

# Travelling Fires for Structural Design

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*To my wife, Ina*



# Abstract

Traditional methods for specifying thermal inputs for the structural fire analysis of buildings assume uniform burning and homogeneous temperature conditions throughout a compartment, regardless of its size. This is in contrast to the observation that accidental fires in large, open-plan compartments tend to travel across floor plates, burning over a limited area at any one time.

This thesis reviews the assumptions inherent in the traditional methods and addresses their limitations by proposing a methodology that considers travelling fires for structural design. Central to this work is the need for strong collaboration between fire safety engineers to define the fire environment and structural fire engineers to assess the subsequent structural behaviour.

The traditional hypothesis of homogeneous temperature conditions in post-flashover fires is reviewed by analysis of existing experimental data from well-instrumented fire tests. It is found that this assumption does not hold well and that a rational statistical approach to fire behaviour could be used instead.

The methodology developed in this thesis utilises travelling fires to produce more realistic fire scenarios in large, open-plan compartments than the conventional methods that assume uniform burning and homogeneous gas phase temperatures which are only applicable to small compartments. The methodology considers a family of travelling fires that includes the full range of physically possible fire sizes

within a given compartment. The thermal environment is split into two regions: the near field (flames) and the far field (smoke away from the flames). Smaller fires travel across a floor plate for long periods of time with relatively cool far field temperatures, while larger fires have hotter far field temperatures but burn for shorter durations.

The methodology is applied to case studies showing the impact of travelling fires on generic concrete and steel structures. It is found that travelling fires have a considerable impact on the performance of these structures and that conventional design approaches cannot automatically be assumed to be conservative. The results indicate that medium sized fires between 10% and 25% of the floor area are the most onerous for a structure. Detailed sensitivity analyses are presented, showing that the structural design and fuel load have a larger impact on structural behaviour than any numerical or physical parameter required for the methodology.

This thesis represents a foundation for using travelling fires for structural analysis and design. The impact of travelling fires is critical for understanding true structural response to fire in modern, open-plan buildings. It is recommended that travelling fires be considered more widely for structural design and the structural mechanics associated with them be studied in more detail. The methodology presented in this thesis provides a key framework for collaboration between fire safety engineers and structural fire engineers to achieve these aims.

# Declaration

This thesis and the work described within have been completed solely by Jamie Stern-Gottfried under the supervision of Dr Guillermo Rein and Prof José Luis Torero. Where others have contributed or other sources are quoted, full references are given. This work has not been submitted for any other degree or professional qualification.

Jamie Stern-Gottfried

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# Scholarly Output

## Journal Papers

Stern-Gottfried, J., Rein, G., Bisby, L.A., and Torero, J.L., "Experimental review of the homogeneous temperature assumption in post-flashover compartment fires". *Fire Safety Journal*, Vol. 45, 2010, pp. 249-261.

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## Invited Talks

"Fires in Large Compartments", Invited Talk at The Rasbash Lecture, hosted by The Institution of Fire Engineers, Andover, UK, 2008.

"Design Fires for Structural Analysis", as part of the workshop on Structural Fire Engineering prior to *The 9<sup>th</sup> International Symposium on Fire Safety Science* in Karlsruhe, Germany, 2008.

## Conference and Magazine Papers

Stern-Gottfried, J., Rein, G., Lane B., and Torero, J.L., "An Innovative Approach to Design Fires for Structural Analysis of Non-Conventional Buildings: A Case Study". *International Workshop in Applications of Structural Fire Engineering*, Prague, Czech Republic, 2009.

Stern-Gottfried, J., Rein, G., Lane B., and Torero, J.L., "A Novel Methodology for Determining Design Fires for Structural Fire Analysis". *6<sup>th</sup> Mediterranean Combustion Symposium*, Corsica, France, 2009.

Stern-Gottfried, J., Rein, G., and Torero, J.L., "An Experimental Review of the Homogeneous Temperature Assumption in Post-Flashover Compartment Fires". *International Congress on Fire Protection and Life Safety in Buildings and Transportation Systems*, Santander, Spain, 2009.

Stern-Gottfried, J., Rein, G., and Torero, J.L., "Travel guide", *Fire Risk Management*, November 2009, pp. 12-16.

Stern-Gottfried, J., Rein, G., and Torero, J.L., "A Performance Based Methodology Using Travelling Fires for Structural Analysis". *8<sup>th</sup> International Conference on Performance-Based Codes and Fire Safety Design Methods*, Lund University, Sweden, June 2010.

Stern-Gottfried, J., Rein, G., and Torero, J.L., "Experimental Review of the Homogeneous Temperature Assumption in Post-Flashover Compartment Fires". *The 12<sup>th</sup> International Interflam Conference*, University of Nottingham, UK, July 2010.

Jonsdottir, A., Rein, G., and Stern-Gottfried, J., "Comparison of Resultant Steel Temperatures Using Travelling Fires and Traditional Methods: A Case Study of the Informatics Forum Building". *The 12<sup>th</sup> International Interflam Conference*, University of Nottingham, UK, July 2010.

Rein, G. and Stern-Gottfried, J., "Travelling Fires in Large Compartments: Realistic fire dynamics for structural design", *International Conference on Applications of Structural Fire Engineering*, Prague, Czech Republic, Prague, Czech Republic, 2011.

## Posters

Stern-Gottfried, J., Rein, G., Lane B., and Torero, J.L., "Design Fires for Structural Analysis of Complex Buildings". *The 9<sup>th</sup> International Symposium on Fire Safety Science*, Karlsruhe, Germany, 2008.

Stern-Gottfried, J., Rein, G., and Torero, J.L., "Experimental Review of the Homogeneous Temperature Assumption in Post-Flashover Compartment Fires". *Spring Meeting of the British Section of the Combustion Institute*, Edinburgh, UK, 2010. (**Awarded Best Poster**).

## Awards

David B. Gratz Scholarship, 2010. Awarded by the Fire Safety Educational Memorial Fund of the National Fire Protection Association (NFPA) in the USA to a graduate student in Fire Science located outside the USA and Canada who demonstrates scholarship achievement, leadership qualities, concern for others/volunteerism, and contributions to international/national fire safety activities.



# Preface

This thesis is written in manuscript format. As such each chapter is a standalone document suitable for journal publication. The material is presented as follows:

**Chapter 1** is a brief introduction to the concept of travelling fires and the research presented in this thesis. While not a full manuscript, it is loosely based on:

Stern-Gottfried, J., Rein, G., and Torero, J.L., "Travel Guide", *Fire Risk Management*, November 2009. pp. 12-16.

**Chapter 2** presents a review of the homogeneous temperature assumption in post-flashover fires that is invoked in many compartment fire models. This manuscript has been published as:

Stern-Gottfried, J., Rein, G., Bisby, L.A., and Torero, J.L., "Experimental review of the homogeneous temperature assumption in post-flashover compartment fires". *Fire Safety Journal*, Vol. 45, 2010, pp. 249-261.

**Chapter 3** is a literature review of research in travelling fires for structural analysis. This manuscript has been submitted for journal publication.

**Chapter 4** presents a collaborative research effort with structural fire engineers. The chapter investigates the impact of the travelling fires methodology

developed in this thesis on a generic concrete frame. In this work, I quantified and reported on the thermal environment. The lead author performed and reported on the structural analysis as part of his PhD thesis. This manuscript has been published as:

Law, A., Stern-Gottfried, J., Gillie, M., and Rein, G., "The influence of travelling fires on a concrete frame", *Engineering Structures*, Vol. 33, 2011, pp. 1635-1642.

**Chapter 5** presents the detailed development of the travelling fires methodology with application to heating of a generic concrete structure. This manuscript has been submitted for journal publication.

**Chapter 6** is a conclusion for this thesis and presents recommended future work. This chapter is not intended to be a published manuscript.

**Appendix A** provides details of the heat transfer calculations utilised in Chapters 2 and 5.

# 1

## Introduction

Close inspection of accidental fires in large, open-plan compartments reveals that they do not burn simultaneously throughout the entire enclosure. Instead, these fires tend to move across floor plates as flames spread, burning over a limited area at any one time. These fires have been labelled “travelling fires”.

Despite these observations, fire scenarios currently used for the structural fire design of modern buildings are based on traditional methods that come from the extrapolation of existing fire test data. Most of these data stem from tests performed in small compartments that are almost cubic in nature. This test geometry allows for good mixing of the fire gases and thus for a relatively uniform temperature distribution throughout the compartment. While this behaviour is different from that observed in real fires, it has generally been deemed a conservative, and therefore appropriate, approach for structural fire design in the absence of better and more relevant data. This approach might be considered acceptable for most

design cases, but the need for better optimisation of structural behaviour in fire will eventually require a more realistic definition of the fire.

Computational methods for determining structural behaviour have matured over the last decade and have enabled analysis of more complex structural systems. This has led to an understanding that many modern structures do not behave in the same manner as simpler, more traditional frame based systems. In order to address these differences, and continue to enable innovation in structural design, a more sophisticated characterisation of fire scenarios is required.

This thesis reviews the assumptions inherent in the traditional methods and addresses their limitations by proposing a methodology to consider travelling fires for structural design. Central to this work is the need for strong collaboration between fire safety engineers to define the fire environment and structural fire engineers to assess the subsequent structural behaviour.

## **1.1 Traditional Methods**

It is important to understand the context of the current design methods to establish a new methodology for design fires for structural analysis. Traditionally, structural fire analysis has been based on one of two methods for characterising the fire environment:

- The standard temperature-time curve (as specified by various standards, such as BS 476 [1], ISO 834 [2], and ASTM E119 [3])
- Parametric temperature-time curves (such as that specified in Eurocode 1 [4]).

While both of these methods have great merits and represented breakthroughs in the discipline at their times of adoption, it is recognised that they have limitations.

The standard temperature-time curve, which is used as the basis for the fire rating system in most building codes and standards worldwide, was first published in 1917 [5]. The curve came from collating various fire tests into one idealised curve. The tests that fed into the development of the standard fire were intended to represent worst case fires in enclosures so that the structure could withstand burnout. However, these tests were conducted and the standard fire created prior to much scientific understanding of fire dynamics. Thus the standard fire, unlike a real fire, has a relatively slow growth period, never reduces in temperature due to fire decay, and is independent of building characteristics such as geometry, ventilation and fuel load.

The next major landmark for structural fire analysis, in terms of design, was a guidance document produced in Sweden in 1976 [6]. This work incorporated the current understanding of compartment fire dynamics based on tests conducted in small scale enclosures. The document presented the key factors of compartment fire temperatures as the fuel load, ventilation, and the thermal properties of the wall linings. The guide gave design recommendations and a series of temperature-time curves for a wide range of the critical parameters, accounting for the cooling period of the fire.

The Eurocode parametric temperature-time curve is based on the same fire science as the Swedish design guide. The Eurocode parametric temperature-time curve was developed to collapse all of the curves given in the Swedish guidance document into a simplified mathematical form.

Eurocode 1 [4] states that the design equations for the parametric temperature-time curve specified are only valid for compartments with floor areas up to 500m<sup>2</sup> and heights up to 4m. In addition the enclosure must have no openings through the ceiling and the thermal properties of the compartment linings must be within a limited range. As a result, common features in modern construction like large

enclosures, high ceilings, atria, large open spaces, multiple floors connected by voids, and glass façades are excluded from its range of applicability. These limitations, which are largely associated with the physical size and geometric features of the experimental compartments on which the methods are based, ought to be carefully considered when the method is applied to an engineering design beyond the recommended ranges of applicability. This is particularly relevant given the large floor plates and complicated architecture of modern buildings.

It is noted that the background document to the UK National Annex of Eurocode 1 [7] suggests that designers can ignore the given limitations on floor area and compartment height and can expand the range of the compartment lining values. While this allows engineers to use the equations on more practical applications, it does not appear to address the observed travelling nature of real fires in large compartments.

## **1.2 Non-Uniform Burning**

The traditional methods mentioned above for specifying design fires for structural engineering analysis assume spatially homogeneous temperature conditions. The accuracy and range of validity of this assumption is examined in Chapter 2, using the previously conducted fire tests of Cardington (1999) and Dalmarnock (2006). Statistical analyses of the test measurements provide insights into the temperature fields in the compartments. The temperature distributions are statistically examined in terms of dispersion from the spatial compartment average. The results clearly show that uniform temperature conditions are not present and variations from the compartment averages exist. Peak local temperatures range from 23% to 75% higher than the compartment average, with a mean peak increase of 38%. Local minimum temperatures range from 29% to 88% below the spatial average, with a mean local minimum temperature of 49%. The experimental data are then applied to typical structural elements as a case study to examine the potential impact of the gas

temperature dispersion above the compartment average on element heating. Compared to calculations using the compartment average, this analysis results in increased element temperature rises of up to 25% and reductions of the time to attain a pre-defined critical temperature of up to 31% for the 80<sup>th</sup> percentile temperature increase. The results show that the homogeneous temperature assumption does not hold well in post-flashover compartment fires. Instead, a rational statistical approach to fire behaviour could be used in fire safety and structural engineering applications.

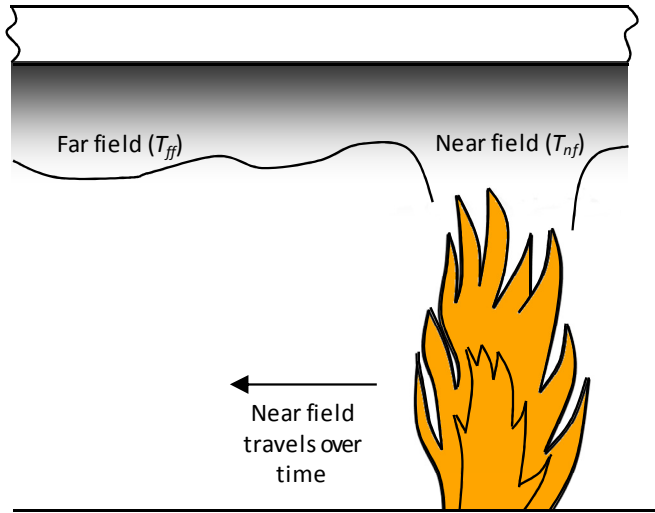
This heterogeneity of the temperature field will be more pronounced when the burning itself is not uniform, as is the case in travelling fires. A travelling fire is when only a portion of a floor plate is fully involved in flames that then move to other areas of the floor as burnout occurs in locations of earlier burning. The fire travels as flames spread to unburnt fuel, partitions or false ceilings break and ventilation changes through glazing failure.

Many large, accidental fires, such as those in the World Trade Center Towers 1, 2 [8] and 7 [9] in New York in 2001, the Windsor Tower in Madrid, Spain in 2005 [10] and the Faculty of Architecture building at TU Delft in the Netherlands in 2008 [11] were all observed to travel across floor plates, and vertically between floors, rather than burn uniformly for their duration. Similar observations were made of the Interstate Bank fire in Los Angeles in 1988 [12] and the One Meridian Plaza fire in Philadelphia in 1991 [13]. Travelling fires have also been observed experimentally in compartments with non-uniform ventilation [14, 15, 16].

### **1.3 Travelling Fires**

Based on the above, it can be seen that the concept of travelling fires is in direct contrast to the fundamental nature of current design methods that assume uniform conditions throughout a compartment for the entire duration of burning of a fire.

A fire that burns uniformly within a large enclosure would generate high temperatures, but only for a relatively short duration. However, a fire that travels will still create elevated temperatures away from the fire (the far field) as well as flame temperatures in the near field. A travelling fire can therefore inflict the structure with elevated temperatures for longer durations. A travelling fire is illustrated in Figure 1.1, showing the difference between the near field and far field.



**Figure 1.1:** Illustration of a travelling fire.

Due to the discrepancy between fire behaviour in actual incidents and that assumed in traditional design methods, it is possible that current practices for structural design do not consider a potentially worst case fire scenario. Non-uniform heating across a compartment floor could cause a failure mechanism in the structure which may not occur if uniform temperatures were applied to the structure. For example, a cool, unheated bay in a multi-bay structure could produce high axial restraint forces and that could result in failure of a heated element. In other situations, however, traditional design methods may be overly conservative compared to the impact of a real fire.

## 1.4 A New Methodology

To address the limitations of the traditional methods, and provide the necessary tools to enable a more realistic determination of a building's response to fire, a methodology has been developed that can incorporate the actual dynamics of a travelling fire into structural analysis. This methodology will better enable structural and architectural design innovation.

Chapter 3 is a literature review of travelling fire research. A brief background to the traditional methods that assume uniform fires is given along with critiques of that assumption, such as the heterogeneity of compartment temperatures and the observation of travelling fires in both accidental events and controlled tests. The research in travelling fires is reviewed, highlighting the pioneering work in the field to date. The main challenge in developing tools for incorporating travelling fires into design is the lack of large scale test data. Nonetheless, significant progress in the field has been made and a robust methodology using travelling fires to characterise the thermal environment for structural analysis has been developed. The research in quantifying the structural response to travelling fires is also reviewed.

Chapter 4 presents a collaborative analysis between fire engineers and structural fire engineers. A basic version of the travelling fires methodology, using a family of fires, is applied to a framed concrete structure. A Finite Element Model of the generic concrete structure is used to study the impact of the family of fires; both relative to one another and in comparison to the conventional methods. It is found that travelling fires have a significant impact on the performance of the structure and that the current design approaches cannot be assumed to be conservative. Further, it is found that a travelling fire of approximately 25% of the floor plate in size is the most severe in terms of structural response. It is concluded that the

travelling fires methodology is simple to implement, provides more realistic fire scenarios, and is more conservative than current design methods.

Chapter 5 gives a more developed version of the methodology. Many of the assumptions of the method are explored, and a robust spatial discretisation scheme is adopted to characterise the far field variation of a linearly travelling fire. Heating of a similar concrete structure to that used in Chapter 4 is examined. A detailed sensitivity study is also conducted, highlighting the critical parameters for design. It is found that the most sensitive parameters are related to the building design and its use and not the physical assumptions or numerical implementation of the model.

## 1.5 Collaboration

As the disciplines of fire science and structural engineering are very disparate in their knowledge base, but have a strong overlap in their application to structural fire analysis, a high degree of collaboration between these disciplines is required [17]. The travelling fires methodology developed and presented in this thesis has been formulated with precisely this degree of multidisciplinary cooperation in mind.

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# 2

## Experimental Review of the Homogeneous Temperature Assumption in Post-Flashover Compartment Fires

### 2.1 Introduction

Post-flashover compartment fires are of particular relevance to the analysis of structural fire performance because of their high severity. Traditional methods for quantifying and modelling post-flashover fires for structural engineering analysis assume homogeneous temperature conditions, i.e. the gas phase temperature distribution is taken to be spatially uniform and does not have considerable gradients. For example, the methodologies for structural fire analysis that use the standard and parametric temperature-time curves assume this uniform temperature

regardless of the compartment size or fire power. This assumption has been necessary to develop simple analytical solutions to the temperature evolution and further the understanding of post-flashover compartment fires and subsequent structural responses [1].

However, the accuracy and range of validity of the homogeneous temperature assumption has not been thoroughly examined before. This is generally due to the limited number of post-flashover fire experiments available and especially to the low spatial resolution of temperature measurements used in such tests.

This paper reviews the validity of this assumption using previously conducted fire tests. The tests chosen for these analyses are the Cardington (1999) and Dalmarnock (2006) tests. The choices are based on the detailed instrumentation and the large geometry of the tests. The paper also examines the impact of the departure from the homogeneous temperature assumption on typical thermal analyses that represent the basis behind structural fire calculations.

## **2.2 The Homogeneous Temperature Assumption**

### **2.2.1 Origins of the Assumption**

Most theoretical models for quantifying the temperature evolution in post-flashover fires are based on the assumption of uniform compartment temperatures [2], which is also referred to as the well stirred reactor assumption. This is the case for both analytical models and zone models. Karlsson and Quintiere [1] note that this assumption, among others, is required for an analytical solution of the energy balance for the compartment. In particular they note that the methods of Magnusson and Thelandersson in 1970 [3] and Babrauskas and Williamson in 1978 [4] adopted this approach. The former is the basis for the Eurocode parametric temperature time curve [1]. Drysdale [5] notes that a justification of this assumption often used is that there is supposedly a small gradient in the vertical temperature

distribution during a post-flashover fire and even smaller horizontal gradients. For example, a single test from 1975 is cited showing a nearly uniform vertical temperature distribution at one moment at the onset of flashover [5]. However, this justification has not been evaluated any further. Furthermore, due to the limited number of thermocouple trees in most fire tests (typically one or two), the presence of horizontal gradients cannot be investigated and is rarely reported.

