to address resiliency. Research need (3) also began to be addressed with the analysis and discussion of the alternative solution, however understanding structural behaviour for fire is an ongoing research need that will continue to evolve as more unique structural solutions are analyzed and more experimental test data becomes available. Lastly, the author put forth their proposed framework for PBFD in Canada to address research need (4). It is expected that this framework will evolve over time and can become more detailed as PBFD sees implementation.

In addition to the research needs that received attention by the author, there also exists research needs lacking guidance within the literature that the author encountered during the research presented herein. These include:

- Development of connection models or assumptions and their acceptance criteria (Section 5.9.2);
- Impact of connection assumptions on global structural behaviour (Section 5.9.2);
- Development of failure criteria for shear stud behaviour and fracture of steel reinforcing (Section 5.7);
- Development of temperature-dependent load-displacement behavior of shear stud connectors (Section 5.7);
- Impact of material differences between Cardington test series and the built designs that are validated off that test data (Section 4.5, Section 5.4);
- Effect of fire-structure interaction on spalling of fire protection coatings;
- Effect of structural response to tenability of enclosed spaces and human egress; and
- Develop post-fire damage assessment tools (Section 5.9.4)

7.4 Recommendations

The benchmark modelling in Chapter 4 revealed a series of future research needs and recommendations. The first recommendation was for transparency in published case studies to increase and all relevant parameters disclosed for peers to critique. In performing the benchmark modelling, it was found that not all required information was available from the literature and the author had to make assumptions. Some assumptions were relatively mundane, such as material properties assumed to be in line with those published in the literature. These assumptions are not expected to have a large impact on

the results. Other assumptions were much more important to the results such as the bracing of the compression flange. These assumptions had to be made posteriori to match experimental results and then rationalized that the authors likely had the same bracing of the compression flange but did not include in the literature. Likewise, there is a need for more experimental data similar to that used in Chapter 4 so that this validation and verification of models can proliferate.

After modelling the second Cardington Corner Test in Section 4.5, some differences were found between the materials and methods used in the model and contemporary construction in Canada as mentioned in Section 7.3. First, as mentioned in Section 4.5.4, the slab reinforcement in Cardington was smooth while contemporary Canadian construction typically uses welded wire fabric with ribs. Future research is required to determine the effect of this difference, particularly in the strains that develop over supporting beams from TMA. Implementation into finite element models will also require study as software packages such as SAFIR simply smear the reinforcement into an equivalent steel area throughout the shell element. As well, it was assumed that the steel beams and concrete slab had a complete shear connection in their composite action. Again, contemporary structures typically use partial-shear connections for economy given modern floor spans. Future research is required to determine if partial shear connection has an effect on the development of TMA. A recent update to SAFIR allows for spring elements which the author believes can be used to model partial shear connections, so this can perhaps be investigated numerically first. Experimental data will eventually be needed to validate models with partial shear connection that develop tensile membrane action as the author believes steel studs could start to become a failure

mechanism themselves if heavily optimized during ambient design. Lastly, Cardington made use of lightweight concrete while Canadian construction uses normal weight. Both the normal weight and lightweight concrete were assumed to behave similarly in their full-floor response since the differences in heat transfer can be accounted for and the mechanical properties can be derived using the Eurocode equations as a function of compressive strength. This requires further study but the author does not believe it will have a large impact on the results or conclusions of the alternative solution.

As well, the alternative solution developed in Chapter 5 demonstrated a behaviour where the composite floor, after deflecting downwards, began to bow upwards before reversing back to a downwards deflection. The author is not aware of any discussion to date on this behaviour and believes it is caused by differential heating and thus expansion between the concrete slab and steel beams, but it warrants further experimental investigation. Also in the alternative solution of Chapter 5, it was proposed by the author with support from the literature that providing ductility in the connection is enough to ensure large connection forces can be dissipated. Further investigation is required if the tensile membrane behaviour that develops is impacted by ductility at the connections, which could not be incorporated into the model with the software package that was used.

Resilience was an important discussion item for the alternative solution in Chapter 5, and represents a strong opportunity for implementing PBFD in practice for property owners, business owners, and developers that value business continuity planning. However, future research is required to better understand how expected performance to the fire limit state from a numerical model translates to real-world performance in terms of ability to be repaired and occupied, and the time and resource costs to do so. The first

steps in this resilience research have been initiated with a multi-year research partnership which builds on the lessons learned and precedents demonstrated by the research work presented herein.

The final recommendation is to begin the discussion around increased PBFD usage and acceptance in Canada by facilitating a workshop with industry, academia, regulatory, and fire service attendance. It is anticipated that this workshop will begin more broad communication which is critical for fire engineering to grow as a profession and increased collaboration amongst the attendees is expected to be a key outcome.

Appendices

Appendix A - Fuel Load Calculation

The fuel load of the compartment may be defined either from a fire load survey, or from a national fire load classification based on occupancy. For the alternative solution of Chapter 5, the latter was chosen and the fuel load density was calculated using the Eurocode formulations (CEN, 1991-2002) which is also referenced in the SFPE Handbook of Fire Engineering (SFPE, 2016). In a design scenario, the specific method for calculating the fuel load will need to be determined with all stakeholders and agreed upon in advance. The design value of the fire load, $q_{f,d}$, is defined in Eq.A.1:

$$q_{td} = q_{tk} \cdot m \cdot \delta_{q1} \cdot \delta_{q2} \cdot \delta_n \tag{A.1}$$

where

q_{td} = Design fire load

q_{tk} = Characteristic fire load density

m = Combustion factor (0.8)

δ_{q1} = Factor for fire activation risk due to size of compartment (Eq.A.2)

δ_{q2} = Factor for fire activation risk due to type of occupancy (Eq.A.3)

δ_n = Factor to account for different firefighting measures (1.0)

For an office occupancy, the average fuel load density is provided as 420 MJ/m^2 with a standard deviation of 126 MJ/m^2. The 80% fractile assuming a Gumbel distribution is 511 MJ/m^2 (CEN, 1991-2002).

$$\delta_{q1} = 1.54 \text{ (linearly interpolated between 250 m}^2\text{ and 2500 m}^2\text{)} \tag{A.2}$$

$$\delta_{q2} = 1.02 \text{ (linearly interpolated between 250 m}^2\text{ and 2500 m}^2\text{)} \tag{A.3}$$

Substituting into Eq.A.1, it is found that the design fire load is **642 MJ/m^2**

Appendix B - Parametric Fire Calculations

The Eurocode parametric fire (CEN, 1991-2002) was developed by fitting curves to graphical solutions developed by Pettersson using a heat balance of fire compartments (Wang et al., 2013). The base expression for the time temperature curve is given in Eq.B.1.

$$\theta = 20 + 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.427e^{-19t^*} \quad \text{(B.1)}$$

where

θ = Compartment gas temperature

t^* = Nonphysical time parameter, equal to $t\Gamma$

Γ = Dimensionless quantity, refer to Eq.B.5

The first step is to determine if the fire is fuel controlled or ventilation controlled. This is determined with the factors t_{max} and t_{lim}. The first, t_{max}, is calculated with Eq. B.2.

$$t_{max} = 0.2x10^{-3} q_{td} \frac{A_t}{A_v\sqrt{h_{eq}}} \quad \text{(B.2)}$$

where

q_{td} = Fuel load (refer to Appendix A)

A_v = Area of ventilation openings

A_t = Total area of compartment

h_{eq} = Weighted average height of ventilation openings

Next, t_{lim} is calculated based on the expected growth rate of the fire. These values are presented in Table B.1.

Table B.1: Fire growth rates for use with Eurocode parametric fire (CEN, 1991-2002)

Occupancy	Growth rate	T_{lim} (hour)
Public space	Slow	0.417
Classroom, office, hotel	Medium	0.333
Library, cinema, mall	Fast	0.250

For the alternative solution of Chapter 5, a growth rate of medium was selected due to the office occupancy of the space. To determine if the compartment is ventilation or fuel controlled, t_{lim} is compared to t_{max} from Eq.B.2

$$t_{lim} > 0.2x10^{-3} q_{td} \frac{A_t}{A_v\sqrt{h_{eq}}} \tag{B.3}$$

If t_{lim} from Table B.1 is greater than t_{max} calculated from Eq. B.2, then the fire is fuel controlled and:

$$t_{max} = t_{lim} \tag{B.4}$$

Otherwise, t_{max} from Eq.B.2 is used as calculated and the fire is said to be ventilation controlled. For the alternative solution of Chapter 5, t_{max} was calculated to be less than t_{lim} from Table B.1 for all ventilation sizes considered which meant the compartment had a fuel controlled fire. The dimensionless parameter Γ also depends on the fire being ventilation or fuel controlled, as shown in Eq.B.5 and Eq.B.6, respectively.

$$Ventilation\ Controlled: \Gamma = 8.41x10^8 \left(\frac{A_v}{A_t}\right)^2 \left(\frac{h_{eq}}{\rho c \lambda}\right) \tag{B.5}$$

$$Fuel\ Controlled: \Gamma = 8.41x10^8 \left(\frac{0.1x10^{-3} q_{td}}{t_{lim}}\right)^2 \left(\frac{1}{\rho c \lambda}\right) \tag{B.6}$$

For the alternative solution of Chapter 5, the compartment thermal properties were To calculate the fire temperatures, Eq.B.1 is evaluated at time steps up to t_{max}. At that time, the fire has reached its maximum temperature and begins to cool. The cooling phase of the fire is linear with time and is determined from Eq.B.7 to Eq.B.9, depending on the parameter t_{max} and t_{lim}:

$$\theta = \theta_{max} - 625(t^* - t^*_{max}x) \text{ for } t^*_{max} \leq 0.5 \tag{B.7}$$

$$\theta = \theta_{max} - 250(3 - t^*_{max})(t^* - t^*_{max}x) \text{ for } 0.5 < t^*_{max} < 2 \tag{B.8}$$

$$\theta = \theta_{max} - 250(t^* - t^*_{max}x) \text{ for } t^*_{max} \geq 2 \quad \text{(B.9)}$$

Based on the above parameters, the parametric fire curves were produced for a range of ventilation conditions. These time-temperature curves are reproduced below in Figure B.1, and can be seen in Figure 5.6 of Chapter 5.

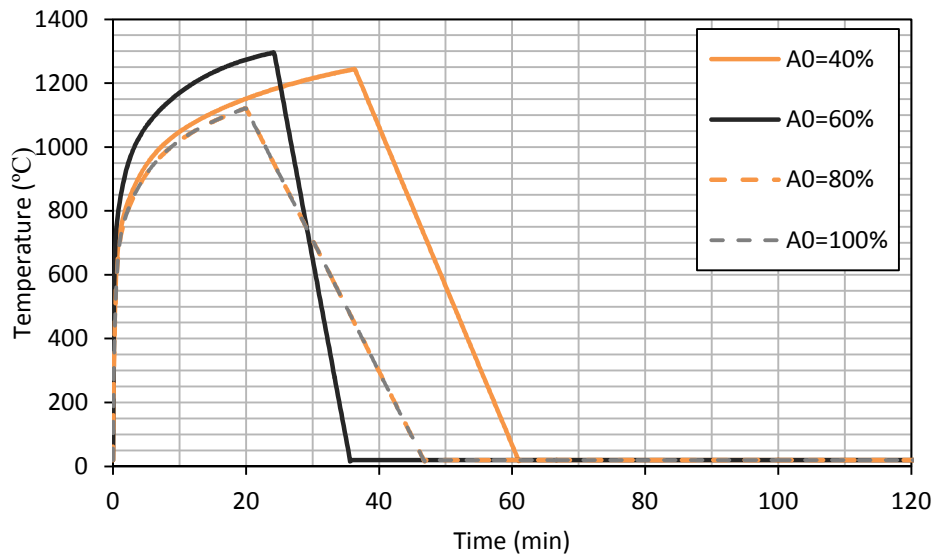

Figure B.1: Parametric time-temperatures curves

Appendix C - Travelling Fire Calculations

This appendix will discuss the formulation of the travelling fire as described by Stern-Gottfried and Rein (2012a; 2012b) as well as Rackauskaite et al. (2015). The equations presented herein are what were used to develop the design fires of Section 5.5.4 which were shown to govern some aspects of the response of the alternative solution in Chapter 5. The travelling fire methodology (TFM) is modular, and consists of four types of input. Within each input type, multiple variables may exist. The four aspects of the model are:

- Fuel Load
- Flame spread rate (travel speed)
- Near-field temperatures
- Far-field temperatures

The above parameters will be discussed further in the following sections. Additionally, it will be discussed why parameters common in other fire models, mainly ventilation and the compartment boundaries, are not included in this particular model. The benefit of the model being modular is that various aspects of it can be tweaked. This has already been seen in recent work related to the model, specifically for the near-field temperature predictions presented in the iTFM.