Franssen proposed modifications to the Eurocode parametric temperature-time curve to better correlate the predicted peak temperatures with those from 48 experiments [6]. However, dispersions of the temperature data about the compartment averages for the experiments are not given, presumably because the assumption of temperature uniformity was automatically invoked.

The uniform temperature assumption is fundamentally inherent in the test methods used for classifying structural fire resistance. The fire rating system adopted by most building codes and standards worldwide is based on single elements of construction being subjected to furnace tests in which the gas temperature evolution follows that of a uniform standard fire. It is a key aim of these tests to produce as uniform a temperature field as possible throughout the furnace. Typical furnace tests include about four to nine thermocouple or plate thermometer measurements in different locations. ISO 834 [7] specifies the compartment temperature as the spatial average from all of the thermocouples monitoring the gas phase. The test requires that each individual thermocouple be within 100°C of the standard fire temperature-time curve specified at all times after the initial 10min. The test also requires that the percentage difference between the areas under the measured compartment average and the standard temperature-time curves be within 15% of each other after the first 10min, 10% after 30min, 5% after 60min, and 2.5% thereafter. BS 476 [8] and ASTM E119 [9] have similar tolerances.

The tight tolerances required in standard fire tests are specifically set to ensure that the temperature field in the compartment is uniform. While standard fire curves have been criticised before on many counts for not representing natural fires [5, 6, 10], the spatially homogeneous temperature assumption has not typically been one of them.

## **2.2.2 Critiques of the Assumption**

Harmathy [11] presents a qualitative critique of the homogeneous temperature assumption, also referred to as the well stirred reactor assumption. The critique states that external flaming close to a vent invalidates the well stirred reactor model. Harmathy suggests division of the compartment into three zones to allow mathematical treatment: a zone of primarily fresh incoming air, a zone dominated by the presence of the flame, and a zone behind the flame with mixed pyrolyzates and combustion products. According to this classification, the homogeneous temperature distribution would only be valid in this last zone. However, this critique does not provide any quantification of the heterogeneity or its effects.

Bøhm and Hadvig [12] reported differences in experimental temperature measurements of 200 to 500°C within a single post-flashover fire, with the hottest temperatures in the centre of the compartment. Their test compartment was 4.6m x 4.6m x 2.5m, and temperature measurements were made at eight different locations. The temperature differences led to difficulties in predicting the heat fluxes to both the fuel surface and the exposed structure, but no further analysis was made of the effect of the non-uniformity.

Welch et al. [13] and Abecassis et al. [14] reviewed the experimental data of the Cardington Tests and the Dalmarnock Fire Tests, respectively, in terms of temperature and heat flux fields and concluded they did not support the conventional assumption of uniformity. These tests are described in Section 2.3.3.

## 2.3 Experimental Review

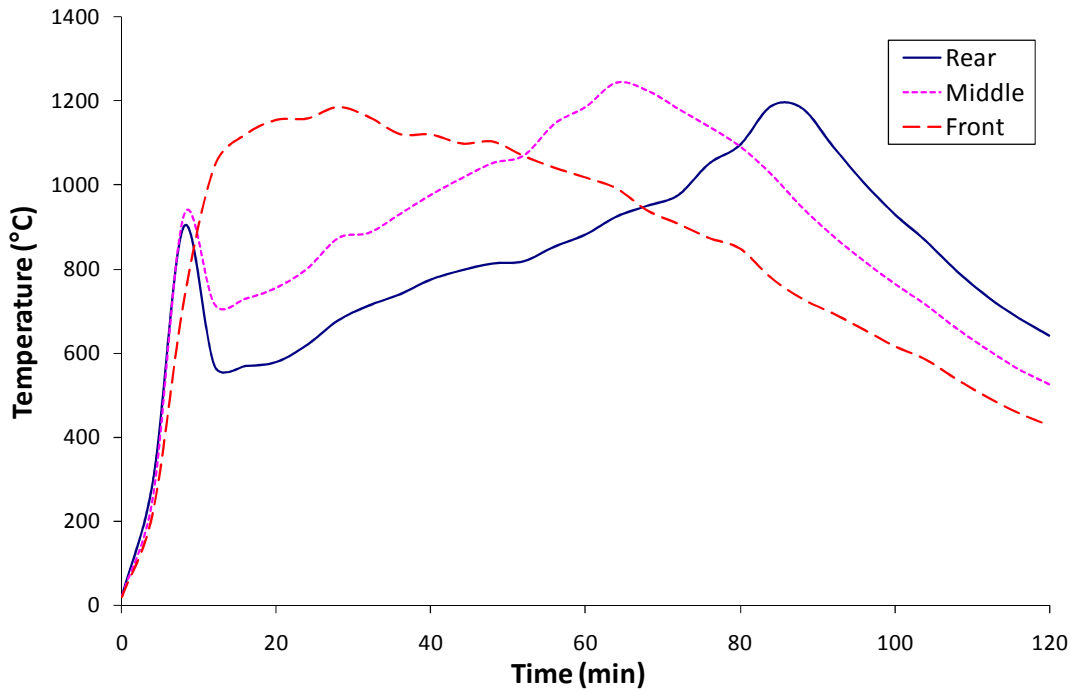
The presence of considerable temperature gradients during post-flashover fires has previously been observed, although not systematically examined. Tests in large or irregularly shaped compartments and real fires can provide insight into the potential dispersion of temperatures and are reviewed here.

### 2.3.1 Non-Uniform Burning in Experiments

Kirby et al. [15] ran a test series burning wood cribs in a long enclosure with approximate dimensions of 22.9m long x 5.6m wide x 2.8m high. All of the tests were ignited at the rear, except one in which all wood cribs were ignited simultaneously. The results of all tests show that the fire moved relatively quickly from the ignition location to the front of the compartment, where the vent was located. After the fuel in the front of the compartment burnt out, the fire progressively travelled back into the compartment and ultimately consumed all the fuel and self-extinguished at the rear. Temperature results of Test 1 from this test series are shown below in Figure 2.1 at the rear, middle and front of the compartment.

Thomas and Bennetts [16] conducted a test series of ethanol pool fires in a small rectangular enclosure (1.5m x 0.6m x 0.6m) to determine the influences of ventilation size and location on the burning rate. They found that there were significant differences in burning rates between having the opening on the short end (long enclosure) or the long side (wide enclosure). They observed temperature differences at different locations up to 500°C, generally with greater temperatures nearer the vents, as this is where the flames resided more often. This work was continued further [17] with another experimental series of pool fires in a larger, long enclosure (8m x 2m x 0.6m), in which the opening size on the short end was varied. The results obtained were similar to both their earlier work [16] and that of Kirby et al. [15].

They conclude that a structural element near the vent would be exposed to more severe conditions than one further inside the compartment.



**Figure 2.1:** Comparison of temperature-time measurements at three different locations, spaced 8m apart, from the rear to the front of the compartment, illustrating non-uniform burning during of wood cribs during the tests of Kirby et al. [15].

### 2.3.2 Travelling Fires

Since the scale of most enclosures in real buildings is significantly larger than the scale in the few experimental tests available, it is likely that even higher degrees of non-uniformity are to be expected in real fires. The real, large fires in the World Trade Center towers 1, 2 [18] and 7 [19] in New York in September 2001, the Windsor Tower in Madrid, Spain in February 2005 [20] and the Faculty of Architecture at TU Delft in the Netherlands in May 2008 [21] were all observed to travel across floor plates. Due to the travelling nature of the fires, it is likely that temperature distributions during these events were highly non-uniform. While no data exist to validate this, extensive numerical simulations conducted for the World

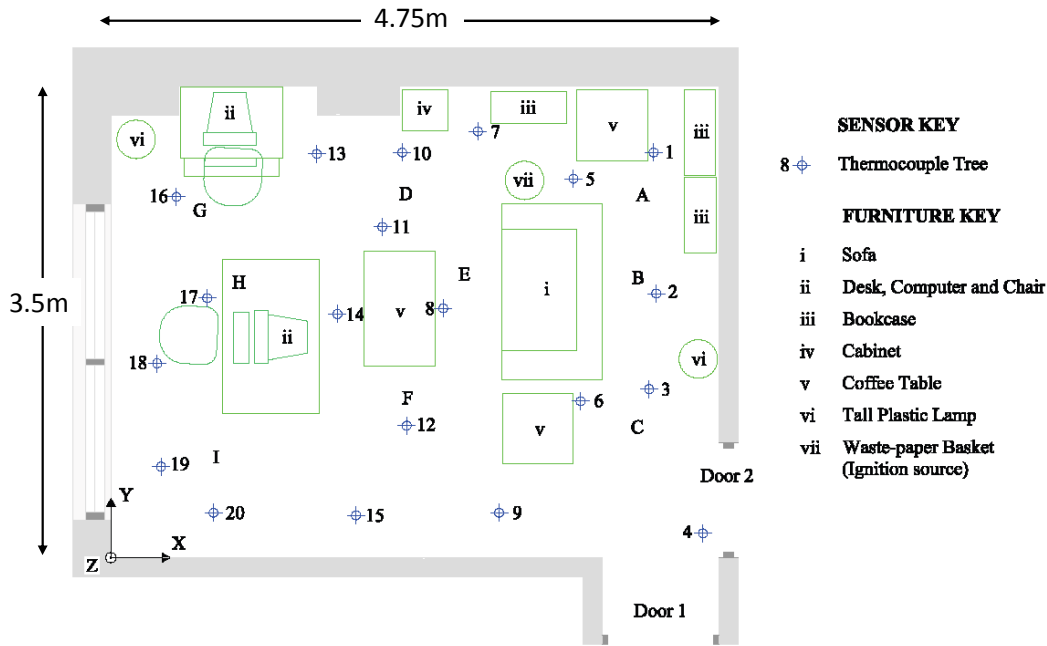
Trade Center investigations by NIST clearly show temperature variations within single compartments of several hundred degrees Celsius [18, 19].

### **2.3.3 Fire Tests with High Spatial Resolution**

Traditionally, most fire tests have only limited spatial resolution in temperature measurements. For example, the series of well ventilated fire tests conducted by Steckler et al. [22], which are often cited in fire model validation studies, monitored the vertical distribution of gas temperatures at only two locations; at the vent and at one internal corner of the compartment. This low spatial resolution cannot provide the necessary insight into the degree of temperature homogeneity and leaves the uniformity assumption unchallenged.

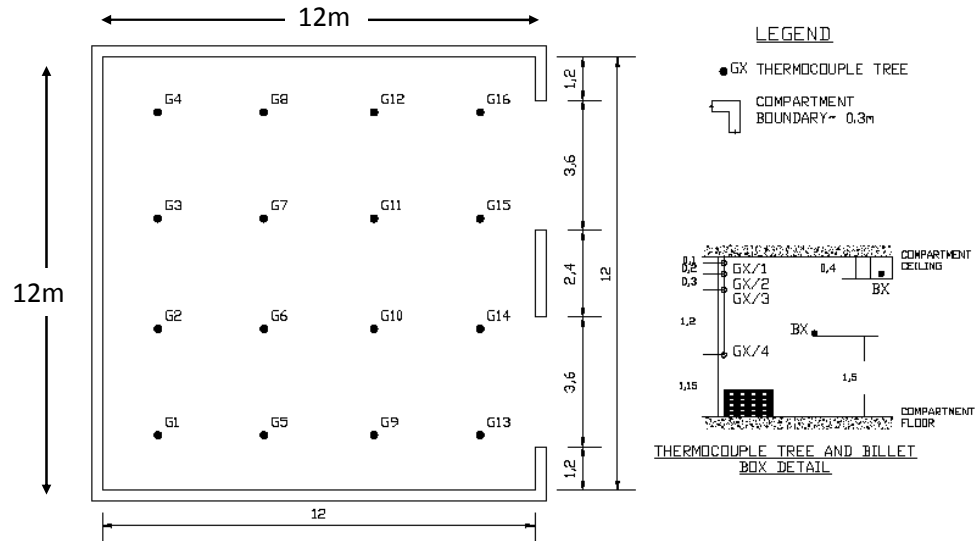
More recent tests, such as the Dalmarnock Fire Tests [23, 14] in 2006 and the Natural Fire Safety Concept 2 test series at Cardington [24, 13] in 1999, have included a much greater spatial resolution of instrumentation. General overviews of these experimental setups are provided here.

The Dalmarnock Fire Tests, which provide the greatest instrumentation density to date, were conducted in a real high-rise apartment building in Glasgow, UK [23, 14]. The two tests conducted had a realistic fuel load of typical residential/office furnishings. The compartment was 4.75m x 3.50m x 2.45m, containing 20 thermocouple trees, each with 12 thermocouples (placed 0, 0.05, 0.1, 0.2, 0.3, 0.4, 0.6, 0.8, 1.0, 1.3, 1.6 and 2m from the ceiling). The Dalmarnock experimental layout is given in Figure 2.2. Ignition occurred in the waste-paper basket adjacent to the sofa. Two tests were conducted, however only Test One is examined as the fire in the second test was manually suppressed before flashover.



**Figure 2.2:** Experimental layout of the Dalmarnock Test One [23, 14]. Locations of the 20 thermocouple trees (each with 12 thermocouples in height) are noted by blue crosses.

The eight Cardington Tests were conducted in a room 12m × 12m × 3m with uniformly spaced fuel load packages distributed across the floor [24, 13]. Sixteen thermocouple trees containing four thermocouples each were placed on a uniform grid in the compartment to record the gas temperatures, shown in Figure 2.3. The tests were conducted with various combinations of fuel type, ventilation distribution, and interior lining material. The tests had liquid fuel channels connecting the fuel packages so that ignition and the subsequent burning could be as uniform as possible.



**Figure 2.3:** Experimental layout of Cardington Tests [24, 13]. Locations of the 16 thermocouple trees (each with 4 thermocouples in height) are noted by black dots.

The Cardington experiments intended to test two types of compartment insulation; “insulating” (I) and “highly insulating” (HI). However, after Test 1 the “highly insulating” material was placed on the ceiling for all remaining tests, creating an intermediate level of insulation (I+). The fuel packages were either just wood cribs (W) or a combination of wood and plastic cribs (W+P). The ventilation openings were either fully open on the front (F) of the enclosure or on the front and back (F+B). A summary of these parameters for all eight tests is given in Table 2.1.

Test Number	1	2	3	4	5	6	7	8
Fuel Type	W	W	W+P	W	W+P	W	W+P	W+P
Insulation Type	I	HI	HI	HI	HI	I+	I+	I+
Opening Location	F	F	F	F+B	F+B	F+B	F+B	F

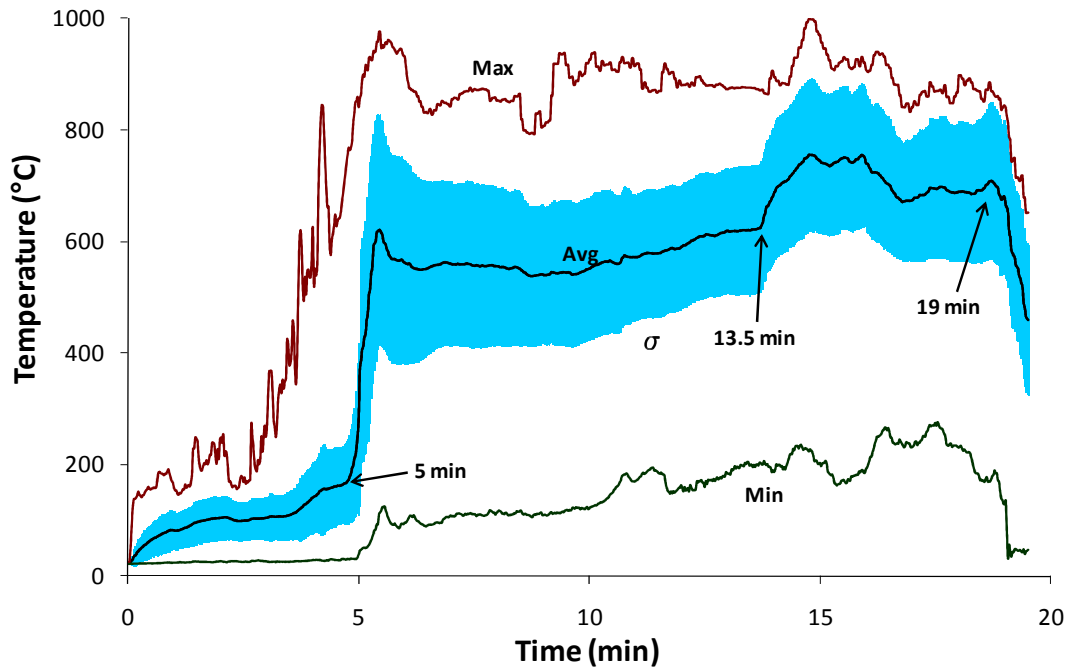
**Table 2.1:** Summary of test conditions in Cardington [24, 13].

Both data sets have a sufficient number of data points to allow for representative statistical analyses. Dalmarnock had 240 points and the Cardington Tests each had 64. The Dalmarnock tests have both well distributed measurement points and a high

density of instrumentation (5.9 thermocouples/m<sup>3</sup>). The Cardington Tests had well distributed measurement points, but not a high density of instrumentation (0.15 thermocouples/m<sup>3</sup>).

The Dalmarnock test data were corrected for thermocouple radiation errors using the method of Welch et al. [13]. The Cardington data have not been corrected. However, Welch et al. [13], using Cardington Test data, report that typically corrections fall in the range of 10 – 40°C, with occasional values as high as 100°C for flame temperatures. Additional calculations were performed using the thermocouple corrections for one of the Cardington Tests to confirm that similar results were obtained to those presented in this study.

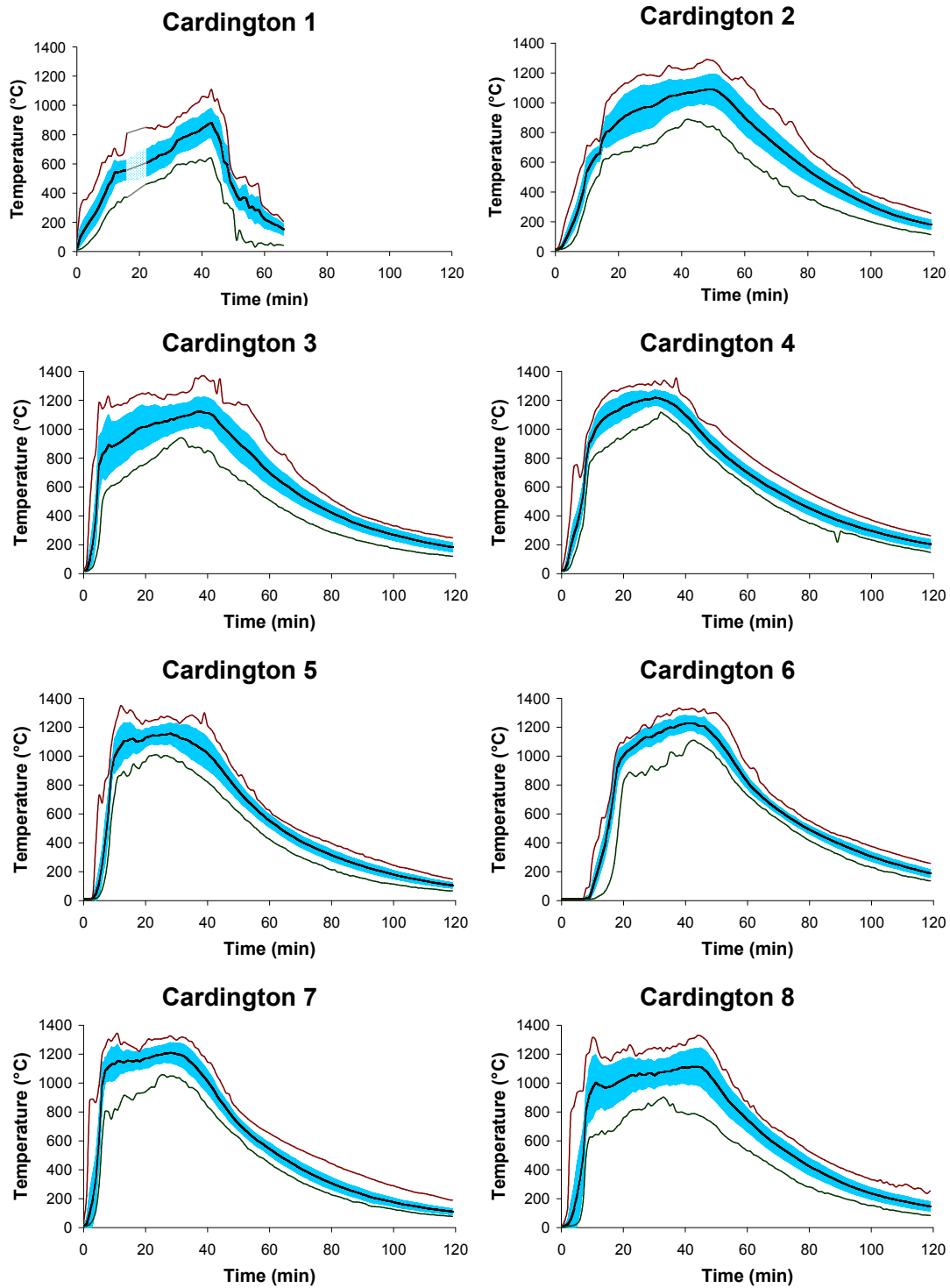
The results from Dalmarnock Test One are given in Figure 2.4. The results are shown with the average compartment temperature and standard deviation in the shaded region, plus the maximum and minimum temperature measurements in the compartment at any given time. Two distinct post-flashover periods can be observed in the Dalmarnock data. The change between the first and second period is caused by window breakage at approximately 13.5 minutes after ignition. The spatial location of the hot and cold spots can be investigated tracking the maximum and minimum temperature curves. Through the test, the maximum temperature was registered at different times in 52 thermocouple locations, distributed over 16 out of the 20 thermocouple trees and all but one of the 12 heights. No particular pattern of where the peak temperatures were located is observed. The minimum temperature was registered at only three different thermocouple locations (thermocouple trees 4, 6, and 18 shown in Figure 2.2) all at the lowest thermocouple (0.45m above the floor). All three locations are near pathways for make-up air to the fire compartment.



**Figure 2.4:** Experimental results of Dalmarnock Test One [23, 14] showing the compartment average, maximum and minimum temperatures, and the standard deviation. Flashover occurred at 5min, window breakage at 13.5min, and the fully developed fire lasted until suppression at 19min.

The results for all eight of the Cardington Tests are shown in Figure 2.5. Note that there was a period between 16 and 22 min of Cardington Test 1 where data collection was temporally lost (interpolation is provided).

The general results are summarised in Table 2.2 which provides the minimum, mean, and maximum standard deviations, as well as the maximum average compartment temperature reached for each test. The standard deviations are only included for portions of the tests where the average compartment temperatures are above 500°C, as the interest of this examination lay in the post-flashover portion of the experiments. Table 2.2 also presents averaged values for two different furnace tests conducted on the same wall assembly to the ASTM E119 standard fire in April 2009 [25]. The tests, carried out at a commercial laboratory to provide a rating for a bespoke wall assembly, included nine gas phase thermocouples.



**Figure 2.5:** Experimental results of the Cardington Tests [24, 13] showing compartment average, maximum and minimum temperatures, and the standard deviation for each test. See Table 2.1 for a summary of conditions for each test.

Test	Min $\sigma$ (°C)	Mean $\sigma$ (°C)	Max $\sigma$ (°C)	Max $T_{avg}$ (°C)
Dalmarnock Test One	105	132	233	733
Cardington 1	38	84	136	857
Cardington 2	31	83	153	1075
Cardington 3	31	100	208	1103
Cardington 4	31	52	93	1199
Cardington 5	18	56	135	1147
Cardington 6	25	44	129	1218
Cardington 7	20	51	159	1200
Cardington 8	32	83	213	1107
Standard Fire Tests	8	12	39	N/A

**Table 2.2:** Summary of the temperature measurements of each spatially resolved fire test and the mean values of two standard fire tests to ASTM E119.

In addition to the values shown in the Table 2.2, peak local temperatures range from 23% (Cardington Test 6) to 75% (Dalmarnock Test One) higher than the compartment average, with a mean peak increase of 38% across all tests. Local minimum temperatures range from 29% (Cardington Test 4) to 88% (Dalmarnock Test One) below the compartment average, with a mean local minimum temperature of 49% across all tests.

Higher mean standard deviations are observed in Dalmarnock Test One (132°C) than all of the Cardington Tests (mean of 70°C). This is to be expected for several reasons:

- Dalmarnock Test One had a much higher density of instrumentation than the Cardington Tests, making it more likely that the full range of temperature conditions was recorded.
- The thermocouple layout in Dalmarnock Test One covered regions with fuel packages and regions remote from fuel packages. In Cardington, all thermocouples were located above fuel packages and thus the data have a bias towards flame temperatures.

- There were only four different thermocouple heights in the spacing of the Cardington Tests, all relatively high, compared to the twelve in Dalmarnock, which were evenly distributed. Thus the Cardington data are biased towards temperatures in the upper portion of the compartment.
- The Dalmarnock Test had a realistic fire scenario where real-world furnishings were arranged in a non-uniform manner and one ignition point was used. In contrast, the Cardington Tests had well distributed fuel packages ignited simultaneously.

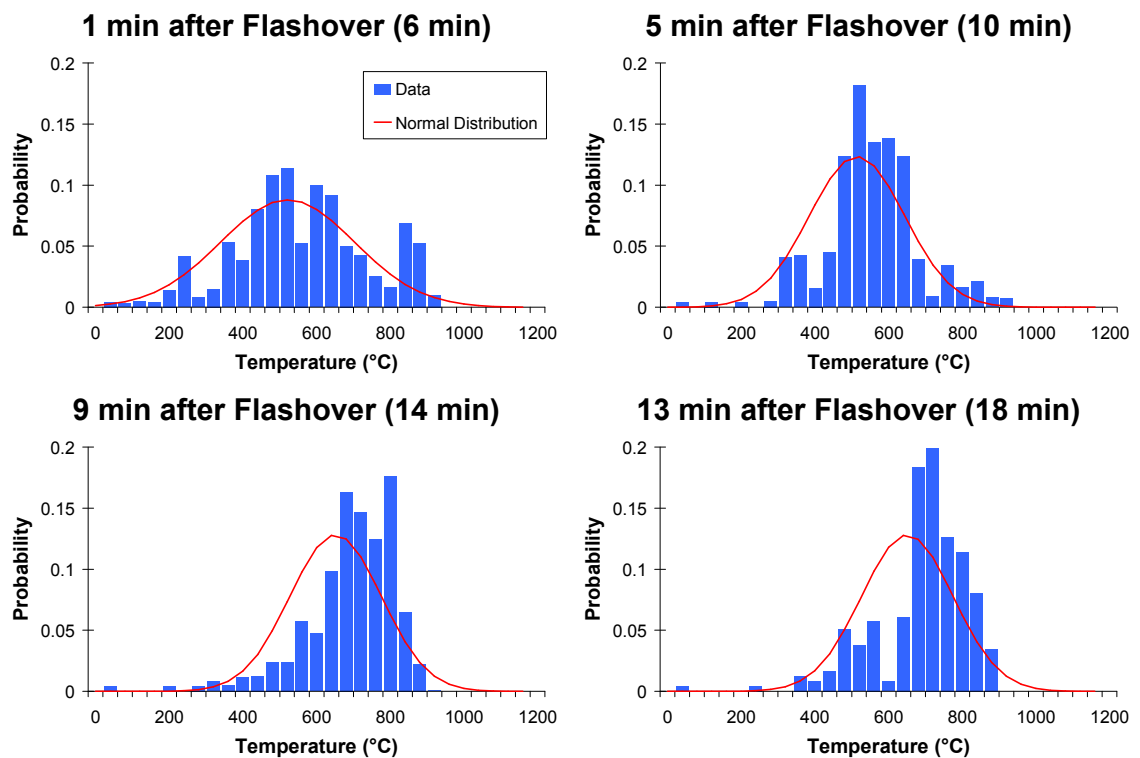
A clear trend can be seen in the results from Cardington. Tests 4 through 7 all have lower standard deviations (mean of 51°C) than Tests 1, 2, 3, and 8 (mean of 88°C). The key difference between the two groups of tests is the ventilation position. Tests 1, 2, 3, and 8 had ventilation only on one side of the compartment, while Tests 4 through 7 had ventilation at two opposing sides. This fact is in line with the results obtained by the studies previously highlighted with long enclosures [15, 16, 17]. Thus there is heterogeneity in the temperature field due to the depth of the compartment relative to the position of the vents. This effect is less obvious for the tests with ventilation on opposing sides.

These results confirm that there is considerable heterogeneity in the temperature field of post-flashover fires. Real world fires are likely to have a level of dispersion in the temperature field closer to that measured in Dalmarnock Test One than those of the Cardington Tests. This is because the high density of instrumentation in Dalmarnock recorded more of the temperature field than those in the Cardington Tests, thus a more complete depiction of the variation was established. Furthermore, the fuel types and distributions of real world fires that can cause heterogeneity are more likely to match those of Dalmarnock than the uniformly spaced cribs of Cardington.

It is also worth noting that the tests examined were conducted in compartments of dimensions that are consistent with the homogeneous temperature assumption. Thus other compartments with larger or more complex geometries will show broader temperature dispersions.

### 2.3.4 Data Distributions

Examination of the statistical distributions of the data from each test provides more insight into the level of uniformity of the temperature field. Figure 2.6 presents the data distributions for four different times of Dalmarnock Test One with the corresponding normal distribution overlaid. The distributions are shown at four times, evenly spaced between flashover and suppression. The temperature measurements are grouped into 40°C bands, as to encompass the experimental uncertainty. If the homogeneous temperature assumption held, there would only be one bar at any given time.



**Figure 2.6:** Comparisons of the measured temperature distributions against the associated normal distributions after flashover for Dalmarnock Test One.

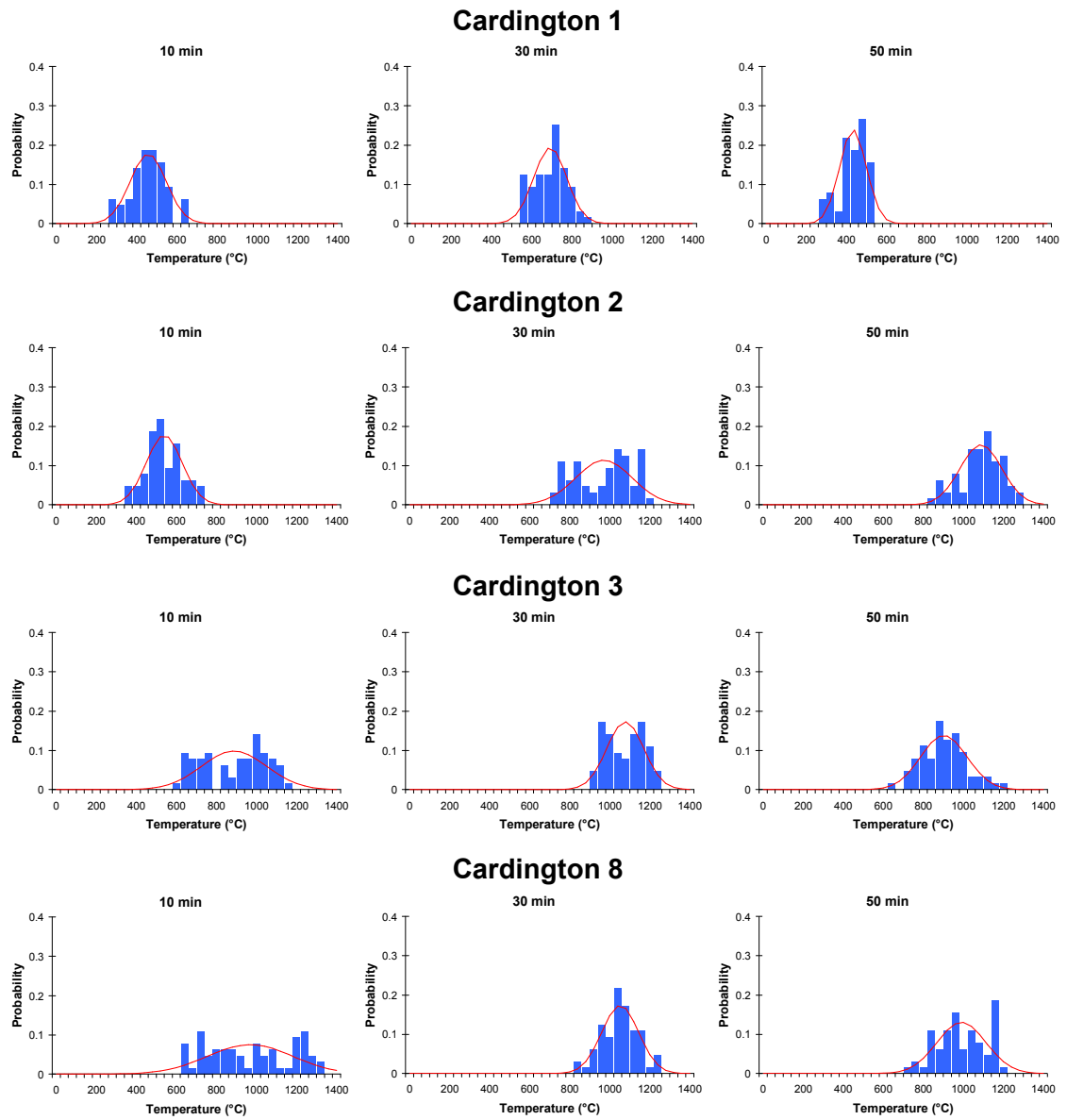
Figure 2.7 and Figure 2.8 provide details for the data distributions of the Cardington Tests. Figure 2.7 presents the data distributions for the four Cardington Tests with ventilation at one side only (F), while Figure 2.8 presents the data distributions with ventilation on opposing sides (F+B). The F data show a greater span in the distributions than the F+B data.

The test data have been presented with standard deviations as a measure of the departure from uniform temperature conditions. For a simplified estimation of the meaning of the standard deviation, it is noted that approximately 65% of all data fall within the span between one standard deviation on either side of the average and approximately 95% fall within the same span of two standard deviations.

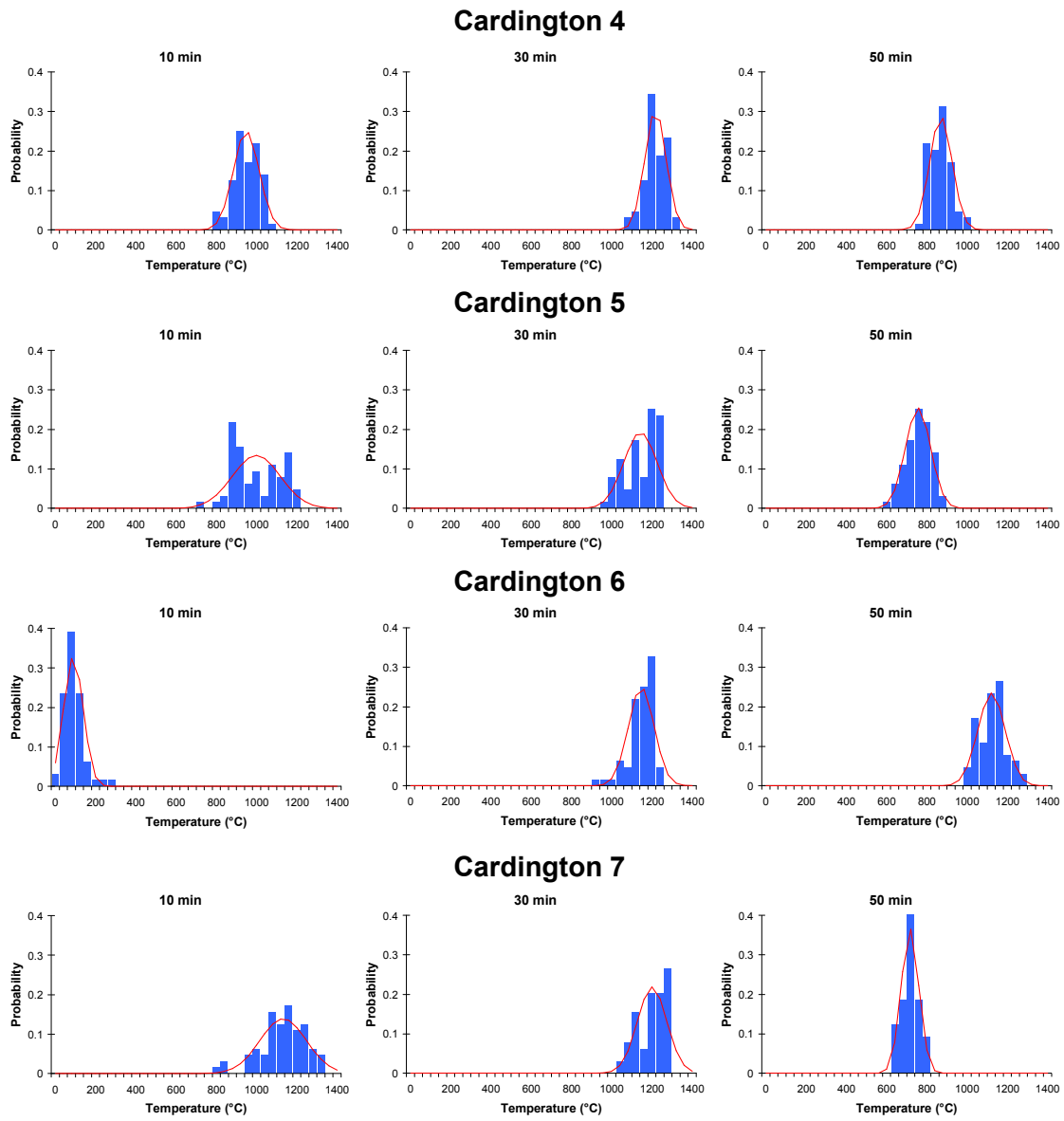
While the data distributions shown in Figures 2.6 through 2.8 do not always fit normal distributions, at most times for most tests they are sufficiently close to treat the data as normally distributed for the purposes of this analysis.

### **2.3.5 Standard Deviation vs. Temperature Rise**

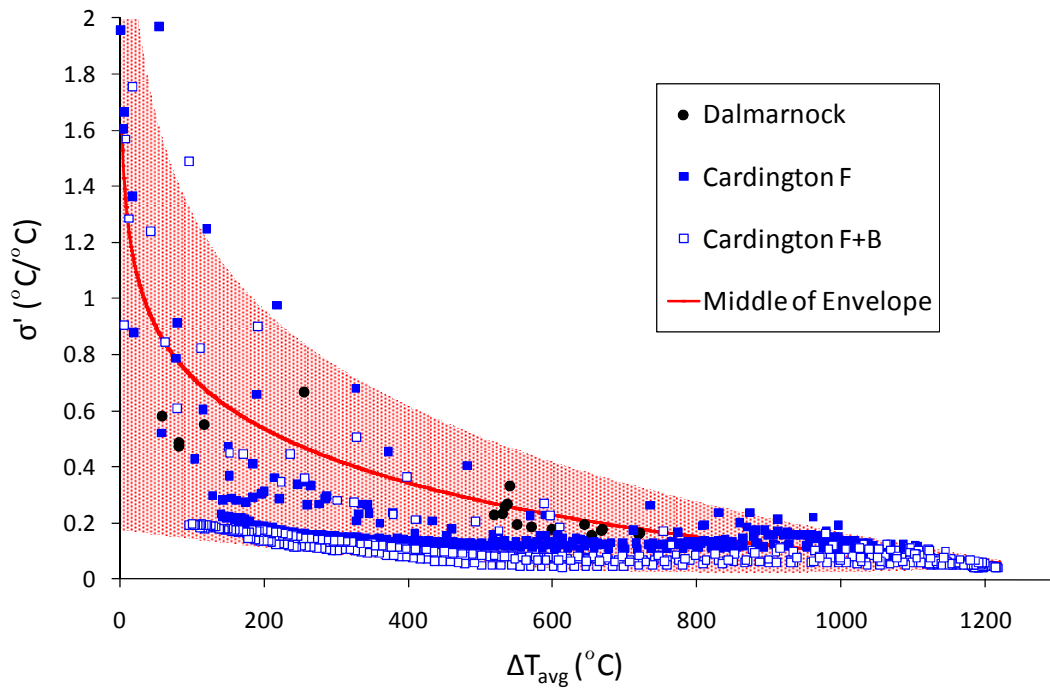
Figure 2.9 shows the relationship between the normalised standard deviation,  $\sigma'$ , and the average temperature rise from ambient,  $\Delta T_{avg}$ . Each data point represents one instant in time, with one point taken every minute for each test. The normalised standard deviation,  $\sigma'$ , is defined as the standard deviation divided by the average compartment temperature rise above ambient,  $\Delta T_{avg}$ . The Cardington Tests have been divided into the two ventilation groups previously noted. Cardington F is the group with ventilation in the front only and Cardington F+B is the group with ventilation from both the front and back.