C.1 Flame spread

In the travelling fire methodology, the user first selects a fire size, L_f *(m)*. This is taken as a percentage of the floor area and is determined by the user. The fire size is generally described as a ratio of the total floor area. The compartment of the alternative

solution in Chapter 5 has a length of 63 m in the direction the fire travels, meaning a fire size of 10% would have $L_f = 6.3$ m. For a fire of size L_f, the travel rate is defined as:

$$\boldsymbol{s} = \frac{\boldsymbol{L_f}}{\boldsymbol{t_b}} \tag{C.1}$$

$$t_b = \frac{q_f}{\dot{Q}''} \tag{C.2}$$

where t_b is the local burning time of each node of the floor (s), q_f is the fuel load density (MJ/m^2), and $\dot{Q}''$ is the heat release rate per unit area (kW/m^2) (this is taken as the total heat release rate, refer to section C.3) (Stern-Gottfried & Rein, 2012b). The travel rate of the fire as well as the total burning duration is indirectly modified by changing the fire size.

C.2 Fuel Load

Fuel load density is determined based on the use of the compartment under consideration, defined as:

$$\dot{Q} = A_f \dot{Q}'' \tag{C.3}$$

where A_f is the fire length, L_f, multiplied by the compartment width w (m). In the initial TFM literature, the designer was advised to try all possible fire sizes from 1% up to 100% of the floor area, while it was suggested that special attention be paid to fires of size ~10% since these were found to be the most severe (Stern-Gottfried & Rein, 2012b; Stern-Gottfried et al., 2009). This range has been refined in the iTFM based on more realistic fire spread rates. For very large and very small fires, unrealistic travel speeds were calculated which had not been observed in any accidental or experimental fires. This is because the flame spread rate is directly related to the fire size in its current

formulation. To account for this, the iTFM proposed and upper and lower bound on the fire spread rate:

$$L_{f,min/max} = s_{min/max} \cdot t_b \tag{C.4}$$

where s_{min} is taken as 0.1 mm/s, which is representative of wood cribs in the open, and s_{max} is taken as 19.3 mm/s, which was the highest observed flame spread rate at the Cardington natural compartment fire tests (Rackauskaite et al., 2015). For these upper and lower bounds of flame spread rates, the model user will arrive at a maximum and minimum flame size to consider for a compartment.

C.3 Far-Field

The far-field temperatures decrease with distance from the burning region (the near-field). Alpert's empirical correlation (Alpert, 1972) has been used to describe this which gives temperature as a function of radial distance for an unconfined ceiling jet:

$$T_{max} - T_{\infty} = 5.38 \frac{(\dot{Q}/r)^{2/3}}{H} \tag{C.5}$$

where r is radial distance from the fire plume (m), H is the floor to ceiling height (m), T_{∞} is ambient temperature, $\dot{Q}$ is the total heat release rate (kW), and T_{max} is the temperature at distance r. When the above correlation was derived from experimental data, r was taken as a radial distance in the unconfined ceiling jet, although in the travelling fire model it is assumed that substituting x for r as a linear value along the compartment is acceptable. In Alpert's correlation, $\dot{Q}$ is taken as the total heat release rate since the experimental data used was based off alcohol pool fires where the radiative fraction is negligible. It has been assumed that this is true for the travelling fires in compartments as well, which could perhaps be a questionable assumption given that structural fires have

complex fuel sources and are likely to have a radiative component in their energy released.

Further, Alpert's correlation assumes an unconfined ceiling with no obstructions and no smoke accumulating. The obstructions requirements will have to be at the discretion of the modeler, while the smoke assumption was ignored in the pursuit of simplicity for the model. Because the model is modular, a more refined far-field correlation can be added to it at any time. The modeler does however need to remember the assumptions in the current TFM iteration and ensure that the compartment being modeled with a travelling fire has a relatively smooth ceiling in order for Alpert's correlation to remain valid. The last assumption, not explicitly stated in the TFM framework, is that Alpert proposed specific distances to differentiate the 'near-field' and 'far-field'. The equation provided above for the far-field is valid when $r/H > 0.18$. For distances very close to the boundary between the near-field and the far-field, the radial distance r needs to be checked to ensure that the equation is still within its range of validity. The TFM literature does not discuss the implications for small values of r such that $r/H < 0.18$, or propose an alternate equation for these cases.

In the iTFM equations, the far-field correlation was clarified but not changed. In the most recent literature, equations have been provided to calculate r as the distance from the point of interest to the centre of the fire. There is no discussion given to why this is done, however the assumption becomes questionable for very large fires where the centre of the fire is actually much further from the point of interest than the leading edge of the fire. Further, the equations are incomplete because they only hold true for cases where the point of interest is beyond (in front of) the fire as it travels. The first equation

presented below is from the iTFM literature, while the equation afterwards is a proposed addition by the author.

$$\text{For } \dot{x} - \frac{L_f}{2} \leq x \ \rightarrow r = x - \ \dot{x} + 0.5L_f \tag{C.6}$$

$$\text{For } \dot{x} - \frac{L_f}{2} > x \ \rightarrow r = \dot{x} - 0.5L_f + x \tag{C.7}$$

where x is the location of interest along the floor (m), and $\dot{x}$ is the location of the front of the fire at time t, equal to the flame spread rate 's' multiplied by the time. It can be seen that the second equation is the first equation multiplied by -1, since the midpoint of the fire has passed the location of interest. This is not explicitly stated in the iTFM literature, but is fairly obvious to anyone that begins using the model and notices errors in the equation when negative values are inserted to Alpert's correlation. A visual image of the far-field distance calculation is shown below in Figure C.1.

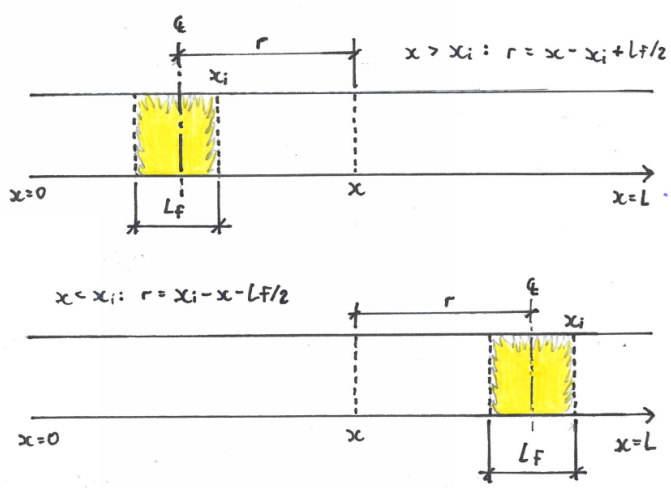

Figure C.1: Calculation of far-field distance to centre of fire, r; adapted from Rackauskaite et al. (2015)

It is the author's opinion that these additional equations for the iTFM only complicate the model, while the original TFM just stated r was the distance to the centre of the fire and let the user formulate these straightforward relationships.

C.4 Near-Field

The initial TFM assumed the near-field was dominated by the presence of flames and occurred over the full length of the fire, L_f. Out of simplicity it was taken as 1200°C, which was said to be the worst case of recorded near-field temperatures (Stern-Gottfried & Rein., 2012b). Since the model is modular, different portions can be substituted for alternative or revised correlations as they become available. This was the case with the near-field temperature correlation, which was refined in 2015 in the Improved Traveling Fire Methodology (referred to herein as iTFM).

Based on experimental observations, the correlation of the TFM was revised to consider a range of temperatures between 800°C and 1200°C. This range of observed temperatures was addressed in the initial TFM but no correlation existed to capture it. The iTFM addressed this by introducing the concept of a flapping angle in the near-field temperature calculation, shown in Figure C.2.

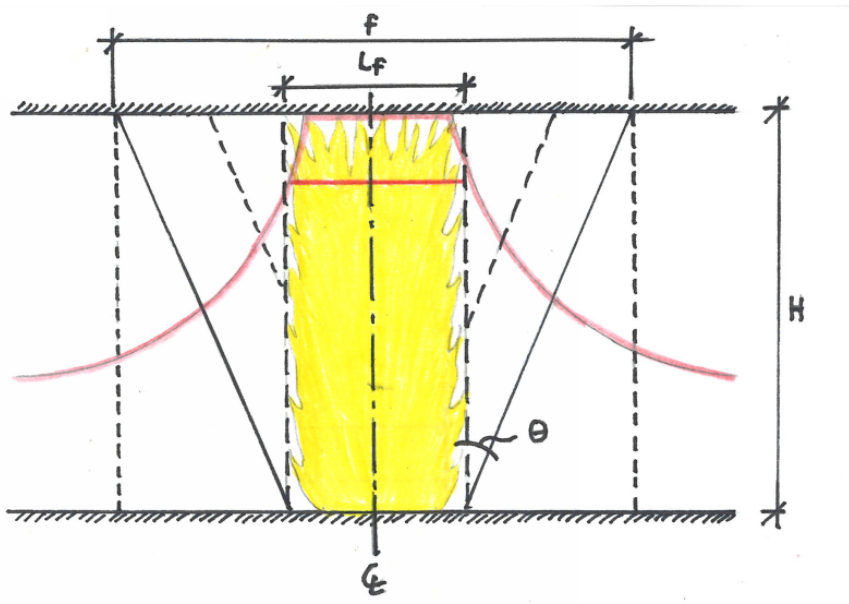

Figure C.2: Visual image of the fire plume with flapping length (f) and flapping angle (θ); adapted from Rackauskaite et al. (2015)

In this updated near-field model, the near-field temperature is taken as the average over the flapping length shown in Figure C.2. The temperatures are calculated along the flapping length and are limited to a maximum of 1200°C. This is calculated as follows (Rackauskaite, Hamel, Law, & Rein, 2015):

$$T_f = T_\infty + \frac{T_{nf}(2r_{x1}+L_f)-2T_\infty r_{x2}}{f} + \frac{32.28\dot{Q}^{2/3}}{H(f)}\left({r_2}^{1/3} - {r_{x2}}^{1/3}\right) \quad \text{(C.8)}$$

where T_f is a reduced near-field temperature, T_∞ is ambient temperature, T_{nf} is the maximum near-field temperature, L_f is the length of the fire, and H is the floor-to-ceiling height (m). The flapping length, f, was not given an equation in the literature. Intuitively, it was seen to be the fire length (L_f) plus a length on each side calculated using the flapping angle (θ) and the floor-to-ceiling height (H). The proposed Eq.C.9 was verified by calculating various reduced near-field temperatures for the examples provided in the literature (Rackauskaite et al., 2015) and perfect matches were obtained. It's exclusion from the literature explaining the methodology is questionable.

$$f = L_f + 2Htan\theta \quad \text{(C.9)}$$

The flapping angle is suggested to be taken as 6.5° based on experimental observations (Rackauskaite et al., 2015). The various 'r' variables in the above equation are used to represent the integration points of the near-field.

$$r_{x1} = max\left[0, r_o - L_f/2\right] \quad \text{(C.10)}$$

$$r_{x2} = max\left[L_f/2, r_0\right] \quad \text{(C.11)}$$

$$r_0 = \dot{Q}\left(\frac{5.38}{H(T_{nf}-T_\infty)}\right)^{3/2} \quad \text{(C.12)}$$

In the above, r_{x1} is the distance from the centre of the plume to the edge of the maximum near-field temperature (T_{nf}), and $r_{x2}$is the same point but used as the

lower bound for integration. The theoretical location of the 1200ºC boundary edge, r_o, is found by re-arranging Alpert's correlation and substituting 1200 ºC in for $T_{nf.}$ The reason that $r_{x1}$and r_{x2} are treated differently is because there are three possible crossing points for r_o relative to $r_{x1}$and r_{x2}. These are shown below in Figure C.3:

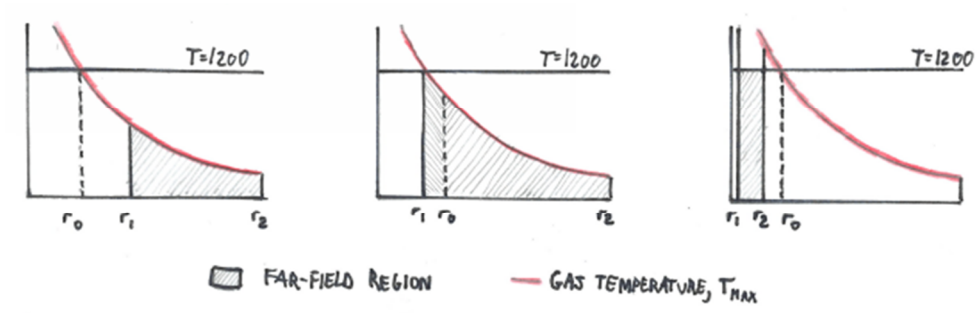

Figure C.3 Limits of integration used for reduced near-field temperatures; adapted from Rackauskaite et al. (2015)

As seen above, the integration used for the reduced near-field temperature extends beyond the flame length, L_f, to capture all of the temperatures within the flapping length denoted by r_2.

C.5 Ventilation

In TFM, ventilation is not an input parameter. This is because the specific underlying assumption of the model is that the fire is not ventilation controlled, nor does the compartment reach flashover. Addressing or confirming this assumption is not discussed in the TFM methodology, although it is assumed that a user would want to verify that the fire is indeed fuel-bed controlled for each fire size investigated.

C.6 Compartment Boundaries

Similar to ventilation, the TFM has no input parameters related to the compartment boundaries. The model calculates temperatures for the near-field and far-field based off

of the heat release rate of the fire, but does not account for any losses through the compartment walls, ceiling, or floor. Structural temperatures are calculated based off the adjacent gas temperatures with a subsequent thermal analysis, but are not included in any sort of energy balance to determine how much heat is lost from the compartment through the boundaries.

C.7 Resulting Time-Temperature Curves

Based on the above formulations, time-temperature curves along the length of the floor were developed for the alternative solution of Chapter 5 as shown in Figure C.4. These curves were applied accordingly to beam and shell elements along the length of the structure.

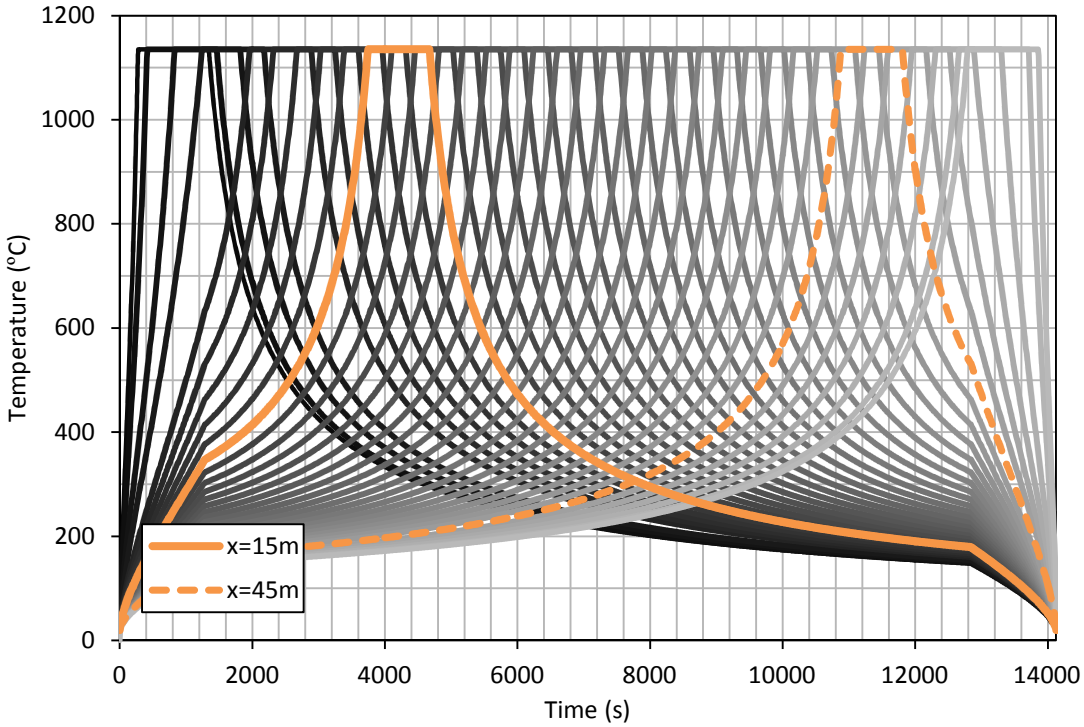

Figure C.4 Limits of integration used for reduced near-field temperatures; adapted from Rackauskaite et al. (2015)

Appendix D - Published manuscript: Structural Fire Design for Composite Steel Deck Construction in Canada[2]

Authors: Matthew Smith
John Gales
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[2] Manuscript is included in its original format with references inclusive.

ABSTRACT

In Canada, there has been a dearth of research and design attention given to developing innovative fire design solutions for composite steel deck construction. This is despite the increasing number of these constructions in densely populated metropolitan centers in Canada. These structures are currently treated with classic prescriptive based fire designs. Isolated building components are designed based on implicit fire relations and as international practice has shown, prescriptive fire design may be viewed as restrictive, uneconomical, and is challenged in its representation of reality. Internationally, practice has been maturing towards Performance Based Fire Design (PBFD) for composite steel deck structures for the last several decades with significant research into the behavior of these structures occurring. Herein; the first objective is to examine the development of PBFD techniques used globally through relevant experiments, experiences, and applications. The second objective is to explore the state of fire design within Canada and what can be learned from global fire design and research experiences – the international state of the art. The goal is to promote best practice discussion on where structural fire design for composite steel deck construction in Canada, and internationally, could move towards.

INTRODUCTION

In Canada, the treatment of fire protection is predominately prescription based. The National Building Code of Canada (NBCC), contains acceptable solutions that, when followed, are intended to provide adequate performance under fire conditions. These assemblies are tested individually under standard fire conditions and given fire ratings that indicate the time to failure, with failure being when the stability, integrity, or insulation criteria are met and exceeded. As far back as 1898, however, the importance of analyzing full building response to fire was understood when pioneering fire engineer Himmelwright stated, "the actual and relative expansion of the materials due to heat and the deflections caused by unequal heating must receive careful consideration…The limit of safety is in some cases dependent upon temperature and in other cases upon expansion" [1]. The objective of this paper is to investigate the development and application of Performance Based Fire Design (PBFD) in a global context, before shifting to an examination of the state of PBFD within Canada as it relates to the design of

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composite steel deck construction. Composite steel deck construction has been selected due to the abundance of the construction method in Canada, coupled with the advances seen globally as it relates to PBFD. The authors present a preliminary framework that builds on lessons learned to advance PBFD in Canada in a responsible and efficient manner.

DEVELOPMENT OF PBFD

Prior to the early 1990's, fire research, design, and construction focused on single member response to fire as made evident through the state-of-the-art design guidelines available. The guidelines, dating back to 1976 with Sweden adopting the "Rational Fire Engineering Method" proposed by Petterson in the 1960's, detailed how to calculate steel temperatures through a heat transfer analysis and related the elevated steel temperature back to a material strength for use in design [2]. In North America, approved fire solutions are related to ratings from the ASTM E119 fire test. This test includes a "steel failure temperature" that was to be met by the approved solution, but was also imposed by the Authority Having Jurisdiction (AHJ) on any performance-based solution with the structure not allowed to exceed it. Seigel proposed that the limiting temperatures were introduced by ASTM and Underwriters' Laboratories as an additional end-point criteria to the E119 standard since it was understood that the fire endurance tests were not representative of the assemblies in their as-built condition, both in terms of arrangement and loading [3]. It does not represent when steel fails as that has been shown to be dependent on arrangement, restraint, and loading [4]. It was not until 1990 that whole building response was demonstrated and testing started to evolve in this direction.

Broadgate Fire, 1990 [5]

In 1990, a 14 storey building under construction in Broadgate experienced a fire that lasted over 4 hours showing no sign of structural collapse. At the time of the fire, the sprinkler system and fire protection was incomplete. This was one of the first opportunities to analyze the effects of fire on a full-scale, contemporary steel building. An investigation into the building's response to the fire concluded that the structure performed well due to load redistribution. Even without fire protection measures in-place, the structural damage was only 6% of the total cost from the fire. The conclusion of the investigation stated "it would be worthwhile in the future to investigate the effects of major fires in significant structures to gain a better understanding of the most important mechanisms in practice". Indeed, the momentum towards contemporary PBFD started here.

Cardington Steel Tests, 1996 [6]

Following the Broadgate fire of 1990, a series of large-scale, non-standard, full structure tests were performed in Cardington, UK. The structure considered was an 8 storey, composite steel building. In total, 7 separate tests were performed at different times, with different combinations of protected primary members and unprotected

secondary members. Bisby et al., provide a succinct summary of each of the tests and the key observations as taken from the available literature. Across the 7 tests, all of the studied members showed significant deflection ranging from 180 mm to 1200 mm with "no signs of collapse". The test members had experienced local failures, including beam buckling, shear-plate fracture (upon cooling), and cracking of the concrete slab. Despite the local failures observed, a global structural failure was not observed. The observations related to PBFD were:

- Load redistribution of a real structure is possible with tensile membrane action. In this condition, the secondary beams are allowed to fail and the load support mechanism is tensile forces in the floor plate resisting gravity loads and supported by protected primary members;
- With tension membrane action, additional forces are imposed on the beam-to-column connections, as well as within the plane of the composite floor deck at the primary beams. This was shown through beam-to-column failures and significant cracking of the concrete over the supporting beams where reinforcing mesh had been missed in the construction of the test assembly; and
- The importance of cooling was demonstrated in Tests 1 and 2 as connection failure was observed through increased load from structural contraction.

The fire resistance of the floor plate was shown to be much different than, in this case greater than, the fire resistance predicted from single member standard furnace tests [6]. The proposed methodology for unprotected secondary beams in a composite steel-framed building formed the basis for further testing [6] and has now permitted model verifications to help apply PBFD for real structures.

APPLICATIONS of PBFD

Load carrying mechanisms exist in real structures that cannot be demonstrated by the single member fire tests. Successful applications of PBFD will be discussed briefly. The first case study uses single member analysis and subsequent testing to verify reserve member capacity, the second refers to whole building response made possible by the test data from Cardington, potential advances are then discussed.

Case Study #1: 600 Grant Street, 1971 [7]

This 64 storey office tower in Pittsburgh contained exposed structural steel columns filled with water to keep temperatures below a certain threshold in a fire. The fire design of this time period revolved around single member analysis and did not take into account the global behavior of the structure. The design was "based on performance" and used heat-transfer rate equations to determine the temperature on the surface of the exposed structural steel. It was rationalized that the internal surface of the steel would not exceed the boiling point of water, and thus an average temperature across the cross section of the steel could be calculated. The maximum computed temperature of the steel columns with 4 hour standard fire exposure was 337C, which was compared to the 538C maximum temperature permitted by E119 [7]. Seigel states that structural integrity is assumed to be maintained as long as the steel is not heated to a level that reduces its strength below

design loads [3]. This case study demonstrates how PBFD had previously been applied to steel columns, however they are just a single element in an entire composite steel-framed structure.