**Figure 2.7:** Comparisons of the measured temperature distributions against the associated normal distributions for Cardington Tests with ventilation on one side only (Tests 1, 2, 3 and 8).



**Figure 2.8:** Comparisons of the measured temperature distributions against the associated normal distributions for Cardington Tests with ventilation on opposing sides (Tests 4, 5, 6 and 7).



**Figure 2.9:** Observed relationship between the normalised standard deviation vs. temperature rise in the spatially resolved fire tests available.

These results indicate that there are significant heterogeneities in the gas field across the whole range of temperatures. Furthermore, the scatter shows a clear trend; the higher the temperature, the lower the normalised standard deviation. The maximum temperature rise, just above 1200°C, marks the peak flame temperature rise above ambient, which is at the upper end of temperature rises possible in a typical post-flashover fire. More intense fires lead to hotter and more uniform conditions in their enclosures, whereas in less intense fires the flame and smoke regions dominate less of the gas field and less uniformity is observed. A clear difference can be seen in the ventilation effect between the two groups from the Cardington Tests, with the Cardington F Tests having less homogeneity than Cardington F+B Tests. Also the greater degree of heterogeneity from the Dalmarnock test can be seen.

The shaded region represents an approximate envelope for all of the data points. The best fit equation for the curve that runs through the middle of this envelope is given in Eq. (2.1).

$$\sigma' = \frac{\sigma}{\Delta T_{avg}} = 1.939 - 0.266 \ln(\Delta T_{avg}) \quad (2.1)$$

This curve could be used as a nominal expression of the standard deviation for any temperature-time curve. The shaded envelope could be expected to apply to fires in compartments of similar sizes as those assessed in this paper. For fires in compartments of a much larger size, such as the real ones previously cited [18, 19, 20, 21], the temperature field will likely be much more non-uniform and a travelling fire should be expected. A general discussion of the temperature fields in travelling fires is available in the literature [26, 27].

The middle of the envelope has been used in lieu of a regression analysis because the data are biased towards the Cardington Tests due to the large number of data points for each test. There were eight Cardington Tests and each lasted longer than Dalmarnock Test One. Therefore the shaded envelope was used to eliminate any bias towards the Cardington data. For the reasons already discussed, the Cardington data are deemed inappropriate to express standard deviations for a general, real fire scenario.

## 2.4 Effect of Temperature Heterogeneity on the Structure

Structural fire resistance calculations are routinely based on averaged temperature values determined on the basis of standard fire tests conducted in test furnaces which are explicitly intended to ensure uniform gas phase temperatures. However, recent studies have shown that the behaviour of certain structural elements are

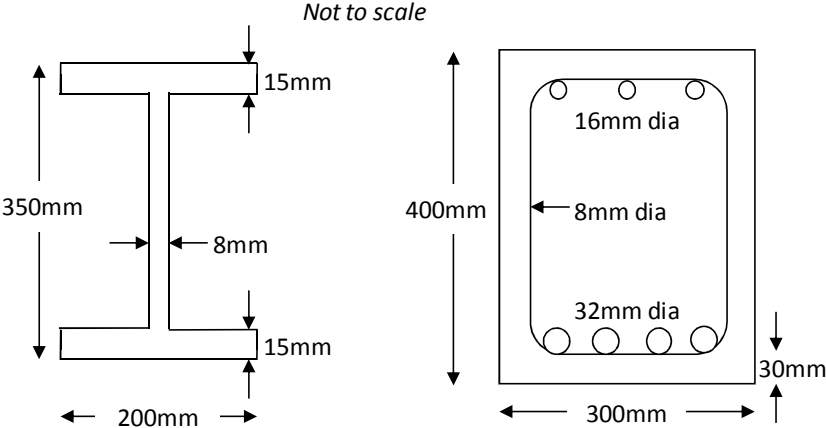
affected by temperature gradients [28, 29], thus there is a motivation to revisit the homogeneous temperature assumption. Moreover, the experimental results analysed above are at odds with the traditional assumption of temperature uniformity, thus the effect of this heterogeneity on the heating of structural elements is reviewed here.

A simplistic method for assessing the impact of non-uniform temperature distributions on single structural elements has been adopted. These calculations are intended to provide insight into the performance of simple structures and are not proposed to be a design methodology or calculation guideline. Further research is required to determine true structural response to non-uniform heating as the analysis of the fire test data indicates that the use of a uniform temperature distribution does not capture the true thermal environment of a real fire. Therefore these simplistic calculations have only been adopted for illustrative purposes, to examine trends for structures heated to temperatures above the compartment average.

It is important to clarify that the impact of non-uniform temperature distributions on full structural behaviour is not being assessed here, nor issues associated with details of heat transfer such as soot concentrations or velocities. While these details will have an impact on the heating of structural elements, they are not usually part of standard thermal calculations for the purposes of structural fire analysis.

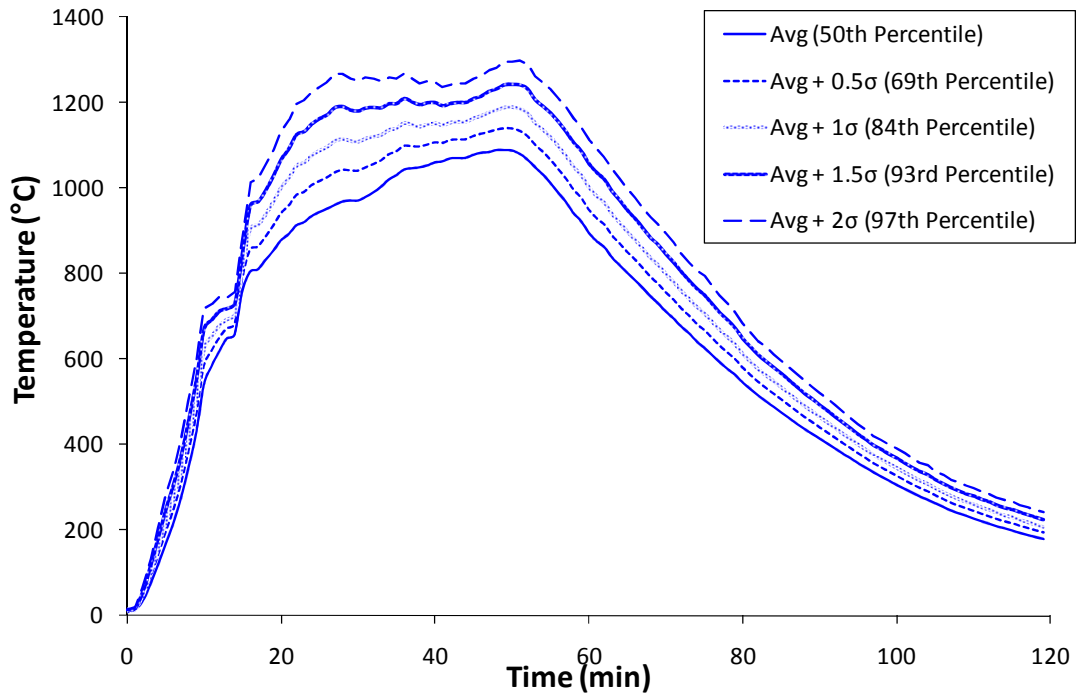
For illustrative purposes, three simplified examples of structural elements are used: (1) an unprotected steel I-beam, (2) a protected steel I-beam fire rated to 60 min using a generic insulation, and (3) a concrete beam with a 60 min fire rating. All three beams, with dimensions given in Figure 2.10, nominally have the same design bending moment capacity under ambient temperature. The beams selected for the analysis are representative of typical beams covering the most common construction types and range of thermal inertias found in real buildings. The unprotected and

protected steel beams have the same dimensions, except that an additional layer of fire protection is applied to the protected beam (12 mm of high density perlite insulation). It is assumed that a concrete floor slab is present above the beams such that they are only heated on three sides.



**Figure 2.10:** Dimensions of the steel (left) and concrete (right) beams used to determine representative structural responses to the varying temperature distributions.

The thermal response of each beam was calculated for a variety of temperature-time curves above the mean. This information was used in conjunction with thermal definitions of fire resistance based on assumed critical temperatures for each material. Each curve was generated from each experimental data set, starting with the average compartment temperature-time curve, and then adding a fraction of the standard deviation to it, in units of one quarter of the standard deviation. Thus, the first curve analysed for each beam from a given experiment was the average compartment temperature-time curve. The next curve used was the average compartment temperature-time curve plus one quarter of the standard deviation, then the average compartment temperature-time curve plus one half of the standard deviation, and so on until the average compartment temperature-time curve plus two times the standard deviation. Figure 2.11 illustrates this by showing every second curve used for Cardington 2.



**Figure 2.11:** Temperature-time curves for Cardington Test 2, ranging from the average temperature-time curve (representing the 50<sup>th</sup> percentile) to the average temperature-time curve plus two standard deviations (representing the 97<sup>th</sup> percentile). Note that this plot only shows every other curve used for structural assessment.

This approach allows the results to be viewed continuously from the average compartment temperature-time curve through to the average compartment temperature-time curve plus two standard deviations. This span, if viewed cumulatively, covers the range between the 50<sup>th</sup> percentile and the 97<sup>th</sup> percentile.

Only values above the mean have been analysed here. This is to focus on the possibility of current design practices underestimating the effect of fire on structures by use of the average compartment temperature only. The non-uniformity will also result in some elements of structure exposed to less severe conditions than currently assumed using the compartment average. This is not considered here, as a common aim of structural fire engineering is to err on the side of conservatism.

From the percentile temperature-time ranges developed, the peak temperature rise and time to failure, based on an assumed critical temperature, were calculated for each beam and each fire test as a function of the temperature percentile. The unprotected steel beam temperature was calculated by lumped mass heat transfer, as given by Buchanan [30]. The protected steel beam temperatures were also calculated by the lumped mass method given by Buchanan. For the concrete beam, the temperature calculated was that of the internal steel reinforcing bars, assumed to be at the same temperature as the concrete adjacent to it, i.e. the temperature at the extreme underside of the bars. This in-depth temperature of the concrete was calculated with a one-dimensional finite-difference method in explicit form, as given by Incropera et al. [31].

The time to failure is taken as the time for the steel to heat to 550°C, as this is normally considered an approximate temperature above which steel loses sufficient strength such that failure of a typical simply-supported beam could occur under the loads assumed to be applied during a fire [5]. Higher temperatures are sometimes used; however 550°C is selected here for the purpose of these calculations, for both steel beam and rebar temperatures.

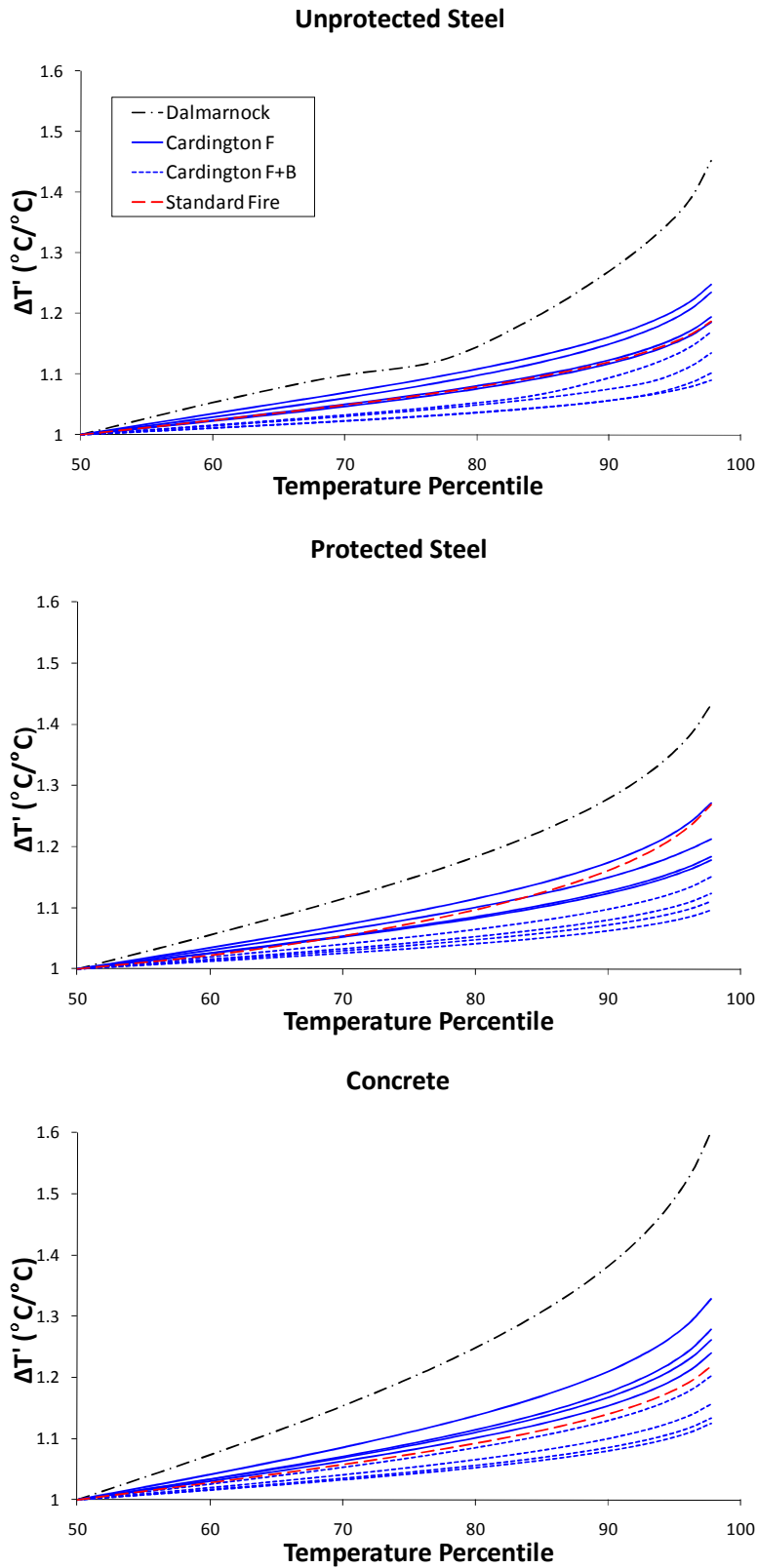
A full description of the calculation methods used is given in Appendix A. It is acknowledged that the calculations and failure criterion are simplistic, and it is important to note that the illustrative approach taken herein does not account for several important issues related to the heating and ultimate response of the structure.

Normalised results for the maximum temperature rise reached against the temperature percentile are shown in Figure 2.12 for all three beam types. The normalised temperature rise,  $\Delta T'$ , is defined as the steel temperature rise when exposed to the given temperature percentile curve divided by the steel temperature rise when exposed to the average temperature-time curve. The resultant hotter steel

temperature would not be calculated if only the average compartment temperature were considered. The standard fire is included using the normalised standard deviation in Eq. (2.1) to generate the full range of temperature-time curves. For guiding purposes, note that if the gas phase were completely homogeneous, a horizontal line at ordinate 1 would be shown.

The results show that the increased temperatures associated with the non-uniformity have a potentially important impact on the structural performance of the beams analysed. Tables 2.3 through 2.5 show the results for temperature rise and time to failure for the 80<sup>th</sup> percentile temperature-time curves (equivalent to the average compartment temperature-time curve plus 0.85 times the standard deviation) for each experiment and the standard fire when compared to the average compartment temperature-time curve. Note that 80<sup>th</sup> percentile values are often recommended in fire safety engineering for design. For example, in the UK PD7974 recommends fire loads for structural fire analysis to be the 80<sup>th</sup> percentile values [32].

Compared to the calculations using the average compartment temperature measurements, the results at the 80<sup>th</sup> percentile show that a higher temperature region in a compartment could result in a steel temperature rise up to 25% higher (15% for the unprotected steel beam, 18% for the protected steel beam, and 25% for a concrete beam) or reach the time to failure, i.e. the fire resistance time, up to 31% faster (31% for the unprotected steel beam, 15% for the protected steel beam, and 22% for the concrete beam). For the 95<sup>th</sup> percentile, temperature rises can be up to 60% higher and fire resistance times 55% shorter.



**Figure 2.12:** Results of the normalised temperature rise for each type of beam analysed. Note that a horizontal line at abscissa 1 would represent a homogeneous temperature field.

Test	Temperature Rise		Time to Failure	
	Difference	% Increase	Difference	% Decrease
Dalmarnock Test One	96°C	15%	3.8 min	26%
Cardington 1	91°C	11%	4.5 min	21%
Cardington 2	87°C	8%	1.0 min	6%
Cardington 3	84°C	8%	1.1 min	15%
Cardington 4	44°C	4%	0.5 min	5%
Cardington 5	61°C	5%	0.5 min	5%
Cardington 6	44°C	4%	0.7 min	4%
Cardington 7	59°C	5%	0.5 min	8%
Cardington 8	109°C	10%	0.9 min	9%
Standard Fire	81°C	8%	3.1 min	31%

**Table 2.3** Summary of the unprotected steel beam results for temperature rise and time to failure for the 80<sup>th</sup> percentile temperature-time curve.

Test	Temperature Rise		Time to Failure	
	Difference	% Increase	Difference	% Decrease
Dalmarnock Test One	30°C	18%	Did not fail	Did not fail
Cardington 1	43°C	12%	Did not fail	Did not fail
Cardington 2	51°C	8%	5.2 min	10%
Cardington 3	59°C	10%	5.6 min	12%
Cardington 4	29°C	5%	3.1 min	7%
Cardington 5	36°C	6%	6.4 min	13%
Cardington 6	25°C	4%	2.6 min	5%
Cardington 7	31°C	5%	3.7 min	9%
Cardington 8	52°C	9%	5.3 min	11%
Standard Fire	71°C	10%	8.9 min	15%

**Table 2.4** Summary of the protected steel beam results for temperature rise and time to failure for the 80<sup>th</sup> percentile temperature-time curve.

Test	Temperature Rise		Time to Failure	
	Difference	% Increase	Difference	% Decrease
Dalmarnock Test One	47°C	25%	Did not fail	Did not fail
Cardington 1	53°C	14%	Did not fail	Did not fail
Cardington 2	60°C	10%	7.0 min	13%
Cardington 3	67°C	12%	6.5 min	15%
Cardington 4	34°C	6%	2.8 min	8%
Cardington 5	48°C	9%	5.6 min	14%
Cardington 6	33°C	5%	2.5 min	6%
Cardington 7	40°C	7%	3.0 min	9%
Cardington 8	66°C	11%	6.7 min	15%
Standard Fire	63°C	9%	15.3 min	22%

**Table 2.5** Summary of the concrete beam results for temperature rise and time to failure for the 80<sup>th</sup> percentile temperature-time curve.

With respect to the heat transfer analysis, the methods used are analogous to those employed for uniform temperature fields, but because they are applied to a range of temperature-time curves above the compartment average, the cumulative results provide insight into the possible heating from heterogeneous temperature fields. It is noted that fully spatially resolved heat transfer analyses, as described by Jowsey [28], were not conducted. That type of analysis could be applied to calculate the non-uniform heating from a heterogeneous temperature field, but requires spatially resolved optical properties and velocities of the combustion gases, which were not available for all of the tests reviewed in this paper.

In terms of the structural behaviour, only a single element has been considered with a fixed temperature representing the failure criterion, thus the method ignores a range of possible structural behaviours including axial restraint, membrane actions, and flexural continuity over multiple spans in a real building. Many more detailed methods and criteria exist to determine the impact of fire on structures for defining their fire resistance [30]. However, given that generic structural elements are being assessed for illustrative purposes only, the current analysis provides useful insights.