Case Study #2: Mincing Lane 2006 [8]

An Arup case study of a composite steel deck system structure provides a stark difference in design approach. The first advance in design methodology in this case study demonstrates the extent of analysis. Using Abaqus, the entire floor plate was analyzed for two different fire protection approaches. First, the building was analyzed using fire protection that met typical prescriptive code requirements. Next, secondary beams were left unprotected and the analyses were run again. The goal of this was to demonstrate analytically that the protection of secondary members was redundant. The second advance in design methodology was the design fires. Three fire scenarios were designed for: a "short-hot" fire where most of the glazing is assumed removed from the structure, a "long-cool" fire which is assumed ventilation controlled, and 90 minutes of standard fire exposure [8]. The 90 minutes of standard fire is actually not representative of a real fire, however many are accustomed to its use from its longevity, including AHJs. Its use to compare a prescriptive design to a PBFD gives a frame of reference for those familiar with it. The acceptance criteria used was to avoid structural failure (identified by runaway deflections, as opposed to a limiting steel temperature) and to maintain horizontal and vertical compartmentation (by ensuring connections at the vertical shaft wall did not fail). A direct relation to the stability and integrity criteria can be seen, albeit adjusted in practice to accommodate a whole-floor analysis in lieu of a single member. The results of the global analysis with secondary beams left unprotected were that runaway deflections did not occur, however strains in the rebar of the concrete directly over supporting beams were around 2%. As a result, a connection model was developed to ensure the support had adequate capacity to allow the development of membrane action. The resulting deflections of the floor plate for the PBFD were shown to be similar to the model with prescriptive fire protection throughout. Having these models, and being able to show that the PBFD behaved quite similar to approved methods, helped in the approval of the design by the AHJ.

Looking Forward

A further observation made during the Cardington tests was a spatial evolution in time of the compartment temperature [9]. This behavior was further demonstrated in the Dalmarnock test series where the increased instrumentation density allowed for spatial distribution of the temperature profile to be analyzed [9]. Moving forward, the design of structures for fire can take into account actual flame spread (travel) within the compartments. This may be of considerable importance for large open office plans where uneven heating of the structure across bays with time, caused by travelling fires, could have detrimental effects on the performance of the structure. An international research program has been initiated [9], which at the time of writing has demonstrated shortcomings of the compartment fire framework where the range of validity is exceeded.

Advancing that framework further to a stage that it can be used more efficiently in practice with test verification may be a next step for advancing PBFD internationally.

CANADIAN MOMENTUM FOR PBFD

In 1994, the Canadian Commission on Building and Fire Codes (CCBFC) formed a task group to develop a long-term strategy for Canada's building and fire codes [10]. At that time, Canadian codes were prescriptive, and one goal of the task group was to evolve them. What resulted was Canada's introduction of the world's *first* objective-based codes in September 2005. The reason for an objective-based approach as opposed to performance-based is because the CCBFC feared that performance-based might create an "anything goes" situation as had been seen globally where poor regulation and implementation accompanied PBFD [11].

Canada's objective based codes contain three divisions within them: Division A provides the scope and definitions, Division B lists acceptable solutions, and Division C is for administrative requirements. The acceptable solutions in Division B automatically satisfy all requirements of Division A, however an 'alternative solution' requires the builder, designer, or building owner to show that it will perform the functional statements of Division A as well as the 'acceptable solution' it replaces. Where it becomes difficult, is in showing equivalence since not all acceptable solutions in Division B have their level of performance in quantitative terms. Objective-based codes are hence "benchmarking" and it must be shown that the alternate solution performs at least "as well as" an acceptable solution according to the functional and objective statements for that structural assembly.

In Canada, there is currently a trend towards PBFD solutions, but for now the objective-based solutions are guiding designers and decision makers in that direction with a few key differences. At this stage, not all acceptable solutions frame their performance in precise terms; but they all contain an inherent level of performance within them that represents society's expectations of building performance to fire [11]. The critical next step in Canada towards full performance-based fire solutions is to develop verifiable performance criteria for the acceptable solutions of Division B. This could be dependent or independent of the acceptable solutions. The National Research Council (NRC) currently has research projects aimed at quantifying the level of performance current acceptable solutions provide with the aim of using this approved performance as the criteria for true performance-based solutions [11].

An eventual shift could have benefits to the building community at-large as objective-based solutions are based on showing equivalency to approved solutions, many of which were tested individually under unrealistic standard fire conditions to meet the requirements of E119 (or equivalent). With objective-based codes, the benefits of analyzing whole building performance, as demonstrated globally and successfully implemented to real building through PBFD, cannot be realized.

Fire Safety Design Practice today in Canada

In Canada, most building fire design follows the prescriptive based approach from NBCC. We are, however, beginning to see PBFD solutions in the area of building egress. Referring to Division B, we see for example that the travel distance to an exit cannot be more than 40m for a business occupancy. The objective of this clause is to limit the risk of delayed movement (OS3.7), with the function being to facilitate timely movement during an emergency (F10). Instead of meeting the prescriptive travel distance of 40m that the acceptable solution states, practitioners have begun to calculate ASET and RSET to show that the available time is greater than the required time for evacuation. In doing so, the objective and functional statements of the approved solution are met and the alternate solution is shown to perform as well as the approved solution.

With regards to composite steel deck construction, it can be seen that existing practice already lends itself to the PBFD approaches being used in practice elsewhere. Typical good construction practice of composite steel deck calls for additional steel mesh to be placed over primary supporting beams. This mesh was shown to be crucial in the Cardington tests [6]. Further, computer analysis has shown the importance of this mesh in developing and maintaining tensile membrane action when and if the secondary beams 'shed' their load [8]. When looking at current floor plans of modern construction, a recurring pattern of primary beams with secondary beams on roughly 3 m centers are typical for an economical design. This same arrangement is what the case studies have shown, since aspect ratios of 1 to 2 for each bay may allow for tensile membrane action to develop provided the primary beams and columns are protected.

The mentioned egress example was for a case where the prescriptive approach is stated in quantifiable terms. The difficulty in Canada for composite steel decks is that the acceptable solutions do not have their performance quantified. To help new solutions be incorporated into construction, the Canadian Construction Materials Centre, offers an evaluation service that will assess a product's performance and compare it to a minimum acceptable solution from Division B. This does not guarantee that the product will be approved, but it does support the objective-based approach by showing equivalence and could help the AHJ, specification writers, design professionals, and builders determine approval or acceptability [11]. It is unclear if this approach can extend to an entire building, for example to show if unprotected steel members in composite steel deck construction can demonstrate as equivalent to an entire floor of steel beams protected with an acceptable solution, however this is an approach put forth by the timber community to show equivalence between timber construction and current prescriptive solutions.

Wood

Due to limitations within the Canadian building code, the authors of "Technical Guide for the Design and Construction of Tall Wood Buildings in Canada" [12] have proposed using the objective based-codes on the basis of equivalent risk for tall timber structures. This rationale is a risk-based approach for a building of combustible construction that could demonstrate the overall level of safety and risk that a noncombustible structure of acceptable solutions would have. This equivalent risk level

would translate into equivalent performance allowing the alternative solution to meet the objective and functional statements of the acceptable solutions. The objective-based approach would extend to the entire building such that equivalent performance is shown for the complete timber structure that would be provided by the group of acceptable solutions compiling the entire noncombustible, code compliant structure [12]. The proposed tall timber structure would be a partially encapsulated solution, meaning that the timber is partially protected from the effects of fire. The goal of this is to ensure that the structure itself does not contribute to the fuel load of the room until flashover. Once flashover has occurred, the fire typically shifts to a ventilation controlled condition where available oxygen controls burning. Once this occurs, the effect of a timber structure contributing to the fuel load is thought not to be as pronounced [12]. However, this means that the fire may have prolonged burning. To compensate, additional fire protection measures would be implemented so that performance is unaffected, such as improved sprinkling. The end goal is for the overall risk to be equivalent to an acceptable solution structure using a risk analysis under different fire scenarios. The risk assessment could be qualitative or quantitative so long as it demonstrates equivalency using accepted methods. If the steel composite deck industry could learn from the timber industry proposals, the overall risk of the structure due to fire could be considered without the focus being on single member behavior as the current application of the objective-based codes have trended. Additionally, there would be a need to compromise to ensure equivalent performance of the overall structure. In the case of timber construction, sprinkling demands increased. In the case of steel composite construction, connections will face increased loads to allow tensile membrane action to develop, sprinkling requirements may be increased, and additional reinforcing of the concrete floor may be required.

DISCUSSION TOWARDS A FRAMEWORK IN CANADA

The authors suggest three main aspects necessary for a proposed framework in Canada to move towards a viable PBFD implementation for composite steel decks. The proposed framework herein must be: responsible; mindful of the Canadian regulatory landscape; and draw successfully from international lessons.

To advance PBFD in Canada, competency must be shown to ensure the practitioners are experienced in the fields of structural engineering and fire engineering. In particular, the interaction of fire with the structure must be understood. Current fire education offerings in Canada appear limited. In particular, Fire Resistance – not Fire Design – is prevalent. Confidence of the AHJ should be increased by ensuring designers are competently trained in both structural and fire engineering. When undertaking a PBFD, the structural engineer must be prepared to accept responsibility for the design since it will be integral with the stamped structural drawings. This accountability can help ensure structures are fire designed in a responsible manner, as is currently standard practice in ambient design.

As PBFD strategies begin to be implemented in Canada, it is expected that the AHJ will initially have understandable reservations. It is proposed that incremental advances in PBFD be implemented as level of comfort increases. The first steel concrete composite designs are expected to optimize fire protection placement on secondary members

(though not omit). The mesh reinforcement that is necessary for tensile membrane action of composite structures is already best practice, but its importance will be highlighted in a fire design. The NBCC currently contains a plethora of acceptable solutions which can be used to benchmark whole building performance with the aim of providing a reference point for the AHJ. Multiple design fires can be rationalized, however eliminating the standard fire completely from the discussion will be difficult given how engrained it has become in current fire testing, approval, and implementation.

Lastly, Canada has the unique advantage of being able to learn from previous international experience as well as other international efforts underway to further develop PBFD practice. In Canada, the regulation of PBFD must be thorough and consistent from the beginning. A minimum acceptable level of peer review, both by the AHJ itself and third parties, must be agreed upon by all stakeholders to ensure no engineered fire design receives less due diligence than any other. As competency in structural fire engineering grows and is demonstrated within Canada, the pool of qualified, local peer reviewers will grow. Canada, as with all countries, will also benefit from increased testing, test data for model verification, and published case studies to spread knowledge and lessons learned.

Any application of PBFD must be done in the pursuit of improved building performance and best practice. It is well documented in the literature that the standard fire is not representative of reality: PBFD could aim to improve this. Although economic and sustainability benefits may be realized, these should not be the only drivers for deviation from the approved solutions which have historically proven satisfactory. As qualified engineers begin to assess a building for fire more realistically, building performance measures can be improved and Canada will see increased resilience of its structures.

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Appendix E - Published manuscript: Integrating Fire as a Load Case with BIM[4]

Matthew Smith [1] and John Gales [2]

[1] Graduate student at Carleton University, Canada, and Structural Engineer at Entuitive, Canada

[2] Assistant Professor in Fire Safety Engineering, Carleton University, Canada

Over the past several years, the consideration of fire to the structural performance of buildings has seen more inclusion in the design process. To this extent, a CISC supported research project (entitled, "Towards a Performance Based Fire Design Framework for Composite Steel Deck Construction in Canada") was undertaken by the authors with the goal of developing a framework for performance-based fire design for composite steel deck structures within Canada. This applicable research is a collaboration of academic, consultancy, and industry interest. The work is drawing on the lessons learned internationally and the various precedents that have already been established in areas of Canada by others. This project will be assessing: the required level of competency across all stakeholders; critiquing the education system that supports performance-based design within Canada (and abroad); performing novel testing of steel sections to begin to quantify the post-fire state that informs business continuity and resiliency of the building; and developing case studies to demonstrate and highlight the range of design options available. To highlight recent progresses towards these goals, the authors present a short example of novel tools and technologies being employed in this project. These tools specifically deal with conceptualization of treating fire as a load within the design process.

Fire as a load

There has been a recent shift towards including fire as a load case on structures, specifically internationally where many structures see their fire performance quantified and considered in design – the overarching goal of this CISC research. Examples of fire as a load can be seen in Annex K of the steel design standard CSA S16-14, as well as Appendix 4 of ANSI/AISC 360-10 that are both entitled, "Structural Design for Fire Conditions". Both of these material design standards now include provisions for the structural engineer to consider the effects of fire, albeit in an introductory manner. This can orient the engineer with performance based fire engineering (as discussed in Advantage Steel issue 39, "Fire Protection of Steel Structures"). The "SFPE Engineering Guide to Performance-Based Fire Protection Analysis and Design of Buildings" has also been previously referenced in Canadian case studies to help guide the process. Within CSA S16-14, a load case for fire is provided and contains the effects caused by the design basis fire as T_s. The design basis fire is discussed in the annex as being due to either a localized fire (non-flashover), or a post-flashover compartment fire. Most of the previously published Canadian case studies have focused on using Computational Fluid Dynamics (CFD) models, such as Fire Dynamics Simulator, to calculate temperatures in the structure for a given fire scenario. While the results and the trends displayed can be insightful, this is not the only tool available and can be quite resource intensive if fire is to be considered as a load case in structural design. Indeed, there needs to be a *range of tools* available to the design depending on the complexity and boundaries of the problem. This is similar to what we see in the rest of structural engineering. In many structural engineering practices Building Information Modelling (BIM) is used by default on projects because of the benefits it brings. This technology is often leveraged to streamline the creation of structural analysis models. Fire Engineering, when fire is considered as a load case, can benefit from this same workflow integration and utilize the information already being captured in the BIM. As the case studies were developed as part of this CISC research project, it is natural to develop links with BIM software to demonstrate synergy with the standard design processes being seen in practice.

Integration with BIM

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[4] Manuscript is included in its original format with references inclusive.

Industry best-practice uses BIM to increase coordination, efficiency, and document clarity. When used, these models are typically started at the very beginning of a project. These models contain both the structural and architectural information, among other disciplines, which means many of the parameters that can be used to define design basis fires are already being modelled (see Figure 1).

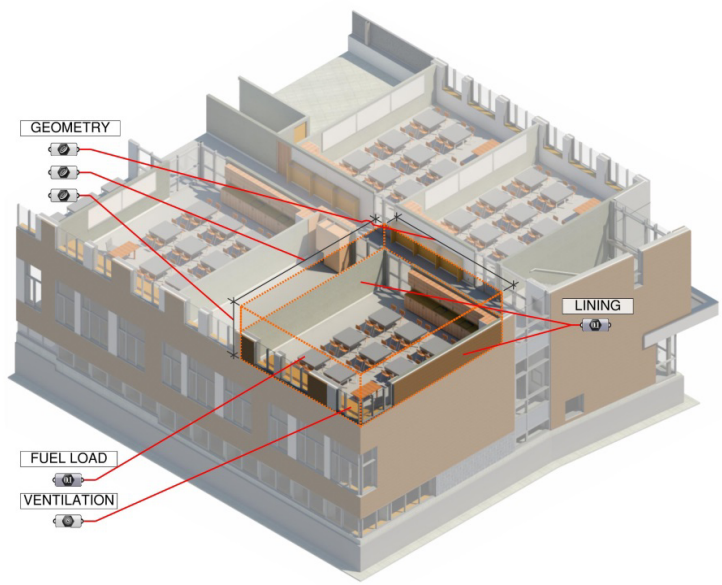

***Figure 1**. Sample BIM showing parameters that can be used for input to design basis fire calculations*

Eurocode Parametric Curves

A time-temperature curve that describes compartment fires and has often been used for design is the Eurocode Parametric Curve (CEN 1991-2002). That design fire is based off heat-balance calculations of average sized compartments. The input to this equation includes the compartment geometry, interior finishes, ventilation conditions, and fuel load. A workflow can be developed that efficiently extracts this information directly from the BIM and calculates the temperatures resulting from a design basis fire for each compartment within a structure (Figure 2). The calculations can be performed by and optimized within Grasshopper by powerful optimization plug-ins such as Galapagos or Octupus. Grasshopper is a graphical programming interface seeing increased usage in structural design that allows for generative algorithms to drive the 3-D modelling capabilities of Rhinoceros. It allows for complex geometry to be parametrically modelled, optimization and form-finding exercises to be run, and direct two-way linkage with structural analysis software. It is an ideal tool to make complex steel structures a reality, from both an architectural and structural perspective.

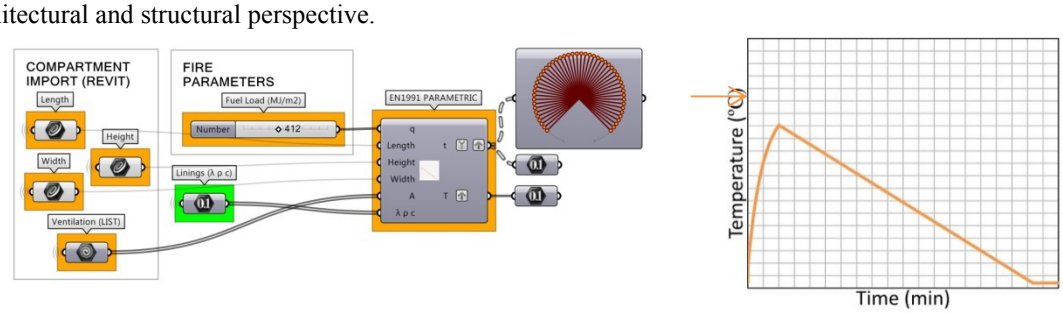

***Figure 2**. Simplified implementation of EN1991 Parametric Curve, using compartment geometry read from BIM and calculations subsequently performed in Grasshopper. Calculations are directly driven from BIM.*

Performing compartment fire calculations with direct input from the BIM allows for efficient analysis of the effects of temperature and can highlight areas that require special attention from the

structural engineer, all without the complexities and resources that a CFD model could require. However, it has to be recognised, there will still be complex cases that exceed the limitations of the analytical correlations and could require a more robust CFD model.

Travelling Fires

In recent years, consideration has been given to fires which do not engulf the compartment homogeneously and instead travel from one end to another. This behaviour is supported by observations of how accidental fires behave, as well as experimental observations. It is a necessary consideration in design since contemporary buildings typically have large open spaces and other attributes that can fall outside the validity of the Eurocode Parametric Curve. An analytical method for determining the effects of a travelling fire have been well developed and documented by Rein et al (2007), Gales (2014), and Rackauskaite et al (2015). The method itself has been used for steel design in Europe, although it is often not the only design basis fire considered. The travelling fire methodology is well suited to being integrated with BIM software since it is heavily dependent on the building geometry and properties. An example of a travelling fire moving through an open compartment is shown in Figure 3.

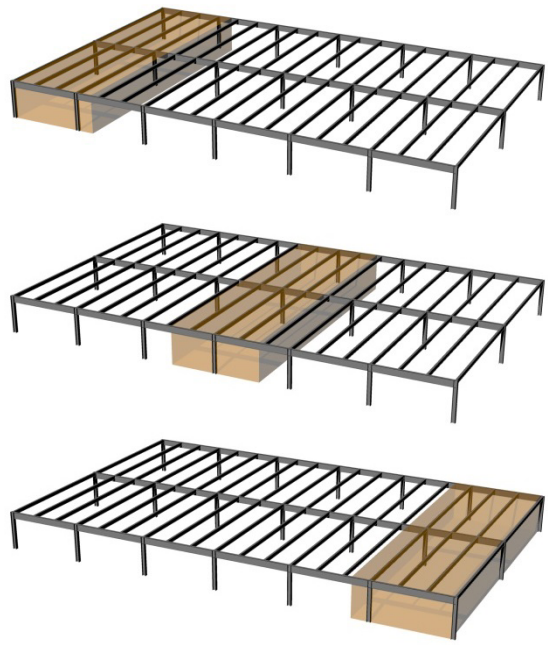

Figure 3. *Visual representation of a travelling fire (equal to 20% of the floor area) within a sample structure*

The benefit of powering the travelling fire calculations directly from BIM is that new geometry is automatically captured and actual structural steel member sizes can be included in the heat transfer calculations. By utilizing Grasshopper to perform the calculations, there is also an opportunity to quickly assess ranges of parameters such as fire size, travel speed, flapping angle, etc. This proposed workflow is presented in Figure 4.

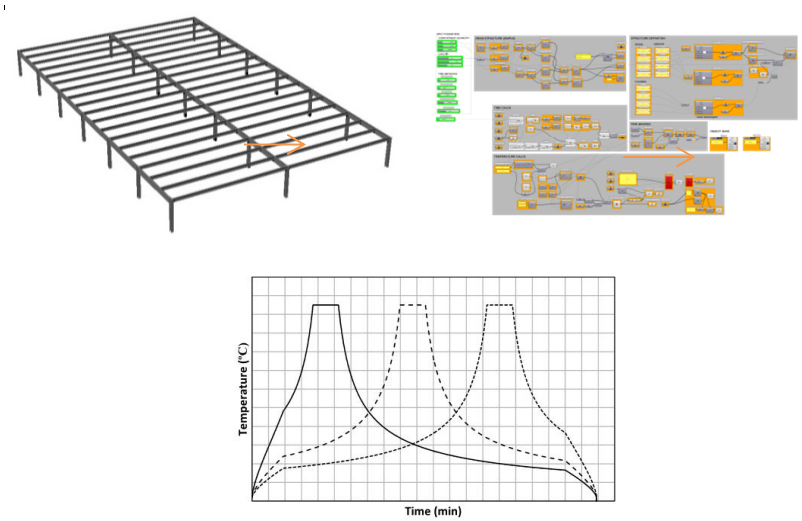

***Figure 4**. Sample workflow of BIM parameters imported to Grasshopper, with temperatures output for a range of design basis fires and locations of interest.*

Preliminary conclusions

As fire begins to see consideration as a load case in structural design in Canada, there will have to be a range of tools considered that vary in complexity, scope, and limits of validity. Analytical correlations such as the Eurocode parametric curves and Travelling Fire Methodology represent opportunities to calculate expected structural temperatures for a range of fire scenarios which can inform a performance-based approach. By using the BIM of a project to provide direct and real-time input with the fire calculations, similar to what we already see in structural design, efficiencies can be found and fire itself will see better inclusion as a load case in structural engineering in Canada and abroad. As fire engineering matures as a profession within Canada it is expected that more links will be created with the structural engineering workflow so that the actual performance of our structures can be better quantified and accounted for in design. Better defining how the performance-based fire protection opportunities can be integrated with the structural design is an essential part of this CISC research project, and developing tools to define the design basis fires of real buildings will allow for demonstration case studies to show what benefits can be realized for Canadian practice. Ensuring these tools are validated, practical, and well integrated will make the performance-based process more appealing and transparent to all stakeholders.

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Appendix F - Submitted manuscript: Developing Fire Safety Engineering as a Practice in Canada[5]

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Abstract

Fire Safety Engineering (FSE) can be defined internationally as a profession in which the goal is to design a fire safety strategy which optimizes human safety and property protection. This is delineated as opposed to a solution in which prescriptive protection is simply specified. In comparison to international practice, Canada could be considered highly underdeveloped in a technical perspective. In Canada FSE is a sub skillset within civil and mechanical engineering, rather than treated as a unique engineering profession in and of itself. With a requirement for more complex infrastructure to meet Canadian societal needs, there is unique stimulus that is fostering a demand to create unique professionals educated with FSE skill sets. This literary study therefore aims to present the state-of-the-art in the design practice of FSE in Canada and abroad with explicit focus on Performance-Based Fire Design (PBFD) for the Canadian practitioner who seeks to develop their expertise in this subject. Herein, the authors aim to provide background information regarding PBFD, education and practice. Details of PBFD are provided on fundamental concepts, strategies and potential uses in infrastructure. Finally, an insight into the future of FSE sheds light on the forthcoming opportunities for Canadian practitioners in this field.