Although not assessed here, the location of the thermal non-homogeneities along a structural member is potentially important, since localised heating in regions of lower applied stresses may be less critical for structural performance than in regions of high applied stress. A more detailed structural analysis accounting for thermal non-homogeneities would be required to investigate the potential impacts of non-uniform heating on full frame response to fire.

## **2.5 Conclusions**

The statistical analyses of the fire tests examined show that there is considerable non-uniformity in the temperature fields of real post-flashover fires. Peak local temperatures range from 23% to 75% higher than the compartment average, with a

mean peak increase of 38%. Local minimum temperatures range from 29% to 88% below the spatial average, with a mean local minimum temperature of 49% below the compartment average. This is in contrast to the common assumption of a homogeneous temperature field often used in quantification and modelling of post-flashover compartment fires.

The contradictions between the assumption of homogeneity and measured heterogeneity means that fire tests with limited spatial instrumentation, which are often only reported as average temperature measurements, may lead to erroneous conclusions. If fire tests are not well instrumented, it may be difficult to determine which portion of the temperature distribution has been measured and which parts were not recorded. It has been shown here with the data from the most densely instrumented experiments to date that this range is on the order of hundreds of degrees Celsius.

This heterogeneity can have a potentially non-negligible impact on the structural fire resistance of steel or concrete beams. This is noticeable in increased structural temperatures (up to 25% higher) and shorter times to failure (up to 31% faster) at the 80<sup>th</sup> percentile values compared to those that would be calculated assuming the average compartment temperature. These results along with the recent studies showing some structural elements are adversely affected by temperature gradients gives motivation to revisit the homogeneous temperature assumption and further explore its ramifications.

While the full implications of the temperature heterogeneity of post-flashover fires are not explored here, it is apparent that post-flashover fires do not reach uniform conditions. The presented results highlight the need to increase the spatial resolution of measurements in fire experiments to capture the full variation within the compartment. Spatially resolved data can lead to a rational statistical approach

to fire behaviour when applied to fire safety and structural engineering applications.

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# 3

## A Review of Travelling Fires in Structural Analysis

### 3.1 Introduction

As architectural trends change and become more ambitious, they challenge the bounds of traditional engineering methods. This is true for structural fire engineering, where the fire scenarios most commonly used for the design of modern buildings are based on traditional methods that assume uniform burning and homogeneous temperature conditions throughout a compartment, regardless of its size.

However, close inspection of accidental fires in large, open-plan compartments reveals that they do not burn simultaneously throughout the whole enclosure. Instead, these fires tend to move across floor plates as flames spread, burning over a limited area at any one time. These fires have been labelled “travelling fires”.

The uniform burning and homogeneous temperature assumptions are at the root of many of the existing methods' limitations and have not been confirmed experimentally for large compartments. The traditional methods were developed based on small scale tests and, while they are known to be of some validity for small compartments, cannot be readily applied to large enclosures.

Developing new methods to enhance optimisation of structural fire design, by obtaining a more accurate characterisation of actual building performance, requires a realistic definition of potential fire scenarios. Specifically, incorporation of travelling fires will be necessary to reflect the state-of-the-art knowledge of fire dynamics in large spaces.

This paper reviews research focused on travelling fires in structural analysis. It highlights the historical developments as well as current uses and examines both the definition of the thermal environment as well as structural analyses based on travelling fires.

## **3.2 Traditional Design Methods**

The earliest attempts of testing to understand structural performance in fire led to the standard temperature-time curve, first published in 1917 [1]. This curve and associated test methods given in standards, such as BS 476 [2], ISO 834 [3], and ASTM E119 [4], have formed the basis for the fire rating systems in most building codes and standards worldwide. The curve came from collating the results of various post-flashover fire tests into one idealised curve. The tests that fed into the development of the standard fire were intended to represent worst case fires in enclosures to determine if the structure could withstand burnout. However, these tests were conducted and the standard fire created prior to much scientific understanding of fire dynamics. Thus the standard fire, unlike a real fire, has a

relatively slow growth rate (which was largely driven by the usage of furnaces heated by manually stoked wood fuel [1]), never reduces in temperature due to fire decay, and is independent of building characteristics such as geometry, ventilation and fuel load [1, 5, 6]. Furthermore, the standard fire does not accurately reflect the nature of real fires which do not uniformly heat building elements [7].

As fire science matured, models of post-flashover fire behaviour were developed to account for a better understanding of compartment fire dynamics based on tests conducted in small enclosures. Most of the theoretical models developed were based on the assumption of uniform compartment temperatures [8]. This is the case for both analytical models and zone models. Karlsson and Quintiere [9] note that this assumption, among others, is required for an analytical solution of the energy balance for the compartment. In particular they note that the methods of Magnusson and Thelandersson in 1970 [10] and Babrauskas and Williamson in 1978 [11] adopted this approach.

Pettersson et al. [12] developed a design guide, based on the work of Magnusson and Thelandersson [10], for specifying the thermal environment to be used for structural design. The guidance document provides a set of temperature-time curves for various compartment ventilation factors, fuel loads, and compartment linings. This work was further developed by Wickström [13] and became the basis for the Eurocode parametric temperature-time curve [14], which is a widely used method in structural fire engineering today.

While other methods exist [15, 16, 17, 18], they all assume homogeneous temperature conditions and uniform burning throughout the fire compartment. Drysdale [5] notes that a justification of the homogeneous temperature assumption often used is that there is supposedly a small gradient in the vertical temperature distribution during a post-flashover fire and even smaller horizontal gradients. For example, a single test from 1975 is cited showing a nearly uniform vertical

temperature distribution at one moment at the onset of flashover. Section 3.3 of this paper presents further critiques of this assumption.

While most of the traditional methods tend to look at full compartment involvement, some methods have been developed to look at localised fires [14, 19] that look at the impact of a fire on only part of a structure. Considering localised fires is a relaxation of the uniform burning assumption. This is relevant to this review paper, as a travelling fire is, in essence, a localised fire that moves. However, the methods developed for localised fires have not considered elevated smoke temperatures away from the fire as methods for travelling fires have (see Section 3.4).

Buchanan [20] and Law et al. [21] provide concise histories of the development structural analysis methods for buildings exposed to fires. What is of relevance to this review is the move from solely analysing single elements to that of whole frame behaviour, which was largely driven by specific accidental fires and large scale testing at Cardington. Travelling fires, which provide highly non-uniform and transient heating in time over the full length of a large compartment, may have a considerable impact on whole frame structural behaviour.

### **3.3 Limitations of the Uniform Burning Assumption**

The traditional methods have known limitations in their application. For example, Eurocode 1 states that the parametric curves are only valid for compartments with floor areas up to 500m<sup>2</sup> and heights up to 4m, the enclosure must also have no openings through the ceiling, and the compartment linings are restricted to having a thermal inertia between 1000 and 2200J/m<sup>2</sup>s<sup>1/2</sup>K, which means that highly conductive linings such as glass façades and highly insulating materials cannot be taken into account. As a result, common features in modern construction like large enclosures,

high ceilings, atria, large open spaces, multiple floors connected by voids, and glass façades are excluded from the range of applicability of the current methodologies.

A recent survey of buildings in Edinburgh, UK [22], underlines the implications of these limitations on the applicability of design fires, particularly for modern structures. For buildings built over a long period of time starting in the early 20<sup>th</sup> century, 66% of their total volume falls within the limitations. However, in a newly constructed, modern building that has open spaces and glass façades, only 8% of the total volume is within the limitations. This suggests that modern building design is increasingly producing buildings that contain compartments to which parametric fires should not be applied.

Furthermore, as noted in Section 3.2, the traditional design methods for specifying the thermal environment for structural analysis are based on an assumption of uniform burning and temperature conditions. Stern-Gottfried et al. [23] have reviewed this assumption by analysis of existing experimental data from well-instrumented fire tests. Results show that dispersion from the spatial compartment average is significant and that the assumption of uniform temperature conditions does not hold well (see Section 3.3.1 for more details). While this review was conducted for relatively small enclosures, the findings are likely to be more relevant for large enclosures as the compartment size increases, the degree of heterogeneity is expected to increase.

It is worth noting that the traditional methods assume that worst case conditions are caused by ventilation controlled fires. However, a recent review by Majdalani and Torero [24] of early CIB tests and the resulting analyses of compartment fire behaviour done by Philip Thomas and others highlights that ventilation controlled fires are unlikely in large enclosures and that they are not necessarily more conservative for structural analysis than fuel bed controlled fires. Majdalani and Torero note that while the different burning behaviour between ventilation and fuel

bed controlled fires was clearly stated in the original studies, ventilation controlled fires have nonetheless been assumed to be the most severe case for design.

Buchanan [6] notes that post-flashover fires in open plan offices are unlikely to burn throughout the whole space at once. Although limited experimental data exist on fire spread and temperature homogeneity in large enclosures, examination of specific tests and the study of accidental fires can provide insight into the fire dynamics of larger enclosures.

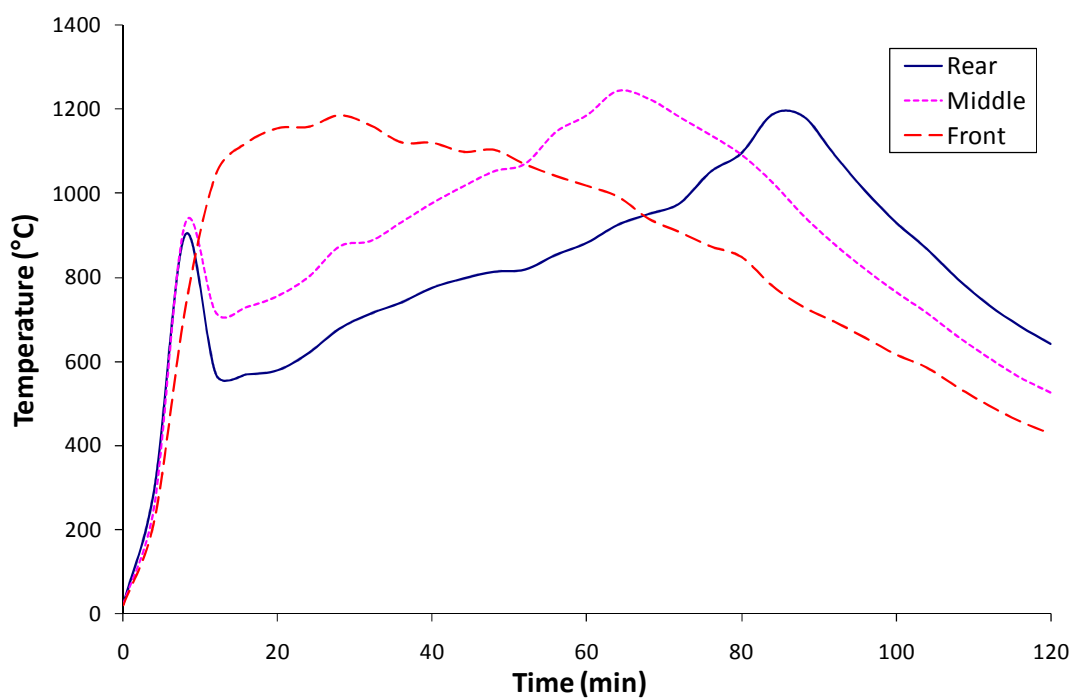
### **3.3.1 Evidence from Experiments**

The well instrumented tests conducted at Dalmarnock [25] and Cardington [26] were shown to have large standard deviations (in excess of 200°C at times) within the temperature field [23]. Additionally, peak local temperatures in these tests were found to vary from 23% to 75% above the compartment spatial averages, and local minimums ranged from 29% to 88% below the averages.

Kirby et al. [27] ran a test series burning wood cribs in a long enclosure with approximate dimensions of 22.9m x 5.6m x 2.8m. All of the tests were ignited at the rear of the compartment, except one in which all wood cribs were ignited simultaneously. The results of all tests showed that the fire moved relatively quickly from the ignition location to the front of the compartment, where the vent was located. After the fuel in the front of the compartment burnt out, the fire progressively travelled back into the compartment and ultimately consumed all of the fuel and self-extinguished at the rear. Temperature results at the rear, middle and front of the compartment of Test 1 from this series are shown in Figure 3.1.

Thomas and Bennetts [28] conducted a test series of ethanol pool fires in a small rectangular enclosure (1.5m x 0.6m x 0.6m) to determine the influences of ventilation size and location on burning rate. They found that there were significant differences in burning rates between having the opening on the short end (long enclosure) or

the long side (wide enclosure). They observed temperature differences across multiple locations of up to 500°C, generally with greater temperatures nearer the vents, as this is where the flames resided more often. This work was continued further [29] with another experimental series of pool fires in a larger, long enclosure (8m x 2m x 0.6m), in which the opening size on the short end was varied. The results obtained were similar to both their earlier work [28] and that of Kirby et al. [27]. They conclude that a structural element near the vent would be exposed to more severe conditions than one further inside the compartment.



**Figure 3.1:** Comparison of temperature measurements over time at three different locations from the rear to the front of the compartment, illustrating non-uniform burning of the wood cribs during the tests of Kirby et al. [27].

All of the tests mentioned here show, even in relatively small scales, that fires travel and do not burn uniformly throughout the whole test enclosure.

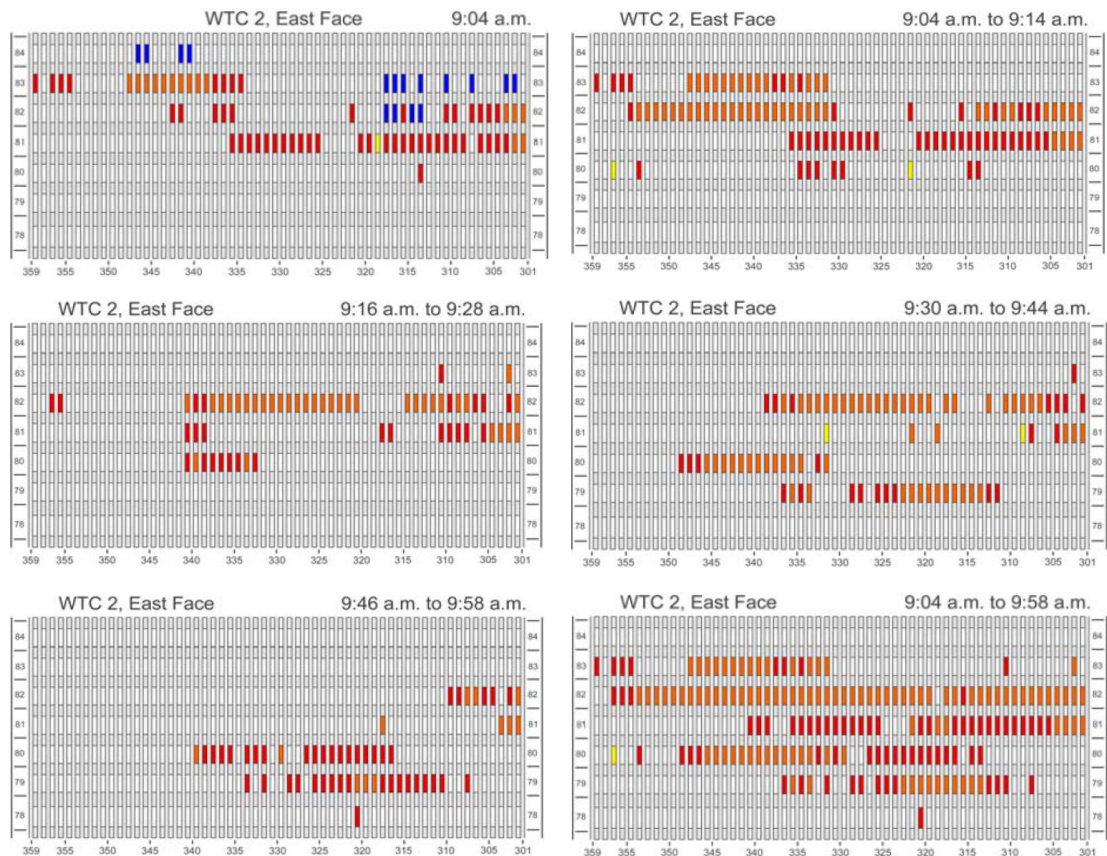
### 3.3.2 Evidence from Accidental Fires

Many large, accidental fires, such as those in the World Trade Center Towers 1, 2 [30] and 7 [31] in New York in September 2001, the Windsor Tower in Madrid, Spain in February 2005 [32] and the Faculty of Architecture building at TU Delft in the Netherlands in May 2008 [33] were all observed to travel across floor plates, and vertically between floors, rather than burn uniformly for their duration. Similar observations were made of the Interstate Bank fire in Los Angeles in 1988 [34] and the One Meridian Plaza fire in Philadelphia in 1991 [35].

The travelling nature of the fire in Tower 2 at the World Trade Center is shown in Figure 3.2, which gives the recorded observations of the fire location and burning behaviour along the East Face [30]. It can be seen that the area of flaming shifts dramatically on the floors of fire involvement, both horizontally across floors as well as vertically between floors.

Other than the fires in Towers 1 and 2 of the World Trade Center, which ended at the time of building collapse, all of the incidents listed above lasted for many hours. The Interstate Bank fire was the shortest and lasted a little under four hours, at which point it was controlled by fire fighters. The One Meridian Plaza fire was the longest, which lasted for almost 19 hours as it burnt from the 22<sup>nd</sup> to the 30<sup>th</sup> floor, where it was eventually controlled by a sprinkler system.

These fires, in addition to being visually observed as travelling, had durations that are well in excess of the time periods associated with the traditional design methods. This difference in time scales is primarily due to those methods assuming uniform burning on one floor only. Therefore the traditional methods may be underestimating exposure times as compared to the lengths of real fires, which in turn could affect the structural heating.



**Figure 3.2:** Observed fire locations over different time periods on the East Face of WTC Tower 2 [30], indicating a horizontally and vertically travelling fire. Blue = observation not possible, White = no fire, Yellow = spot fire, Red = fire visible inside, Orange = external flaming.

## 3.4 Pioneering Methods

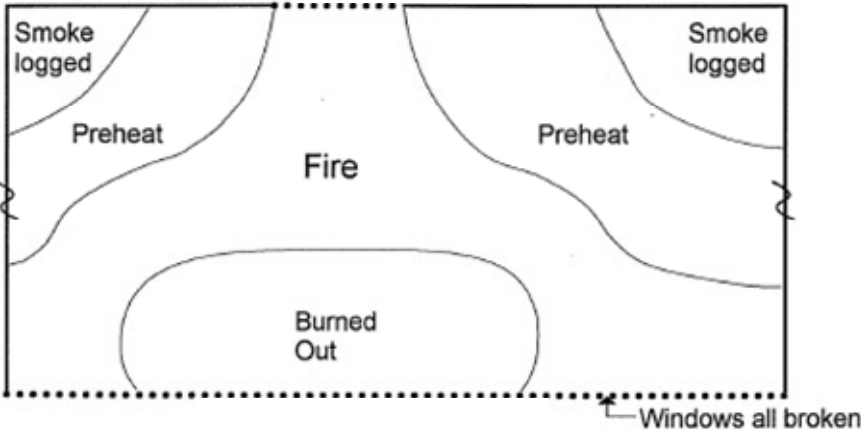
To progress past the limitations of the traditional methods, it is necessary to develop engineering techniques that account for travelling fires. This section reviews the published methods utilising travelling fires.

### 3.4.1 Large Firecell Method - HERA New Zealand

As part of a long term research programme at HERA in New Zealand aimed at understanding the behaviour of complete steel frames exposed to fire, Clifton [36] produced a first of its kind report related to design using travelling fires. The report, entitled “Fire Models for Large Firecells” and referred to as the Large Firecell

Method (LFM) in this paper, gave an approach to apply specific fire models to develop temperature-time relationships for travelling fires through a “firecell”. By Clifton’s definition, a firecell is essentially one compartment of a building. For example, an open plan office floor would be a single firecell.

Clifton applied two different fire models to generate temperature-time curves and created a set of rules on how these should be applied to “design areas” within the firecell. Each design area of the firecell at any one time could be classified as one of the following conditions: fire, preheat, smoke logged, or burned out. This is illustrated in Figure 3.3 at a fixed moment in time.