1. Introduction

The goal of fire safety engineering (FSE) has always been to design a fire safety strategy which optimizes human safety and also property protection. In Canada, our engineering community does not need to look very far to see the challenges our society is currently facing with respect to fire safety related to our infrastructure. In the last five years there have been multiple severe fires, including the Fort McMurray wild fire, the L'Isle Verte care home and Kingston's 2013 conflagration. In particular, a surge of complex structures has become the norm as our society is rapidly undergoing a significant shift to urban populace, relying on novel building technologies and material optimization strategies for which are pushing the boundaries of our existing building codes (see Figure 1). These issues have necessitated a more holistic view of fire safety which may, in turn, warrant a discussion of our treatment of FSE in Canada.

The purpose of this state-of-the-art study is to create awareness of novel FSE concepts used internationally and discuss potential growth for the Canadian design practice. This report serves to begin the discussion.

Herein, an introduction to performance-based fire design (PBFD) is provided. A review of the fundamental and supporting FSE concepts, strategies and benefits as seen historically and internationally is provided. The study aims to provide a review of exemplar FSE projects

[5] Manuscript is included in its original format with references inclusive.

developed at leading international engineering firms. A summary of some best-suited project types is provided. Finally, an insight into the future of FSE sheds light on the forthcoming opportunities for Canadian practitioners in this field.

Recognizing that FSE in a Canadian design context is relatively novel, this study concludes with an appendix that provides key terminologies and acronyms to facilitate easy access for the practitioner.

Figure 1. In Toronto alone, there are now over 25 complex and highly optimized buildings that are 50 stories or higher. Nearly 15 are under construction and nearly 30 are approved and/or proposed. Additionally to accommodate the urbanization significant and complex infrastructure is proposed. Each engineering project may have the potential for application of advanced FSE to facilitate safe best practice benefits seen globally (Author's photo).

2. Background

As FSE is a relatively nascent field, this section aims to provide the reader with sufficient background on its evolution. A description of PBFD within the Canadian context is given as this is the basis of Fire Safety Engineering. Additionally, information on the condition of FSE education and an introduction to FSE practice around the globe are provided.

○ *Performance-Based Fire Design*

With prescriptive based fire design, infrastructure is generally understood to be safe. However the degree of safety provided may not be rationally understood in all cases, and may vary from application to application. PBFD is an alternative method of designing to meet the objectives of the building code, without expressly using the limiting prescriptive clauses and requirements. Speaking specifically to fire, the code clauses developed are all-encompassing, applicable to a wide range of structure sizes and functionalities, which could be inhibiting in a unique design scenario. In many cases, the fire safety strategies outlined in the code could be conservative and may result in a highly overdesigned fire safety strategies, causing unnecessary costs to the client and users. These costs may come in the form of excessive fire protection, loss of profitable floor area, or sacrifice of original architectural vision. In other cases, with detailed analysis, the prescriptive code provisions may prove to be insufficient for increasingly complex and unique buildings. However, currently a high level of fire analysis and modelling to support an alternative design is required for regulatory approval. As best said in 1995 by pioneering PBFD international experts Margaret Law and Paula Beever: "The desire of regulators to have simple rules and tests for administrative convenience contrasts with the need of designers to have maximum flexibility in order to arrive at optimum solutions. The magic numbers embodied in regulations are accepted without question, while any engineering solution is subject to a disproportionately high standard of proof [1]."

The use of PBFD solutions requires a deep understanding of FSE. Careful attention to the fire protection effectiveness during the design, construction, and management phases is critical. Any

mistake or omission could result in the failure of the whole fire protection strategy. The British Standards Institution has developed BS 9999, a code for PBFD [2], in which it is stated: "It has been found in practice that designs can frequently be compromised due to incorrect or poor installation, substituted materials or products, missing materials or products, lack of integration of active systems, inadequate inspection, lack of full commissioning, abuse during normal use of the building, inadequate maintenance and/or testing, and problems resulting from inadequate management documentation and training[2]." Due to this fact, it is crucial to develop a comprehensive design that is integrated flawlessly within the building systems. There are limited technical resources available for PBFD (even internationally), and a large amount of learning from past case studies must be done – particularly for new practitioners eager to pursue this area – as has been compiled in this study. Of the existing literature on PBFD, readers are encouraged to consult 'Performance-Based Fire Engineering of Structures' by Wang et al (2013) [3] and 'Structural Design for Fire Safety, Second Edition' by Buchanan and Abu (2017) [4] for education on performing more advanced design techniques which are beyond the scope of this report to detail in text.

Currently in Canada, with the introduction of the objective-based code in 2005 that contained the Alternative Solutions clause, designers had more freedom to deviate from the code prescribed solutions.

Part 1 in Division A in the National Building Code of Canada [5] outlines the use of Alternative Solutions for code compliance, and Part 2 in Division C gives the administrative framework for creating an Alternative Solution proposal. Clause 1.2.1.1, sentence 1(b) states that "Compliance with Division B shall be achieved by using Alternative Solutions that will achieve the level of performance required by the applicable acceptable solutions…". Each code clause that may be adopted to an equivalent solution is linked to Functional Statements and Objectives through the MMAH Supplementary Standard SA-1, that explain the underlying requirement of a code provision for which a PBFD solution may be developed.

The first design examples in Canada which utilised this procedure have referenced a PBFD guideline published by the Society of Fire Protection Engineers (SFPE, 2000) [6], such as the Nova Scotia Community College [7] in 2005 and the Citadel High School [8] in 2006. The general approach for these case studies was to use a Fire Dynamics Simulator (FDS) to calculate expected temperatures in the exposed structural steel and show they were below the limiting temperature prescribed by the standard fire test, CAN/ULC-S101[9]. That same trend has continued in Canadian case studies, which appear to be heavily reliant on FDS for temperature calculations but do not consider, or at least discuss in the literature, whole building structural response to fire. Other applications of the Alternative Solutions clause in Canada are related to human egress during fire events[10], as well as in certain high-rise applications to demonstrate overall equivalency to a high-rise comprised of acceptable solutions[11].

- ***Education***

In Europe, which is arguably a leader within FSE, there exists a colloquium of universities that provide engineers the necessary background to practice FSE within the context of their countries building codes. For example the joint International Master of Science in Fire Safety Engineering[12] involves a co-operation of universities from Belgium, Sweden, and the United Kingdom. The programme extends to offering trainees experiences in Switzerland, Australia, and the United States. The curriculum encompasses fundamentals in fire dynamics, risk analysis, human behaviour, structural design and a short research thesis (in that order to minimize course redundancy). The program, however, is rather inaccessible as it would demand Canadians training abroad which is not conventionally feasible to all interested practitioners.

To train fire safety engineers in Canada, there currently are two institutions offering undergraduate and graduate courses in fire safety engineering: Carleton University and the University of Waterloo. These courses include fire dynamics, and structural behaviour in fire. Human behaviour in fire was recently introduced to the curriculum in 2015 only at Carleton University; prior to this no formal education existed in Canada on this topic since 2008. Each university gives the option to the practitioner to undertake a research based thesis. These educational facilities offer practitioners the option to study via online lectures which is highly adaptable to the desires of practitioners. Structural behaviour is taught at both institutions yearly using the first edition text Structural Design for Fire Safety by Buchanan [13]. This text is beginning to lag relative to international practice, research, and computational abilities. Notably, the document is written before the tragic events of September 11th in 2001 which renewed North American research and education in fire safety. This text is due to have a second edition in early 2017 and it is anticipated that the structural fire course will require a curriculum update to match the scope and content of this updated text. As well, the structural fire course, and the fire safety engineering courses in general, have a lack of design problems that are provided in international educational experiences. Such problems would train students to apply the knowledge fundamentals to solve real-world problems, which is crucial in professional academic programs, to train future engineers to provide PBFD [14].

While the Canadian curriculum certainly has strengths that have provided historically acceptable fire safety solutions in the past, this critical review suggests there is room for improvement when comparing to our international peers.

These improvements may include removing redundancy from the courses offered, focusing more on first-principles of fire safety, introducing more design-type problems, and increasing the collaboration between industry and academia. These characteristics of a successful fire safety engineering program have been demonstrated in-part internationally [15] and have been shown to lead to a more developed fire safety profession with many consultancies actively participating and a plethora of innovative case studies to learn from, as will be discussed below.

- ***Practice***

The major companies discussed below (ARUP, Trenton Fire, BurroHappold) have been primarily focused on in this review as they have consistently published works on their projects (in the form of journal articles, magazine articles and conference proceedings), allowing the FSE profession to be developed and validated in the public realm. It is the authors' opinion that in order for FSE professionals to be recognized as competent engineers, it is essential that design solutions be published for the community to learn, and critique. Professionals must practice within the public view, under regulating body and peer review, if the field is to advance and provide opportunity for the safe adaptation of past FSE designs into new projects [16].

Combined, the companies discussed below have provided fire engineering services on hundreds of international projects and this has allowed them to be part of the design team for some of the most innovative and iconic buildings in the world. Several other firms providing PBFD in FSE exist and they cannot be discounted. Many are new, having responded to the recent demand in this niche area, and some larger firms have branched out to include an FSE group. The following sections give a brief background on how FSE was developed at selective major firms. These companies can be looked upon as exemplars for Canadian companies to model if not emulate.

<u>Arup fire</u>

Margaret Law started at Arup in the UK in the mid 1970's with the job description of starting fire engineering services at the firm [17]. Until the late 80's, Margaret struggled with arbitrarily increasing fire protection requirements and size limitations from the government which she saw as largely political but unsupported by research. She proposed the need for research to prove that

for more complex and innovative buildings, a unique fire engineering design can be developed for each case which will result in a more economical solution, directly proportionate to the people and property at risk[13].

The first dedicated fire engineering design team was established in 1988, and Arup began working on large projects such as covered shopping centres and terminals which extended beyond the limitations of the fire code provisions. The first building, to the knowledge of the authors that Arup Fire applied their contemporary PBFD strategies to was the London City Hall building, which opened in 2002. For this project they reduced structural fire protection using a time-equivalency method, and developed smoke management and evacuation procedures for the mixed occupancy of general public and office staff [18]. Later, in 2004, the fire group had developed and applied the first ever thermo-mechanical analysis and design solution in London. On the Plantation Place South development [19], they were able to eliminate structural fire protection on several beams, relying on tensile membrane action as validated by the (then) recent Cardington fire tests [20].

To date, there are over 200 projects listed on Arup's website that are related to fire. They have advanced PBFD concepts refined over the past quarter of a century, developed a highly sophisticated human behaviour modelling software MassMotion [21,22] as well as an environment for structural and fire analysis in LS-DYNA [23], and have over 180 fire engineers at 30 locations worldwide. They remain privy to the latest technology through a continuous commitment to supporting research.

<u>BuroHappold</u>

Michael Green, a partner at BuroHappold, founded the Fire Engineering, Design and Risk Assessment (FEDRA) group in the mid 90's, after beginning to work in fire safety in 1979. He and several others from the firm have since contributed to the writing of many standards and handbooks on FSE in the United Kingdom [24]. Over the years, the group has done work on numerous iconic buildings, specializing in sports, cultural, transportation and commercial projects[20]. In conjunction with the University of Sheffield beginning in 1985 [25], BuroHappold developed Vulcan [26], a finite element analysis software for steel frame and composite structures in fire that is now widely used in the fire engineering industry. Their first application of performance-based design methodologies and the Vulcan software on a project ensued at the end of the 20th century. Currently, the group works in over 20 offices worldwide. They provide fire safety analysis, structural protection design and fire safety management strategy services, and offer fire risk assessments which will be further discussed in the last section of this report.

<u>Trenton Fire</u>

Trenton Fire was originally established as a fire safety consultancy division of a larger company Butler & Young in 1992, and worked on several governmental contracts at the time. Four years later, the division became an independent limited company and Trenton Fire was officially incorporated. The firm offers services in FSE, fire simulation, fire risk assessment and several other specialized product testing and services. In their two decades of work, they have provided their services on hundreds of new buildings and historic rehabilitation projects. They also do research work and have developed a tool for assessing historic buildings which has been adopted by several heritage organizations [27].

<u>Branch-Out Fire Groups</u>

Large multidisciplinary firms (WSP, AECOM, SOM, Olsson etc.) have also branched out into providing fire engineering services. The field in Canada is still relatively new, but international ties have led to the establishment of fire teams within the Canadian environment at various engineering firms. These firms have yet to publicize data on any large scale PBFD projects, but offer clients services in the realm of FSE in Alternative Solutions as illustrated above.

3. Design

This section aims to provide the reader with a solid foundation of FSE concepts through outlining the fundamental theories and common effective strategies which have been developed and applied by leading international universities and firms. Furthermore, a list of best-suited projects and case studies are given to demonstrate where FSE may be employed in Canada.

o *Fundamentals*

In order to use FSE to produce PBFD solutions, there are several fundamental theories which are applied to each unique project. An understanding of these FSE concepts is the baseline requirement for the ability to provide fire engineering services to clients.

Design Fires

Under the current code provisions, all construction materials and components are tested with a Standard Fire Test [28]. This testing is typically done in a furnace with a standard time-temperature curve which was developed early in the 20th century by the ASTM, in which the temperature rises to almost 700°C in the first 10 minutes. By the 60 minute mark, the temperature in the fire compartment is over 925°C, and continues to rise almost linearly but indefinitely. The time until which a component fails in this test under an applied load is its "Fire Resistance Rating." The test was first used in June 1917 as part of a comparative index of ratings for concrete, steel and timber columns under a service load and standard heating. In the test the isolated materials were then ranked by time of failure. The appropriateness of these tests was further limited by the difficulties of manual temperature control, especially with timber specimens which may give off their own heat [29]. It is additionally important to note that components tested with this methodology are then incorporated into building systems in which they interact with other components. These interactions can then lead to unique failure mechanisms which can only be observed if an analysis of the entire structure, or a part of a structure with appropriate boundary conditions, is performed. The test would later evolve to include the concepts of restraint to represent these boundary conditions, but still cannot accurately resemble true structural supports which are highly variable.

Additionally, this Standard Fire as defined by temperature alone is not representative of a real fire and has no physical meaning in reality, other than providing a standardized temperature exposure[30]. Generally, building fires can reach much higher temperatures in varying amounts of time and will enter into a decay phase after either the available fuel or oxygen is consumed. In the standard test the fire is uniform along the specimen, whereas in reality the fire may not be. Recent testing in the UK has indicated that for some structures, non-uniform (or localized) fire exposure may be more critical and a better representation of real fire dynamics [31]. Several highly-variable factors contribute to the way a fire will burn in a structure. The major conditions that affect fire burning times, temperatures, and behavior include the type and amount of fuel present, building geometry and thermal insulation, and the ventilation rate based on openings.

In order to appreciate the significance of design fire definition, it is important to understand a brief history of how fire behaviour knowledge has developed. Due to the limited understanding of fire dynamics and structural mechanics at elevated temperatures in the 1920s, FSE at that time relied on equivalency based rules (now known to be inadequate to represent reality) to the standard fire. The methodology was developed by Simon Ingberg, and was meant to be representative of a fire from start to burn-out [32]. In 1967, the Fire Research Station in Boreham, UK held an important fire symposium. Here, it became evident that these prescriptive rules make it difficult to quantify just how much safety or little safety can be provided for real fires and complex assemblies. Prominent attendees including Margaret Law (then of the Fire Research Station) and Tibor Harmathy of the National Research Council Canada (NRCC) remarked on the lack of accurate tools and experiments available to quantify this concept [33]. Over the following decades, Harmathy would continue efforts towards understanding fire dynamics and developing

tools for fire modelling. He performed experiments in which he studied under- and over-ventilated fires at the NRCC, and created numerous empirical approximations [34]. Others would continue to study these aspects internationally, and the reader is encouraged to consult the work of Majdalani and Torero, 2014[35] for a more contemporary review of the subject.

By the early 1990's, research facilities and consultancy companies began to study structural fires. This was mobilized by the Broadgate Fire, a composite steel-frame structure which caught fire during construction, prior to the application of fire protection [36]. A series of large-scale, real fire experiments were conducted on three structures – a composite steel frame, a concrete frame and a timber frame structure. The tests provided abundant insight into real structural behaviour, but yielded limited data for real fire modelling and verification. A subsequent set of similar tests were then performed at Cardington which focused on fire dynamics in the late 1990's.

Following these preliminary experiments, several researchers began to study realistic fire behaviour. Tests by Kirby began to illustrate unusual fire behavior in exceedingly large compartments[37], and tests by Lennon in the early 2000s illustrated interesting dynamics associated with compartment boundaries and ventilation [38]. Dalmorack Tests, a compartmentalized fire in a reinforced concrete tower complex, would follow in the late 2000s led by the University of Edinburgh which provided additional scrutiny for our modern fire modelling tools [39], and today a series of large scale compartment tests are being performed by an international consortium.

Today, there are two accepted categories of realistic fires that may occur inside a building. The first is in a compartment such as enclosed offices or retail spaces, which burn very hot and fast in a high-ventilation situation and burn slow and long in a low-ventilation situation [40]. The second is in an open space such as an atrium or concourse, also now contemporary hypothesized as a Travelling Fire (discussed in proceeding sections). Evidently, there are a large number of different fire scenarios that may occur within a building. A deterministic approach to fire safety design considers several key "Design Fires" which are determined to be critical cases, and are then used to develop an appropriate fire strategy and protection scheme.

Hand Calculations & *Modelling*

After the Design Fires have been determined, hand calculations and modelling follow. Calculations are either done through spreadsheet and/ or validated computer software. The two major underlying numerical methodologies to be understood are finite element analysis (FEA) and computational fluid dynamics (CFD).

FEA is used in a variety of ways in a fire engineering analysis. Using software such as Vulcan [22]or SAFIR[41], the design fire parameters are input and the temperature histories of structural members in those fires may be determined. This is typically done by applying environmental time-temperature data as boundary conditions on an element cross-section. The temperature differential that transfers through the cross-section of each element is then calculated at each time step. FEA is also used subsequently to analyse the structural performance of the elements at these elevated temperatures with applied loading. In updated software such as an LS-DYNA interface[19], the thermal and subsequent structural analyses can be coupled into a single simulation encompassing all of the fire effects. Models created for these simulations should reflect non-linear and local plastic deformations and failures, and consider changing material properties at elevated temperatures.

CFD is typically used to model smoke generation and migration through a space or building, and can be used to design and justify smoke control system requirements or lack thereof. Based on the fire, materials and building geometry, the CFD analysis predicts how the hot and toxic smoke layer will develop which is a limiting factor in occupant egress safety. It can also determine the

effects of smoke buoyancy and exhaust – critical parameters in the design of atria, stairwells and interconnected spaces.

Another modelling tool which is currently under rapid research and development is software which models human behaviour and movement within a building. The analysis uses a combination of pedestrian flow information and contributions from randomized personalities in each individual "agent" (occupant in the simulation). The tools are being developed (such as MassMotion[17,18], Pathfinder, etc.) using extensive data observed from fire drills and from normal pedestrian movement, and each agent has their own agenda or priorities affecting their behaviour. This type of simulation can be used to determine problems in evacuation routes or justify code deviations in travel distances or exit widths. This has yet to be applied to FSE in Canada on a published fire design though its principles and benefits have been realised in other civil engineering fields such as transportation infrastructure [42].

Design Strategy Implementation

A vital component to a successful PBFD is working with key stakeholders and governing bodies from the very beginning. At the preliminary design phase of the project, fire engineering strategies should be planned and discussed with the client and their insurance provider to determine their business or property priorities, and with the authority having jurisdiction to ensure a smooth and timely approval process.

- ***Strategies***

There are three types of PBFD design measures which can be employed in a combination of strategies. These categories include Occupant Egress, Smoke and Flame Control and Structural Fire Protection. Various strategies have been assembled after the review of numerous case studies from successful fire engineering projects.

Occupant Egress

4. A direct way to demonstrate the safety of an occupant egress strategy is to compare the Required Safe Evacuation Time (RSET) for the occupants to exit the fire zone, versus the Available Safe Evacuation Time (ASET) before fire conditions are no longer tenable for human safety [43]. This is typically done through occupant movement simulations or calculations, and smoke and heat development analyses. This can be used to justify longer travel distances or reduced exit widths for egress routes. Note that it has been observed that the width of the exit (and therefore the queueing time at the exits) usually governs the RSET rather than increased travel distances, where spaces are open and clear [13,33].
5. There are several different evacuation strategies that can be developed for the building management. Phased evacuation is when only the zone in the building with immediate hazard is evacuated first, allowing for reduced demand on the egress routes and increased business continuity in the event of a false alarm. Horizontal evacuation can be used with fire separated zones, where the egress strategy is to move occupants into another safe compartment within the building. This reduces total evacuation times and again allows for business continuity. Egress elevators are another emerging strategy in which working elevators can be considered means of evacuation, especially for accessibility or tall building applications, but are still under much scrutiny in practical situations as generations of individuals have been trained not to utilise elevators in an emergency.
6. It is often justifiable and beneficial to design normal circulation routes as primary means of egress as it has been proven that occupants typically exit the building the way they entered in emergency scenarios, and alternate egress routes or stairwells are often overlooked [44].

Smoke & Flame Control

1. Smoke control is an effective way of increasing ASET through exhaust or pressurization systems. Exhaust systems can be implemented through natural or power ventilation, and

should be validated using CFD analysis. In the majority of cases, a highly ventilated fire is desired because hot, toxic smoke is removed and the temperature of structural elements lags behind leading to a shorter duration of heating in the members. If designed properly, pressurization systems utilize high pressure zones within egress routes or fire-fighting shafts to induce air flow out of the zones and therefore inhibit infiltration of smoke into said zones. However, there is concern that the systems do not always perform as intended during design[45] which demonstrates the importance of a holistic approach to FSE, and for designers to understand the limitations of the systems they implement.

2. A "cabin" methodology can be used to contain flame and smoke within one area, through the use of suppression systems and smoke control. These areas are typically high-risk fire zones with large fuel loads, and can be compartmentalized through permanent walls or fire shutters, or open concept. In any situation, the cabin should be designed so that smoke and flames can be contained within that zone with a combination of automatic sprinklers, strategically placed barriers, and smoke exhaust vents or reservoirs [33].
3. The "island" methodology is another concept developed in which fire spread is contained simply through distance between combustible materials. This can be employed in large, open-concept areas such as concourses or waiting areas in transportation buildings, where there are very small amounts of combustibles. The heat release from a fire in one island of combustibles, or fuel, is determined considering the type of materials and ventilation. Next, an analysis is done to ensure the heat release is not sufficient to ignite another adjacent combustible island, therefore preventing the spread of the fire from its origin [33].
4. Various detection systems can be designed or specified to ensure a fire engineering strategy is well executed, and to aid in reducing RSET and increasing ASET. The choice of detection system is highly important, and the appropriate type of system varies in every building application. The varieties include smoke detectors, heat detectors (either sensing a rate of temperature rise or a given temperature threshold), and infrared flame detectors. "Double-knock" detection systems can also be employed which only activate minor alarms (rather than throughout the entire building) until a second detector or manual pull is triggered, reducing the likelihood of a false alarm. Avoiding false alarms is not only important for business continuity, but also to reduce the desensitization of building occupants to emergency alarms and evacuations.
5. Automatic sprinkler systems can often aid in the containment of fire within high fuel load compartments, but can be ineffective in multi-story atria. Simulations should be run to determine whether or not air and smoke reaching the sprinklers will be hot enough to activate the system. Typical ceiling sprinklers can be used, along with other options including long-distance throw or side wall sprinklers if required.

<u>Structural Fire Protection</u>

1. A structural failure in a fire can be judged by three criteria: loss of load bearing capacity, infiltration of smoke through cracks or openings, and transfer of sufficient heat to ignite materials on the opposite side of the structural element. The latter two criteria are applicable for load-bearing fire separation wall and floor assemblies, or non-load-bearing fire separation partitions.
2. A strategy that has been used to optimize the amount of fire protection is partial fire protection, which should be validated through FEA of critical members in several design fires[46]. When proposing to leave an element unprotected, redundancy is critical. At least two separate analyses could be done to prove that (a) the element can be removed from the structure without inducing a progressive collapse of the building, or (b) the element can be allowed to thermally expand and the induced loading on the surrounding structure will not cause a progressive collapse of the building. The extent of local failure allowed is an important acceptance criteria to be developed which may have different implications in each unique building system. Typically elements that can be proven to stay unprotected are

secondary beams and structural elements over several metres high from the floor. Generally, structures only supporting a roof assembly (that is not a publicly accessible roof) do not require protection, but in complex cases a design fire analysis should be done to determine whether or not fire protection for the roof structure is critical to the whole building fire safety. Additionally, frame connections are critical elements in the stability of a building and should be given specific attention in the thermal and structural analyses.