**Figure 3.3:** Representation of a spreading fire in the LFM [36]. Reproduced with permission from the author.

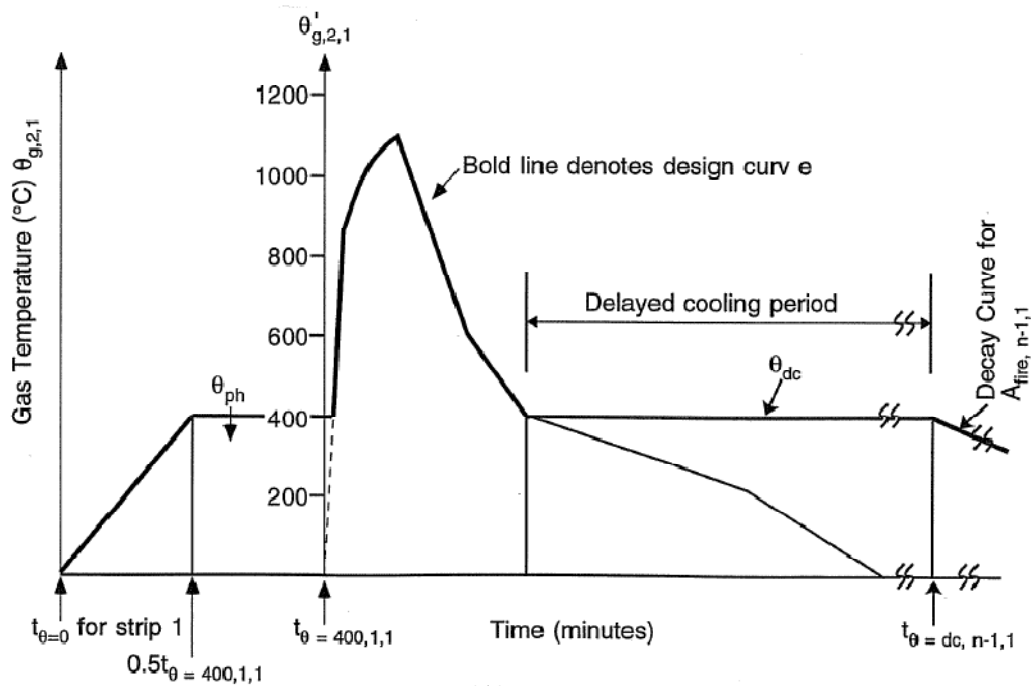
The temperature-time curves for the design areas were calculated by one of two models given, both for ventilation controlled fires. Temperatures for the preheat and delayed cooling (for after burnout) periods were taken to be between 200 and 675°C, depending on the type of construction used in the first version of the report and then subsequently modified to 400 to 800°C in the proposed changes to the document.

In the first version of the LFM, Clifton set the size of each design area based on the fuel load density. He suggested 50m<sup>2</sup> for a fuel load under 500MJ/m<sup>2</sup>, 100m<sup>2</sup> for fuel loads between 500 and 1000MJ/m<sup>2</sup>, and 150m<sup>2</sup> for fuel loads greater than 1000MJ/m<sup>2</sup>. This was modified to have the design area be 50m<sup>2</sup> for all fuel loads in the proposed changes. Windows were assumed to break once the adjacent gas temperature reached 350°C. The rate of fire spread was based on the Kirby experiments [27] highlighted in Section 3.3.1 and was specified to be 1m/s for well ventilated conditions and 0.5m/s for less ventilation, as determined by the opening factor of the case being examined.

Combining all of the various inputs in the method gives temperature-time curves at any structural element. An example is shown in Figure 3.4.

Clifton acknowledged the challenges of developing this type of methodology. He stated that no such method existed before and that there was a “paucity of experimental data available”, which required “a crude and simplistic approach to their development”. Therefore the model necessitated numerous assumptions regarding fire size, ventilation conditions, fire spread, fuel distribution and fuel type. Due to the assumptions needed, and the lack of experimental data, Clifton stated that the LFM should mostly function as a research tool and should only be used for single element checks in design.

Moss and Clifton [37] used the LFM in analysis of the large frame tests conducted at Cardington. However, they noted that this method, combined with detailed structural analyses led to results “that appeared to be realistic,” but “could not be related to any directly comparable experimental results”. Further development or applications of this method are not readily apparent in the literature.



**Figure 3.4:** Temperature-time curve of one design area in the LFM [36]. Reproduced with permission from the author.

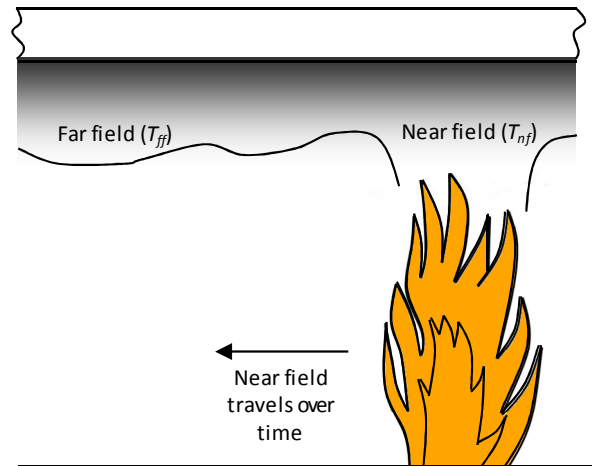
### 3.4.2 Travelling Fires Methodology – University of Edinburgh

The Travelling Fires Methodology (TFM), which has been developed by the authors independently of the LFM over the last few years, incorporates travelling fires for structural design. Full details of this method are given in Chapter 5 [38].

The TFM calculates the fire-induced thermal field such that it is physically-based, compatible with the subsequent structural analysis, and accounts for the fire dynamics relevant to the specific building being studied. In order to achieve this, a fire model is selected that provides the spatial and temporal evolution of the temperature field.

The fire-induced thermal field is divided in two regions: the near field and the far field. These regions are relative to the fire, which travels within the compartment, and therefore move with it. The near field is the burning region of the fire and where structural elements are exposed directly to flames and experience the most

intense heating. The far field is the region remote from the flames where structural elements are exposed to hot combustion gases (the smoke layer) but experience less intense heating than from the flames. The near and far fields are illustrated in Figure 3.5. The near field region is analogous to the design area of the LFM.



**Figure 3.5:** Illustration of near and far fields in the TFM [38, 42].

Early work on the TFM in 2006 by Rein et al. [39] used Computational Fluid Dynamics (CFD) to study both uniform and travelling fires in a multi-storey high rise building, with atria connecting groups of three floors into “villages”. Later work by Stern-Gottfried et al. [40] simplified and refined the method for a single floor, utilising a ceiling jet correlation to generate far field temperatures. Jonsdottir et al. [41] took this updated version and examined resultant steel temperatures. Collaboration with structural fire engineers led to work [42] exploring the response of a generic concrete frame to travelling fires, including a detailed sensitivity study. Stern-Gottfried and Rein [38] then developed the methodology further by extending the examination of the concrete frame via simplified heat transfer and identified the critical parameters for applying the method to design.

The TFM does not assume a single, fixed fire scenario but rather accounts for a whole family of possible fires, ranging from small fires travelling across the floor plate for long durations with mostly low temperatures to large fires burning for

short durations with high temperatures. Using the family of fires enables the TFM to overcome the fact that the exact size of an accidental fire cannot be determined a priori. This range of fires allows identification of the most challenging heating scenarios for the structure to be used as input to the subsequent structural analysis.

Each fire in the family burns over a specific surface area, denoted as  $A_f$ , which is a percentage of the total floor area,  $A$ , of the building, ranging from 1% to 100%. Compared to this approach, the conventional methods only consider full size fires, which are analogous to the 100% fire size in the TFM. All other burning areas represent travelling fires of different sizes which are not considered in the conventional methods.

The TFM assumes that there is a uniform fuel load across the fire path and the fire will burn at a constant heat release per unit area typical of the building load under study. From this the total heat release rate can be calculated by Eq. (3.1).

$$\dot{Q} = A_f \dot{Q}'' \quad (3.1)$$

where  $\dot{Q}$  is the total heat release of the fire (kW)

$A_f$  is the floor area of the fire (m<sup>2</sup>)

$\dot{Q}''$  is the heat release rate per unit area (MW/m<sup>2</sup>)

Furthermore, the local burning time over the fire area can be calculated by Eq. (3.2).

$$t_b = \frac{q_f}{\dot{Q}''} \quad (3.2)$$

where  $t_b$  is the burning time (s)

$q_f$  is the fuel load density (MJ/m<sup>2</sup>)

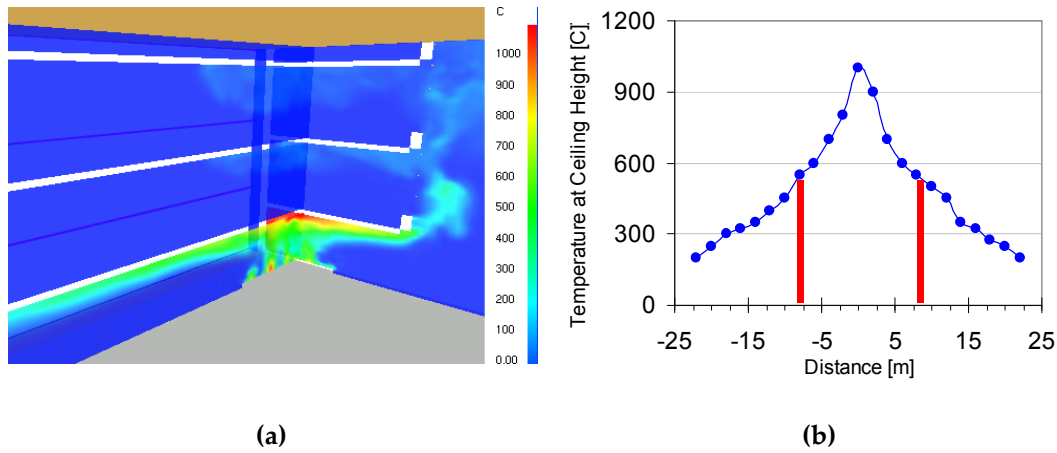
Values typically used in the application of the TFM are 570MJ/m<sup>2</sup> for the fuel load density and 500kW/m<sup>2</sup> for the heat release rate per unit area. This leads to a characteristic burning time,  $t_b$ , of 19min. This time correlates well to the free-burning fire duration of domestic furniture, which Walton and Thomas [43] note is about 20min. It is also in line with Harmathy's [44] observation that fully developed, well ventilated fires will normally last less than 30min.

Note that the burning time is independent of the burning area. Thus the 100% burning area and the 1% burning area will both consume all of the fuel over the specified area in the same time,  $t_b$ . However, a travelling fire moves from one burning area to the next so that the total burning duration across the floor plate is extended. This means that there is a longer total burning duration for fires with smaller burning areas.

As noted above, the TFM splits the temperature field into two portions: the near field (flaming region) and the far field (hot gases away from the fire). In the case of the 100% burning area, all of the structure will experience near field (flame) conditions for the total burning duration (which is equal to the burning time,  $t_b$ ). However, for the travelling fire cases, any one structural element will feel far field (smoke) conditions for the majority of the total burning duration and near field conditions for the burning time when the fire is local to the element. Therefore the TFM must quantify both the near field and far field temperatures.

The TFM assumes the near field is 1200°C to represent worst case conditions, as this is the upper bound of flame temperatures generally observed in compartment fires [5]. To calculate the far field temperatures in the TFM, an engineering tool must be selected and applied to each member of the family of fires developed. The TFM is modular in this aspect, as any calculation method that takes fire size and geometry as inputs and produces temperature as a function of distance from the fire may be used.

As stated above, the early work [39] used a CFD fire model to study the temperature field as a function of distance from the fire. As the case study for that work involved an atrium, a detailed three-dimensional model was needed. Indicative results from the case study are shown in Figure 3.6.



**Figure 3.6:** (a) Use of CFD with the TFM in a case study with an atrium; (b) Calculated far field temperatures for the same case study [39].

Later variations of the TFM [38, 40, 41, 42] focused on a simpler method to obtain far field temperatures by using a ceiling jet correlation developed by Alpert [45]. This correlation is given below in Eq. (3.3).

$$T_{max} - T_{\infty} = \frac{5.38(\dot{Q}/r)^{2/3}}{H} \quad (3.3)$$

where  $T_{max}$  is the maximum ceiling jet temperature (°C)

$T_{\infty}$  is the ambient temperature (°C)

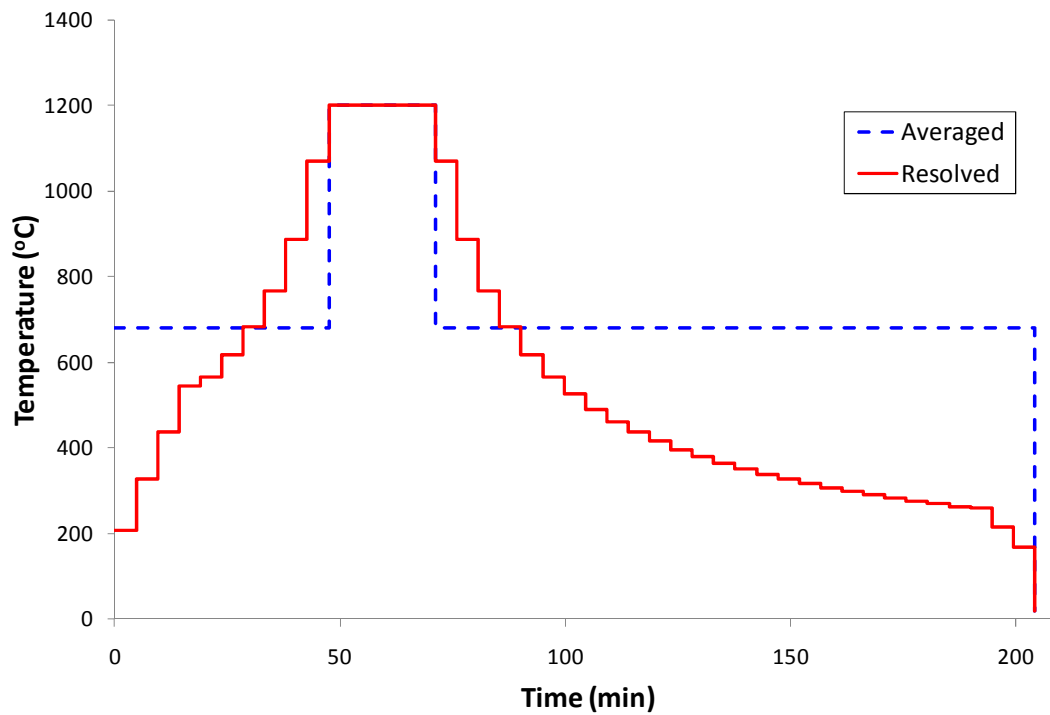
$r$  is the distance from the centre of the fire (m)

$H$  is the floor to ceiling height (m)

Note that while Alpert gives a piecewise equation for maximum ceiling jet temperatures to describe the near field ( $r/H \leq 0.18$ ) and far field ( $r/H > 0.18$ ) temperatures, only the far field equation is used as the near field temperature is assumed to be the flame temperature in the TFM. Although it was acknowledged that the ceiling jet correlation does not fully characterise the fire dynamics of the scenarios selected, it provided sufficiently accurate results to progress the development of the TFM.

In order to limit the amount of information passed to the structural analysis, the first iteration of the TFM by Rein et al. [39] only took a single far field temperature from a point away from the flaming region (see red lines showing indicative temperature in Figure 3.6b). Later versions used a fourth power average of temperature for the far field in a bias towards radiative heat transfer [40, 41, 42]. However, in more recent work by Stern-Gottfried et al. [38], this assumption has been relaxed and a spatially resolved temperature field that varies with distance from the fire is used. Instead of the average, the compartment is divided into discrete nodes, each with their own temperature. Figure 3.7 shows representative temperature-time curves developed at a single point for averaged and spatially resolved far field temperatures.

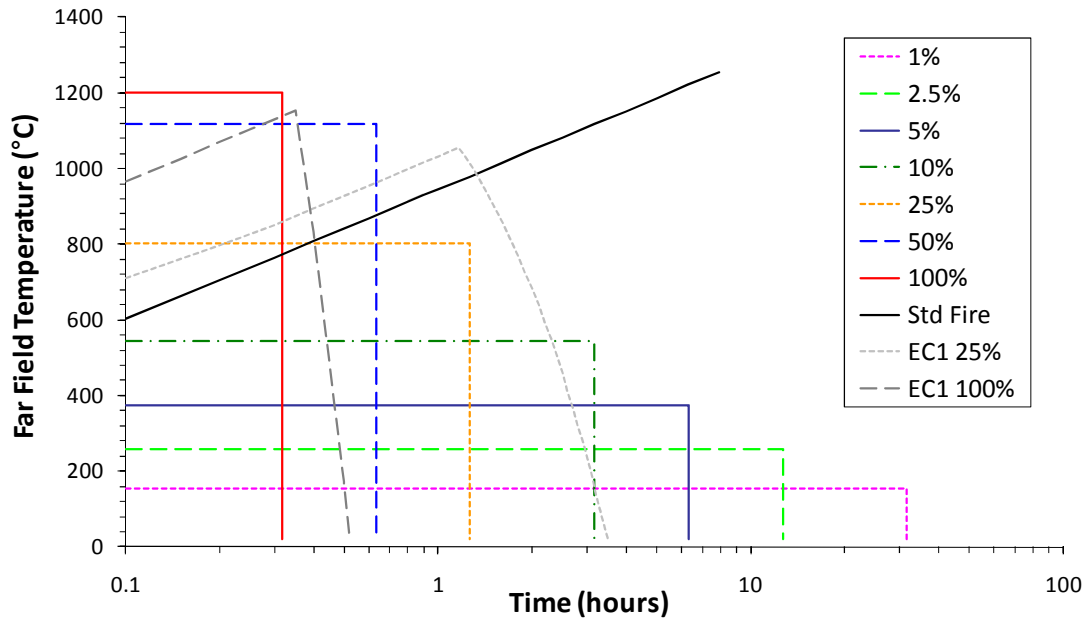
The TFM provides results of the full temperature field evolution over time, which can be used to examine particular structural elements or full frame behaviour. The fire travels at a velocity related to its size. These velocities vary from centimetres per minute for small fires to metres per minute for large fires, which is a broader range than that used by Clifton in the LFM. The range of fire sizes examined in the TFM is deemed to cover the full extent of what is physically possible in an enclosure fire.



**Figure 3.7:** Temperature-time curves at a single location in the TFM, showing averaged and resolved far field temperatures.

In the TFM when averaged far fields are used they can be plotted together and compared to examples of traditional methods. This is shown in Figure 3.8.

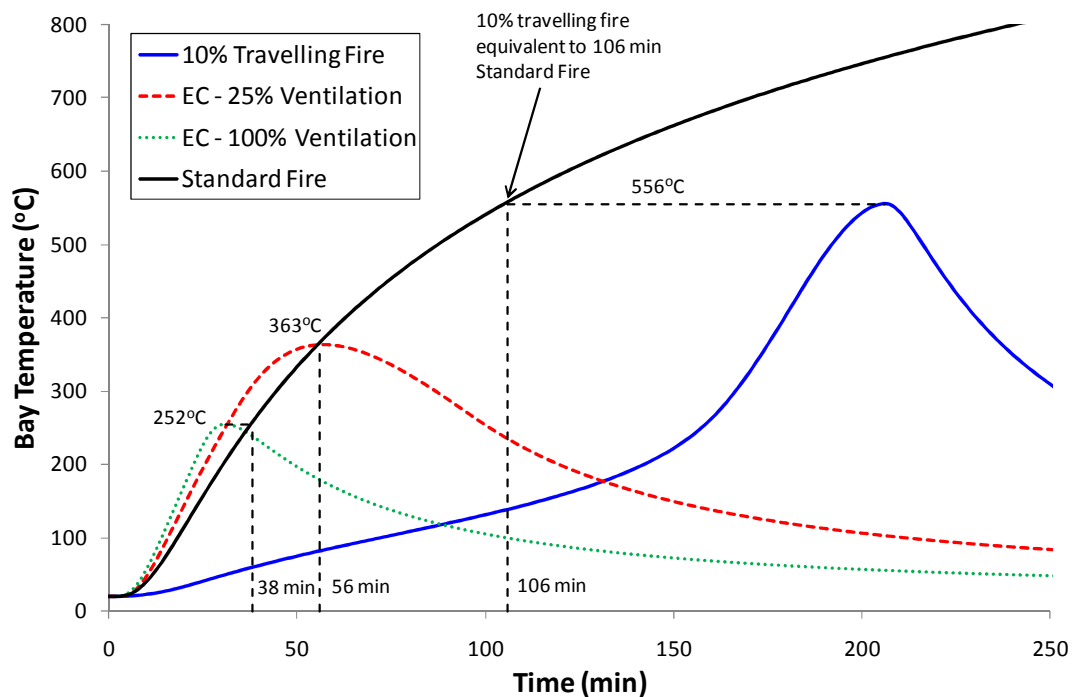
It can be seen from the results of the TFM that hotter far field temperatures last for less time than cooler ones. The standard and parametric temperature-time curves give similar temperatures to those in the far field from travelling fires of sizes between 25% and 50% but do not account for the near field conditions like the TFM does. The results of the standard fire curve cannot be explained after one hour of burning in terms of the possible fire dynamics in.



**Figure 3.8:** Averaged far field temperatures for a family of fires in the TFM and traditional methods as applied to a generic concrete frame 42m x 28m x 3.6m per floor [42].

While a simple plot cannot be shown for a resolved far field, the results can nevertheless be used for heat transfer and structural analysis. Their results are better compared to the traditional methods via the resulting structural performance, as shown in Figure 3.9, which compares rebar heating from exposure to a 10% travelling fire, the standard fire and two different Eurocode parametric temperature-time curves.

The temperature fields generated from the TFM have been applied to both concrete and steel structures by means of heat transfer analyses [38, 41]. These analyses have looked at the temperature of either steel rebar within concrete or steel beams as a loose surrogate for structural performance. The results showed that travelling fires have a significant impact on the performance the structures examined and that conventional design approaches cannot automatically be assumed to be conservative. Medium sized fires between 10% and 25% of the floor area were found to be the most onerous for the structure. This is due to a balance of burning duration and far field temperatures.



**Figure 3.9:** Comparison of rebar temperatures calculated using a 10% fire size from the TFM, the standard fire, and two Eurocode parametric temperature-time curves in a similar generic concrete frame as shown in Figure 3.8 [38].

Detailed sensitivity analyses of the input parameters of the TFM have also been conducted [38, 42], showing that the structural design and fuel load have a larger impact on structural behaviour than any numerical or physical parameter used in the methodology.

### 3.5 Structural Response

In his plenary lecture at the IAFSS Symposium in 2008, Buchanan [20] stated:

*The two disciplines of combustion science and structural engineering are miles apart, so two groups of experts will always be needed. For this reason it would be very foolish to rush towards coupling of fire models with structural models. Any such coupling would lead to a “black box” mentality with a major decrease in our ability to make accurate predictions of structural fire behaviour.*

*Fire engineers and structural engineers need to talk to each other much more than they do now, and each group needs to learn as much as possible of the other discipline. These two topics are too big and too different for us to educate combined specialists in both disciplines.*

The comments made by Buchanan, and reinforced by Law et al. [21] highlight the need for close collaboration between the two disciplines. The TFM has been developed with such collaboration in mind [38, 39, 40, 41, 42].

This section reviews research involving detailed structural analysis of travelling fires.

### **3.5.1 Steel Frame**

The first detailed analysis of structural behaviour in response to travelling fires was conducted by Bailey et al. [46]. This work, which was notably conducted prior to publication of Clifton's LFM and twelve years before Buchanan's call for multi-disciplinary collaboration, was pioneering in its recognition for the need to consider the structural impact of a more realistic fire environment than the conventional methods by examining travelling fires.

Bailey et al. extended use of a Finite Element Model (FEM) from previous research involving uniform fires to study a two-dimensional frame exposed to a spreading fire. The work began with a focus on the effect of the cooling phase of a fire on the structure. The authors then note that incorporating the cooling phase allows consideration of "fires which spread progressively from an ignition point in a single compartment (or a zone within an open-plan area) to adjacent areas of the building". They go on to state:

*The effect of a spreading fire is that both cooling and heating are taking place simultaneously in different zones. This is arguably a more typical condition than the assumption that the temperature changes uniformly throughout the fire-affected zone, and in view of the effects of restraint observed during cooling is one which requires investigation.*

The study compares the response of a two-dimensional bare steel frame exposed to a spreading fire with that of a uniform fire, over both three and five structural bays. The uniform fire was defined by a temperature-time curve representing a “natural” fire. The travelling fire was represented by the same natural fire curve, but offset in time for the bays of secondary fire involvement. Once the temperature-time curve in the first bay reached its peak, the fire was assumed to begin in the adjacent bays. Similarly, the bays of tertiary fire involvement were assumed to ignite when the temperature-time curve reached its peak in the secondary bays. The temperature-time curves used are shown in Figure 3.10.

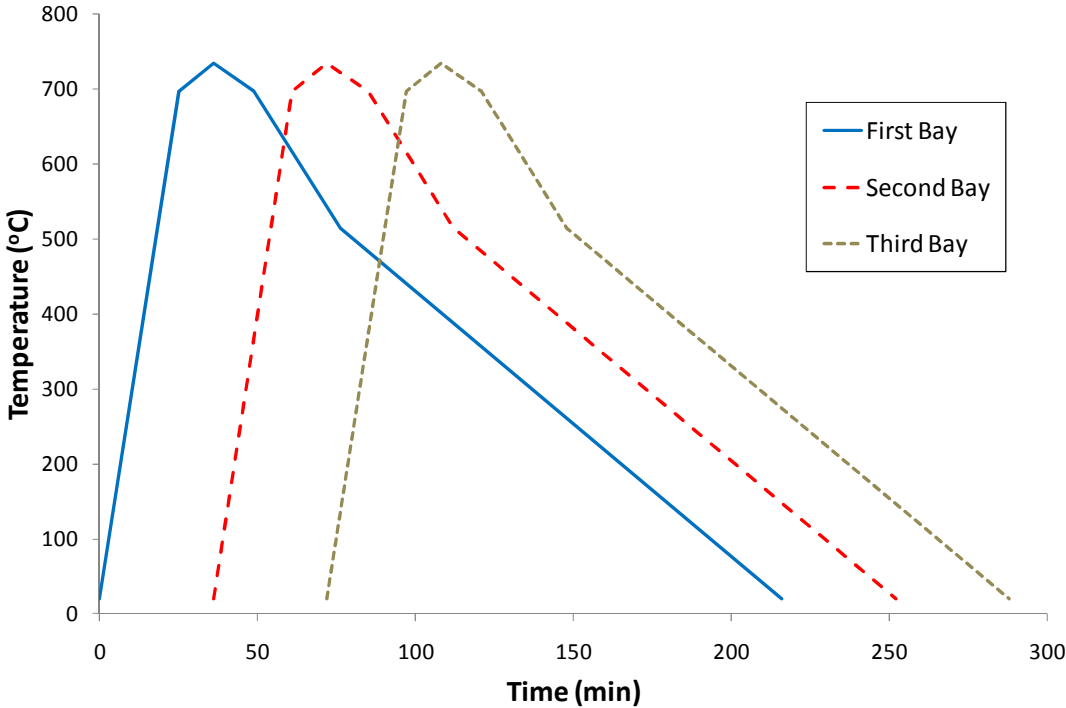


Figure 3.10: Temperature-time curves used by Bailey et al. (adapted from [46]).

While this method replicates the movement of the near field associated with travelling fires, the use of a temperature-time curve reproduced with a delay does not capture the far field of a travelling fire, as can be seen by the relevant details discussed in Sections 3.3 and 3.4 of this paper. The temperature-time curve used assumes a ventilation controlled fire. However, a fire burning in only one bay of a structure nine bays wide is unlikely to be ventilation limited, especially in the early

durations of the fire, as the air available from the rest of the structure may provide sufficient oxygen to keep it well ventilated. Additionally, local exposure to flame temperatures (near field conditions), and not just compartment average temperatures associated with the calculation methods of ventilation limited fires, are likely.

Furthermore, this method does not account for elevated smoke temperatures of the far field away from the fire. The temperature in a bay adjacent to the first one exposed remains ambient until its curve begins at 36min. Given that the bays are 8m in dimension, it is much more likely that temperatures in the adjacent bay would be well above ambient at this time. This behaviour could be explained if each bay were a fully enclosed, fire rated compartment that fails 36min into the fire, however this is not the scenario described by the authors.

Bailey et al. went on to examine the vertical displacements and axial forces in the beams of the structure. They found that higher beam displacements occur for the spreading fire cases than the uniform ones. The authors noted that these conclusions cannot be readily generalised and further study is required.

### **3.5.2 Concrete Frame**

More recently, and after the publication of the LFM and TFM, two simultaneous papers on the impact of travelling fires on concrete frames have been published.

Ellobody and Bailey [47] conducted a study of the impact of horizontally travelling fires on a post-tensioned concrete floor. While this study utilises sophisticated structural analysis, including a three-dimensional FEM, the fire definition is very similar to that used by Bailey et al. [46]. Specifically, a base temperature-time curve is applied to the first bay of heating and is shifted in time to provide the heating of bays that become subsequently involved in the fire. In this study, the base

temperature-time curve was taken from Eurocode 1. Two time delays were examined; one of 64min and another of 30min.

The structural response was viewed in terms of tendon temperatures, deflections and axial displacements. These parameters were examined at several critical locations over time as well as in terms of their final residual values. Ellobody and Bailey noticed that the “change in heating/cooling scenarios between zones resulted in cyclic deflection patterns at some locations”. They also found that the time delay used for shifting the temperature-time curve had an impact on the structural response and the worst case could result from a uniform heating case or a non-uniform travelling fire. The authors recommended that engineers consider a range of travelling fires for use in structural design to ensure the most onerous scenario is found.

Given a very similar method for thermal definition was used in this paper as Bailey et al. the same critiques of the that method apply; namely the inherent assumption of a ventilation limited fire in an open space and the lack of consideration of the far field (hot smoke away from the fire). In fact, the cyclic deflection patterns observed by Ellobody and Bailey could be affected by the presence of elevated far field temperatures. This is because in their analysis some elements would be exposed to ambient gas phase conditions while others to peak temperatures, when in reality the ambient exposure would more likely have been that of smoke temperatures on the order of several hundred degrees Celsius.

The work of Law et al. [42], a collaborative research project between the fire engineers Stern-Gottfried and Rein and structural engineers Law and Gillie, applied the TFM to a generic concrete frame 42m long x 28m wide. The temperature field was generated as explained in Section 3.4.2 and then applied to a FEM of the concrete frame.

The structural modelling results were examined in terms of rebar temperature, sagging tensile strain, hogging tensile strain, and deflections. The results for rebar temperature showed that fire sizes between 10% and 25% of the floor area produced the most onerous results for the structure. All of the more detailed structural metrics showed that the 25% fire size was most challenging for the structure. In all four metrics the travelling fires proved to be a worse case for the structure than the Eurocode parametric temperature-time curves. A detailed sensitivity study showed that variations in the far field definition and differing fire shapes and paths of travel had little impact on the results.

In his PhD thesis [48], Law further examined the structural behaviour resulting from travelling fires, using sectional and utilisation analyses. Generally he obtained similar results, but did notice that 5% to 10% fire sizes gave the worst case results for the structure when using a utilisation analysis of all columns. The strength of the methods applied by Law is that data from numerous fires can be viewed cumulatively to get a better understanding of the behaviour of each column. This is well suited to analyse results from the TFM which produces a family of fires.

### **3.5.3 Vertically Travelling Fires**

Noting that large, accidental fires tend to involve multiple floors, Röben et al. [49] examined the impact of vertically travelling fires on a multi-storey structure. The building they examined was used in previous work by the authors to understand the effect of the cooling phase on structural performance and had a concrete core and a steel-concrete composite floor system.

The study assumed three floors were on fire. Although Röben et al. noted that “horizontally travelling fires would give a more realistic representation of the fire spread through a compartment”, the authors assumed horizontally uniform fires for their study, stating that it is “a common assumption in structural fire design”. The heating pattern used was similar to the horizontal studies by Bailey et al. and

Ellobody and Bailey, i.e. the same temperature-time curve was applied to each floor but with a time delay between floors. The heating curve used was a generalised exponential curve given by Flint [50]. Röben et al. noted that this curve was selected because analysis by Flint “showed it to be a better approximation for large compartments than the more commonly used ‘natural fire’ curves given, for example, in the Eurocodes”, however no theoretical background or physical justification of the method is given. The cooling phase was assumed to be linear between the maximum and ambient temperatures over a period of 1400s.

Three fire scenarios were used; uniform heating on all three floors, a time delay of 500s between each floor, and a time delay of 1500s between each floor. The authors noted that many factors influence the vertical spread rate. The values used in the study were to roughly capture the range of eyewitness accounts of vertical flame spread of between 6 and 30min in the Windsor Tower fire.

The results, primarily examined in terms of horizontal displacements of columns and total axial forces of floors, showed that the vertically travelling fire with a short time delay induced a similar structural response to that of the uniform heating case. However, the primary difference observed was a “cyclic pattern induced in columns” for the travelling fire. This pattern was also observed for the long delay travelling fire, but with longer time intervals. The authors note that this cyclic deflection pattern has not been examined before and has a significant impact on the structure and, therefore, should be considered in design.

The observation of a cyclic pattern is similar to that of Ellobody and Bailey. However, this finding perhaps has more relevance for vertically travelling fires because compartment floors will likely limit the spread of hot gases that may preheat the upper floors prior to full fire involvement. Notwithstanding this argument, the nature of the column deflections may be affected by consideration of

horizontally travelling fires as well. However, no studies to date have examined this.

## 3.6 Practical Applications

The work highlighted so far in this paper have pioneered or developed the concept of travelling fires and the subsequent structural analyses, which is a research topic that is beginning to grow within the fire engineering community. This section reviews recent applications of travelling fires to real building projects found in the literature.

The first version of the TFM has been applied to case studies in the two real buildings. These case studies are described below.

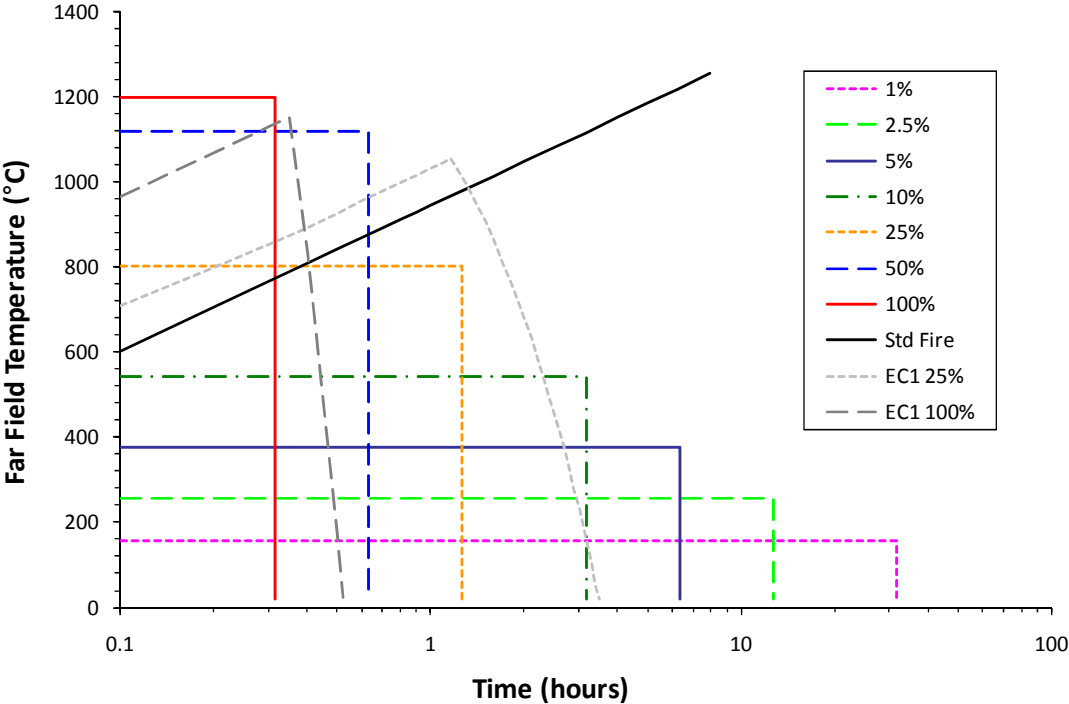
In 2009 Stern-Gottfried et al. [40] applied the TFM to the Mumbai C70 building project, shown in Figure 3.11a, during its early design stages. The building had 13 storeys and was approximately 60m tall. It had a unique structure, including an external diagrid megaframe consisting of hollow structural steel members designed to carry wind loads and a proportion of the gravity load, an internal reinforced concrete core system designed to carry gravity load, and a hat truss at the top of the building. The exact shape of each floor varied, with most being over 2000m<sup>2</sup> in area. Much of the external façade was glazed, thus placing the building outside the range of applicability of the traditional design methods.

The 9<sup>th</sup> floor of the Mumbai C70 building was selected for structural fire analysis, as this floor had the longest beam spans as well as slender diagrid members compared to those found lower in the building. Therefore this floor was deemed to be where a severe fire would be most challenging to the structure and therefore the location studied. The first version of the TFM was used to generate temperature-time curves, utilising an averaged far field temperature, specific for the 9<sup>th</sup> floor. The far field

temperatures plotted against total burning duration for a range of fire sizes for this study are given in Figure 3.11b, with comparison to the temperature-time curves from the standard fire and two Eurocode parametric cases.



(a)



(b)

**Figure 3.11:** (a) Architectural image of Mumbai C70 by James Law Cybertecture; (b) Far field temperatures vs. total burning durations for different fire sizes, with the standard temperature-time curve and two parametric Eurocode curves for reference [40].

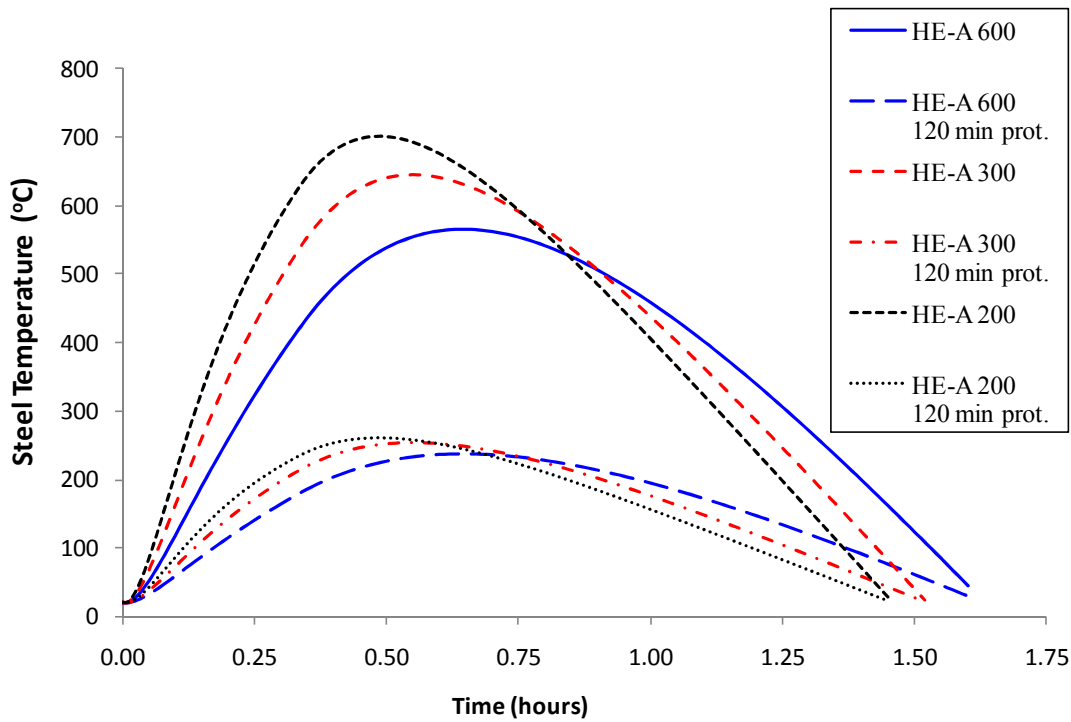
In 2010 Jonsdottir et al. [41] calculated the resultant steel temperatures from a thermal field generated by the TFM for the Informatics Forum at The University of Edinburgh, shown in Figure 3.12a. The Informatics Forum was completed and occupied in 2008. The building was chosen for the study because of its unique architectural features. It is a seven storey office building for lecturers, staff and researchers, with a central glass atrium, and a floor area of approximately 1700m<sup>2</sup>.