3. Another common strategy of reducing fire protection requirements is on exterior steel structural members. According to the building code, members at least one metre away from the façade in certain building types do not require protection. However, it has been justified to leave unprotected elements that are closer, through the two analysis types discussed previously.

- ***Examples***

There are several types of buildings that lend themselves to PBFD strategies. Large and complex buildings often have spaces that require extra care in their design to ensure occupant safety, or have been envisioned with architectural or engineering feats which would not be possible with a prescriptive code-compliant solution. There are also many buildings in which the code prescribed fire protection measures and evacuation limitations may prove overly conservative, and there are opportunities to employ fire engineering expertise to develop a more economically efficient solution for the client. The following are examples of the types of projects which are considered best suited for PBFD in the opinion of the authors as precedent case studies can be used as reference. Each is presented along with listed case studies which exemplify the methods used.

Transit Stations

Often in transit stations with terminals, concourses and waiting areas, it is desirable to have large, expansive open spaces. This desire is typically constrained by required occupant egress travel distances, so a fire engineering design and supporting analysis justifying increased distances can be beneficial. Additionally, smoke control strategies and CFD analyses can become very important and valuable in these types of spaces. Transit stations can often be divided into low-risk fire areas including the aforementioned open spaces, and high-risk fire areas including retail shops, restaurants, luggage holding and ticketing areas. The low-risk open areas often have high ceilings and may be a good opportunity to reduce the structural fire protection required. The high-risk areas can be designed for using the “cabin” or “island” strategies. For exemplary projects which have made use of these strategies, the reader is encouraged to consult case studies on the Beijing South Railway Station [33] and the Transbay Transit Center in San Francisco [47].

Atria

Atria present unique fire engineering challenges within buildings due to the openness and connection of multiple stories. Smoke control can either benefit or suffer in atria depending on the geometry and height, so in each unique case comprehensive CFD should be done. Many atria are designed for their natural ventilation advantages, which can often be utilized for passive smoke exhaust systems. CFD can support the elimination of mechanical smoke exhaust systems based on larger air speeds and the size of openings at the base and top of the atrium, utilizing the natural buoyancy of hot smoke. However, if the height of an atrium becomes too high, smoke can be cooled and lose its buoyancy before it is exhausted out of the top vents, which could be detrimental to the smoke control plan. To keep the openness of atria but further control smoke flow, horizontal or vertical fire shutters may be employed to separate floors or adjacent areas in the event of a fire. Additionally, the large size of atriums usually lends itself to a partial structural fire protection strategy which could save time and money. Projects which utilized such strategies in their approval are discussed in the case studies of The Greater London Authority Building (shown in Figure 2) [14] and the Y2E2 Stanford University Building [48].

Figure 2. The Greater London Authority Building was one of the first projects approved in London using modern FSE concepts and strategies which allowed the unique building to be executed[7] (Authors photo).

Stadiums and Arenas

Sports venues are yet another type of building where extremely large open spaces are the norm, with the added challenge of thousands of spectators compressed into tightly spaced, elevated, fixed seats. Occupant egress modelling can prove very important in the design of these buildings. An analysis of pedestrian flow can be used to optimize the building layout and surrounding landscape so that movement is easy and queues are avoided. These buildings are good candidates to utilize the normal circulation areas as egress routes, as mentioned above as the third strategy within the Occupant Egress category, or horizontal evacuation plans. High-risk fire zones such as restaurants and retail shops can be treated as cabin zones to mitigate fire spread, and adequate smoke ventilation should be provided in case of a fire on the lower service levels. Additionally, the large open spaces and high roof create the opportunity to eliminate most of the structural fire protection required, if justified through design fire simulation. The reader is directed to case studies on the First Direct Arena [49] and the Emirates Stadium [50] in the UK for example applications of these strategies.

Densely Furnished or Occupied Open Spaces

In projects where expansive, highly-utilized spaces are desired, PBFD strategies may be required to justify less compartmentalization and longer travel distances. These spaces could include open-plan offices, conference centres, casinos and department stores, for example. In one of these building types constructed of regular grid steel-composite structures, often the fire protection on secondary beams can be reduced or eliminated via the methodology in the third case study below.

Unique occupant egress strategies (such as horizontal or even upwards evacuation – see case studies one and two below) can be developed in these situations along with a detailed design of signage and management plans, and of building geometry to promote clear, logical evacuation paths. The design of the building geometry can also be used to strategically control smoke migration throughout the space, using complex bent or curved spaces as an advantage. Often CFD and occupant modelling will be required to demonstrate that the predicted smoke behaviour and occupant RSET are justified. Additionally, in these large, open, highly-utilized spaces (unlike transit concourses with little combustible materials), large fire shutters or doors can be employed which close in the event of a fire. These systems again require a detailed evacuation strategy and method of communication to occupants as to their required egress destination. Several case studies demonstrate the use of some of the listed strategies, including the Marina Bay Sands in Singapore [51], the Shenzhen Stock Exchange in China [52] and a UK Retail and Leisure Complex [53].

Mixed-Use High Rise

Code limitations are often very inhibiting in very tall buildings, especially when multiple functionalities are located throughout the height of the high rise. Typically code mandates at least two discrete egress stairs for each separate occupancy in the building, which could consume a

large amount of profitable area in sky scrapers with 4 or 5 different functionalities (i.e. retail, office, hotel, residential, etc.). Occupant modelling, CFD to determine ASET and phased evacuation strategies can be used to justify a significant reduction in the required stairwells and therefore increase in lettable floor area.

On the other end of the spectrum, at times code provisions can prove insufficient for complex high-rise projects. Design fire modelling should be done to develop with confidence a fire protection plan which covers critical structural elements to ensure the RSET is met. Another emerging trend in tall buildings is placing the steel superstructure outside of the building façade. This is an excellent opportunity to do a design fire analysis and justify the elimination of fire protection on exterior structural steel members. Examples of projects which utilized FSE strategies for the realization of iconic high-rises are found in The Shard in the UK (seen in Figure 3) [54] and in the EW11 tower in the United Arab Emirates [55].

Figure 3. The Shard high-rise building in London, England used innovative fire strategies to incorporate the functionality and spatial needs of the building [42] (Authors photo).

Specialized or Highly-Constrained Projects

Highly specialized buildings or constrained projects such as historic restorations provide an opportunity for unique fire engineering strategies to be developed. In these restricted projects, often floor space is a valuable commodity due to space limitations. On a larger scale, specialized projects can involve several buildings on a site for which a total fire safety design must be completed. In this case, it is beneficial to the project to provide a fire engineering master plan to set standards for the entire campus, and to work with local authorities and fire brigades from conception to ensure a safe and approvable design.

Additionally in sensitive historic projects, new fire safety or protection measures must not be invasive, and must be respectful of the heritage values within the building. Often modelling can be done to either demonstrate a reduction in the required egress width (resulting in more lettable floor space) or that the inherent fire protection in the building is sufficient for the required structural integrity or ASET. Integrating added egress routes into new additions in heritage buildings can also minimize the intervention required in historic staircases or corridors. The reader is directed to several case studies which exemplify these strategies for more information on the Singapore Flyer Terminal [56], the London Olympic Park [57] and The Corbin Building at the Fulton Center in New York [58].

7. The Future of Fire Safety Engineering in Canada

There are potential benefits of using performance-based design in FSE in Canada which could be realized. In complex buildings with uncommon uses, layouts or geometries, FSE can improve upon a prescriptive solution by either increasing the fire protection measures to ensure that an acceptable level of safety for people and property is provided, or optimize the fire protection measures to create a more economical and efficient design. Furthermore, many new iconic building designs would simply be impossible without the use of FSE. Once these advantages of PBFD become clear to practitioners in Canada, the demand for fire safety engineers will increase and the subject being viewed as a profession will gain traction within the building engineering community. The following sections give a brief glimpse into some future and potential strategies for PBFD for use in Canada.

- ***Travelling Fires***

As aforementioned, the Travelling Fire methodology is a concept being developed which does not use the typical assumption that a building fire provides a uniform temperature over the whole compartment it occupies. Rather, the fire migrates throughout the space where fuel is available (see Stern-Gottfried & Rein, 2012) [59]. It is still in the early stages of being accepted globally, but several of the firms mentioned in this document have utilized the Travelling Fire as a design basis for PBFD efforts [60]. This type of fire simulation is thought to be exceedingly more realistic for large open spaces such as atria, transit station concourses and stadia and is currently being studied in large compartment tests by various researchers to improve user friendliness and accuracy. It currently considers more uncertainties that exist in a real fire such as non-uniform heating of the structure, and could reduce the fire load on the structure since any single element is not typically exposed to the extreme heat for the entire fire duration. Conversely, a localized heating effect could prove to be more severe for some types of building configurations.

- ***Probabilistic versus Deterministic***

Once the use of design fires in Canada is well established, fire engineering could progress to a probabilistic rather than deterministic approach. Probabilistic analysis uses a risk-based approach, rather than a worst-case scenario design. This analysis is based on risk factors affecting the probable frequency of a fire in a certain time period, the probability of failure due to that fire, and the consequences of that fire. The assessment is done over a large range of virtually all possible fires in all possible compartments, and a solution is reached once a predetermined degree of risk is achieved. A probabilistic approach is desirable as it considers uncertainties that invariably exist in the real world. Additionally, it is a more systematic approach and aligns with the NBCC "acceptable level of risk" methodology, which will be more comprehensible to authorities having jurisdiction and easily calibrated or compared to local building codes.

- ***Risk and Resilience***

Increasingly, asset owners are beginning to look at the Resilience of their properties. When extreme events occur, either malicious events or natural events brought on by extreme weather and climate change, buildings must be able to sustain the events to a level that is acceptable by the users. A brief consideration of the aforementioned Canadian case studies in the introduction are evident of these issues. A building that can remain operational during an event or shortly thereafter will enable the occupants of the building to be more resilient. The business continuity plans of the occupants may rely on the performance of the building to be effective. PBFD offers an opportunity to quantify the risk exposure from a range of fire events and determine the expected damage levels ahead of time so that business continuity plans and repair strategies can be developed accordingly. This is a fundamental shift from the current prescriptive approach, where risk due to fire is not explicitly calculated except for extrapolating historical losses from similar building types. Further research is required internationally to better define the damage

levels and required repair for various design fires, but PBFD can lead to more resilient and robust structures.

8. Conclusions

It is the authors opinion, that just as structural engineers are not often celebrated in building design for making beautiful, famous buildings stand (as they are simply expected to do by the general public), fire professionals in Canada are often not acknowledged until a devastating event leads to the need for an expert witness. Likewise, in the Canadian building construction industry, fire safety engineers are often an afterthought only sought if the architect wishes to deviate from the prescribed building code. Once architects and developers in Canada are made aware of the flexibility that PBFD can provide in innovation as seen abroad, it could be difficult to imagine a future built environment without FSE. Following the international example, FSE in Canada could become an obvious component of the integrated design process from the outset in the future. This can potentially lead to truly optimized buildings and fire protection strategies.

Should we proceed in this route, the demand for competent fire safety engineers could grow, thus increasing the number and quality of FSE education programs in Canada. This, in turn, will facilitate the advancement of design firms providing PBFD services and of authorities having jurisdiction approving these projects. This cycle, along with the active publication of FSE projects and solutions, will serve to stabilize the FSE as a profession on its own like civil or mechanical engineering - creating a solid foundation upon which it will stand unique. This will lead to a future built environment in Canada where fire safety is demonstrated, rather than just assumed, and the public will benefit from better performing buildings that enable resilient operations.

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