The case study examined structural heating of three different steel beams based on the temperature–time curves of the family of fires generated from the TFM. Example results of this analysis are given in Figure 3.12b. This paper was the first to report the resultant structural temperatures caused by travelling fires. These results were then compared to calculations of the steel beam temperatures utilising the traditional methods. It was found that the TFM method resulted in 10% to 55% higher peak steel beam temperatures than the traditional methods for medium sized fires of 10% to 25% area.

Apart from the LFM and TFM, other researchers have applied similar ideas related to travelling fires. Sandström et al. [51] developed a pre-processing tool to rapidly apply travelling fires as input to a CFD model (FDS v5.5). They examined a 20m x 40m x 10m, open plan building with natural ventilation in the roof and on all four sides. They developed a design fire based on Eurocode 1 [14] guidance, which ramps up at a “medium”  $t^2$  rate, to a peak of 95MW, then linearly decays. This design fire was then applied using different heat release rates per unit area to define different fire scenarios, some of which were stationary (covering 12.5% and 100% of the floor area) and others that were travelling (using fire sizes of 0.125%, 0.5% and 2% of the floor area). It is not clear how the travelling fires were implemented and no descriptions of the travelling nature, velocity, or burnout characteristics of the fires were given.



(a)



(b)

**Figure 3.12:** (a) Informatics Forum at The University of Edinburgh; (b) Resultant steel temperatures vs. time for three different beam types that were both unprotected and fire rated to 120min [41].

The results were reported as average smoke layer temperature-time curves, including comparison to simulations using the two-zone model OZone [18], thus averaging the near and far fields together into a single temperature. Because of this,

the analysis more closely resembles the traditional design methods that assume homogeneous temperature conditions than the travelling fire methods already cited.

In 2010, Shestopal et al. [52] provided a review of two case studies where travelling fires were implemented with CFD (FDS v5). The authors stated that the worst case scenarios resulted from a spreading fire, which they used to justify the reduction of fire resistance levels against those nominally required by the local building code. The case studies presented were for a supermarket and an office building. In the supermarket case study, the travelling nature of the fire was modelled via flame spread predictions within the CFD model. In the office case study it was represented by user specified sequential ignition along the floor (with time delays ranging from 45 to 90s) set to match experimental heat release rate data.

From the limited information presented, it appears the analyses for the supermarket may have extended beyond the capabilities of current CFD models, by predicting flame spread which is a challenging physical process to accurately model [25]. However, the spirit of this work, which examined spatially varying far field temperatures, is in line with the ethos of the travelling fires methods.

### **3.7 Conclusions**

The concept of travelling fires suggests a paradigm shift in structural fire engineering. The dynamics of travelling fires are central to better understanding the true structural performance of buildings exposed to real fires, and therefore the potential to enable architectural innovation and structural optimisation.

However, given the importance of travelling fires, there has been only a limited amount of research to date on the topic and more is needed. The earliest research by Clifton and Bailey et al. in 1996 established the need for robust methods to account for travelling fires. The development of the TFM in 2006 offers such an engineering

technique. However, refinements to the TFM for horizontally travelling fires are needed to make it more robust. Additionally, fundamental work is needed to examine vertically travelling fires. As opposed to horizontally travelling fires, no framework exists to explore the dynamics of vertically travelling fires, which is currently hindering their application in structural analysis, despite the numerous incidents of vertically travelling accidental fires.

Of particular importance in the development and application of travelling fire methodologies is the close collaboration between fire engineers to define the thermal environment and structural engineers to determine the subsequent structural behaviour.

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# 4

## The Influence of Travelling Fires on a Concrete Frame

### 4.1 Introduction

Since the early 20<sup>th</sup> century, the Standard Fire test and associated temperature-time curve [1, 2] have been used world-wide to give fire ratings to structural assemblies and to design complete structures [3]. The Standard Fire temperature-time curve was created in an attempt to regulate testing between different laboratories thereby ensuring a uniform standard of safety. However, almost as soon as it was conceived, a number of problems were identified with it. Notably, no account is taken of differences in fuel load, fire compartment size or ventilation conditions, all of which profoundly affect the behaviour of a compartment fire. To address some of these shortcomings, other temperature-time curves have been proposed. Perhaps the most widely known in structural design are the “parametric” fires curves. Initially developed by Pettersson [4], these curves have been modified and are incorporated

into the Eurocode structural design standards [5]. They allow design fires to be calculated that, unlike the Standard Fire curve, depend on the fuel load, thermal inertia of linings, and ventilation conditions of a fire compartment. Parametric fires therefore predict more realistic temperature-time curves than the Standard Fire and can be roughly replicated by burning wooden cribs in a small fire compartment. Despite these benefits, parametric fires remain very crude representations of fires in any but the simplest of compartments, as will be described in Section 4.2. Moreover, they are unsuitable for application in the large, open-plan spaces that are a common feature of many modern buildings. Thus, there remain significant shortcomings amongst the traditional design methods for specifying the thermal inputs for use in structural fire design, particularly for large compartments.

By contrast, over the past 20 – 30 years, knowledge and understanding of how structures respond to elevated temperatures has developed rapidly and to a point where it is now possible to include a large variety of phenomena in structural models and to predict the response of structures subject to known temperature loading with good accuracy [6, 7, 8]. Coupled with the recently developed performance-based design codes [9, 10], these capabilities have given engineers the freedom to design structures to resist high thermal loadings in innovative, efficient ways.

Thus, while the ability to predict subsequent structural behaviour has reached an advanced level, the thermal inputs used in structural fire design remain simplistic, unchanged, and not representative of actual fire dynamics in large compartments. The various limitations inherent in the traditional design methods mean that it is difficult to justify continuing to develop and use complex structural models when one of the dominating input parameters – thermal loading – remains very crudely defined. Without some development of the method for specifying design fires, it will be impossible to obtain the “consistent level of crudeness” which has been identified as a need within the discipline [11]. In an attempt to rectify the mismatch

in the levels of sophistication that are currently used for design fires and the subsequent structural analysis, this paper adopts a new approach [12, 13, 14]. First, a method of defining design fires that are sufficiently flexible to be applied to any fire compartment is presented and discussed. The method has the key benefits of not assuming a uniform temperature within a large fire compartment and allowing for fires that travel within a compartment. Second, the paper considers the implications of using these new design fires by applying them to the analysis of a concrete framed structure subject to full-floor fires and comparing the predictions of various measures of “structural distress” with those obtained when traditional fire curves are used.

## **4.2 Limitations of Current Design Fires**

Parametric and Standard Fires were validated by test data from small fire compartments that were almost cubic. This test geometry allows for good mixing of the fire gases and so is more likely to produce uniform temperature field within a compartment. These conditions do not exist in real fires [15] and consequently limitations must be placed on the form of compartment in which the traditional fire curves may be used. For example, Eurocode 1 states that the parametric curves are only valid for compartments with floor areas up to 500m<sup>2</sup> and heights up to 4m, the enclosure must also have no openings through the ceiling, and the compartment linings are restricted to having a thermal inertia between 1000 and 2200J/m<sup>2</sup>s<sup>1/2</sup>K, which means that highly conductive linings such as glass façades and highly insulating materials cannot be taken into account. As a result, common features in modern construction like large enclosures, high ceilings, atria, large open spaces, multiple floors connected by voids, and glass façades are excluded from the range of applicability of the current methodologies.

A recent survey of buildings in Edinburgh, UK [16], underlines the implications of these limitations on the applicability of design fires, particularly for modern

structures. For buildings built over a long period of time starting in the early 20<sup>th</sup> century, 66% of their total volume falls within the limitations. However, in a newly constructed, modern building that has open spaces and glass façades, only 8% of the total volume is within the limitations. This suggests that modern building design is increasingly producing buildings that contain compartments to which parametric fires should not be applied.

Additionally, an assumption that has remained unquestioned with each temperature-time curve no matter how they have been applied has been that of uniform burning and uniform compartment temperature. It is assumed that every part of a structural element or compartment is uniformly subject to the same temperature – as defined by the temperature-time curve adopted. Although it may be possible to replicate these conditions in a furnace, a recent experimental review of post-flashover tests [15] has clearly demonstrated that temperature conditions are non-uniform in most compartments. Moreover, major fires at the Windsor Tower [17], World Trade Center [18, 19] and TU Delft [20] have shown that fires tend to travel around large compartments rather than burn uniformly. Tests have also shown there is a high degree of temperature variation even within small compartments [21, 22, 23].

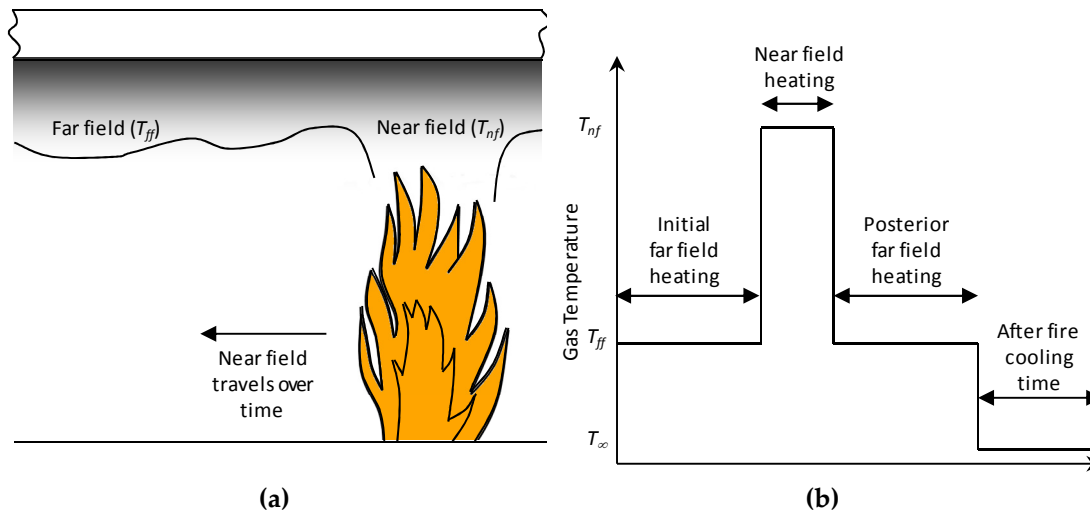
Therefore, at present, designers are forced to either use parametric fires in compartments for which they are not strictly applicable, apply unrealistic Standard Fires to large compartments, or to resort to CFD models of fires in large compartments that are labour intensive to produce. There is a clear need, then, to address the limitations of the currently available design fires if modern performance-based design is not to be restricted.

## 4.3 Travelling Fires

In light of the various limitations outlined above, a new method for estimating compartment fire temperatures based on the fundamental fire dynamics of the compartment has been proposed [12, 23, 24]. This new method will be used throughout this paper. It uses two temperature fields to represent the gas temperature in a compartment: a high temperature in the flaming region of the fire (the near field); and a cooler temperature for the rest of the compartment (the far field). This approach provides a flexible technique whereby a large range of possible fires in any compartment can be represented. For example, a fire which engulfs an entire large floor plate simultaneously, as in traditional design methods, can be represented, as well as a small fire that travels slowly from one end of a compartment to the other. The full range can then be explored by parametrically varying the size of the fire. This avoids the weakness of previous methods assuming that arbitrary events lead to particular fire conditions, such as assuming that glazing failure leads to one single temperature-time definition for an entire region. Instead, consideration of a wide range of possible fire sizes covers for the inherent variable nature of real fire events (outcome of the combination of a particular ignition location, fuel distribution and ventilation conditions). Thus, a family of fires is created ranging from a small travelling fire that burns for a long duration as it travels, to a fire uniformly burning over the full extent of the floor for a shorter time period. Therefore, the method addresses the two key shortcomings of existing methods – restrictions on the nature of applicable fire compartments and the assumption of uniform gas temperatures within a compartment – while still being sufficiently concise for use in structural design.

### 4.3.1 Temperature Definition

The new design approach represents the horizontal temperature distribution of a fire compartment by means of near field and far field regions, as illustrated in Figure 4.1.



**Figure 4.1:** (a) Illustration of a travelling fire; (b) Near field and far field exposure durations at an arbitrary point within the fire compartment.

The near field is the flaming region of the fire. Peak values in small fires have been measured in the range of 800 to 1000°C [25] but temperatures of 1200°C have been measured for larger enclosure fires [5]. This maximum value of 1200°C is chosen for the near field to represent the worst case conditions. The far field represents the temperature of the hot gases away from the flaming region. Far field temperatures can be calculated using any engineering tool that gives temperature distributions away from the fire, including hand calculations or computer modelling. For this study, the simple ceiling jet correlation developed by Alpert has been used [26] and is given in Eq. (4.1).

$$T_{max} - T_{\infty} = \frac{5.38(\dot{Q}/r)^{2/3}}{H} \quad (4.1)$$

where  $T_{max}$  is the maximum ceiling jet temperature(°C)

$T_{\infty}$  is the ambient temperature (°C)

$\dot{Q}$  is the heat release rate (kW)

$r$  is the distance from the centre of the fire (m)

$H$  is the floor to ceiling height (m)

This correlation was developed for a stationary fire during steady state conditions but is valid for travelling fires because the flame spread rate ( $\sim 0.01\text{m/s}$  [5]) is much lower than the velocity of the smoke ( $\sim 1\text{m/s}$ ). Thus, the far-field temperature distribution in Eq. (4.1) moves with the fire in a quasi steady state form.

As the fire consumes the available fuel and ignites new material in its path, it moves around the floor-plate. Consequently, the gas temperature adjacent to any given structural element is constantly changing as the fire travels both near that element and remote from it. To make the amount of information passed to a structural analysis manageable, the monotonically decreasing far field temperature distribution from Alpert's correlation is reduced to a single characteristic value,  $T_{ff}$ . To do this, the far field temperature is taken as the fourth-power average of  $T_{max}$  (to favour high temperatures in a bias towards radiation heat transfer and onerous structural conditions) over the distance between the end of the near field,  $r_{nf}$ , and the end of the far field,  $r_{ff}$ . This average is calculated by Eq. (4.2).

$$T_{ff} = \frac{\left[ \int_{r_{nf}}^{r_{ff}} (T_{max})^4 dr \right]^{1/4}}{(r_{ff} - r_{nf})^{1/4}} \quad (4.2)$$

Figure 4.1 illustrates the concept of a near field and a far field for a travelling fire. Any given location is exposed to the far field temperature for a period before the arrival of the flaming, near field region. After all the fuel at the location has been consumed and the near field moves away, it is then subjected to the far field temperature again until all the fuel in the entire compartment has been consumed, at which point the temperature returns to ambient and the structure cools.

### 4.3.2 Fire Size

The flexibility of the method stems from parametrically varying the size, shape, and path of the fire. It is assumed that, once alight, any area of the floor plate will

continue to burn at the same rate until all the fuel is consumed. The local burning time for any fire size can, therefore, be simply calculated from the fuel load and the heat release rate. Once the local fuel is burnt out, the fire will move to a new area. After the fire has travelled around the whole compartment, the cooling of the structure takes place. The fire size is varied, in this study from 1% to 100% of the compartment floor area. Assumptions and details of how to calculate the resultant heating from this method can be found in other papers by the authors [13, 14, 23].

## 4.4 Structural Failure Criteria

The methodology presented above can be used to study the impact of different travelling fires on the response of a structure. However, without a means to compare the structural response, it is impossible to draw any conclusions. There are many different methods of assessment available for fire-affected structures of varying degrees of complexity.

The simplest and most widely used measure of structural distress is maximum deflection. Typically, failure is defined as a ratio of deflection, e.g. span/20 [2]. The allowable deflection does not represent a value at which an assembly catastrophically loses stability; rather, it is the maximum deflection allowable in a furnace test in order to protect expensive experimental equipment. In spite of this, deflection is a simple and useful measure which can be used to give some indication of structural distress. It is possible to use the relative deflections caused by different fires as a means for comparison.

Another simple measure of performance for concrete structures is the maximum temperature of the tension reinforcement. Failure in steel members is often said to have occurred when the axial capacity of a section is half its ambient capacity. For reinforcing steel in concrete, this critical temperature is typically taken as 593°C [27]. Again, although this is a fairly arbitrary measure of “failure”, the temperature of the

rebar offers a simple and easily comparable metric that can be used to examine the relative impact of different fires on a structure.

The ultimate strain in the tension reinforcement is also often used as a definition of failure; beyond this strain, the rebar can be assumed to have failed. This measure is better suited to the numerical analysis of structures than to fire tests because of the difficulties associated with instrumentation of rebar. However, the strain in the tension steel provides another measure which can be used to compare the relative impact of the different fires. The ultimate strain for steel at any temperature is typically taken as 0.2 [10, 28].

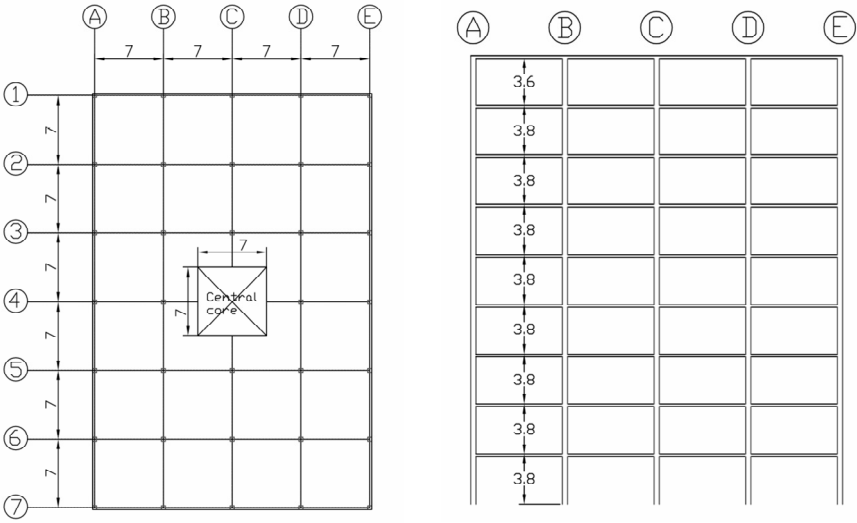
## **4.5 Structural Modelling**

The remainder of this paper is a case study that demonstrates how the above travelling fire methodology and failure measures can be applied in a structural analysis. Initially, a number of base case scenarios are considered and the differences between the predicted structural responses compared; a parametric study is then conducted to assess the validity and effect of the various assumptions made by the new approach. Finally, the impact of the shape and path of the fire is considered.

### **4.5.1 Structural Arrangement**

The case study analyses the impact of travelling fires on a generic concrete office building. The structure is a nine storey, flat-slab concrete frame, designed in accordance with the Eurocodes [29, 30, 31]. A plan and elevation of the structure are shown in Figure 4.2. The floor slabs are 200mm thick; the interior columns 400mm x 400mm; and the exterior columns 300mm x 300mm. The design strength of the concrete in the columns is 48MPa, and that in the slabs 40MPa. In this paper fires burning on the fourth floor are considered. This allows the structural effects of a

mid-level fire to be analysed without the need to explicitly consider effects of the foundations or the building's top storey.



**Figure 4.2:** Plan and elevation of concrete structure, dimensions in metres.

Two finite-element models of the central floors of the structure were created using the commercially available Abaqus [32] software. One was a heat-transfer model developed to determine structural temperatures, the other a stress analysis model produced to predict the mechanical response of the structure. The models were sequentially coupled so the heat-transfer analysis results affected the mechanical response. Both models extended from the base of the columns at the third-storey level, to the top of the columns at the fifth-storey level. The floor slabs were modelled using shell elements, the columns using three-dimensional solid elements and the rebar using truss elements.

In the heat-transfer model, thermal properties were specified in accordance with those of a 1.5% moisture content concrete, as defined in Eurocode 2 [9]. Heating of the structure was analysed by applying relevant radiative and convective boundary conditions to the surface of the structure. For the purposes of this study, an

emissivity of 0.7 and a convective coefficient of 25W/m<sup>2</sup>K were assumed in accordance with Eurocode guidance [9].

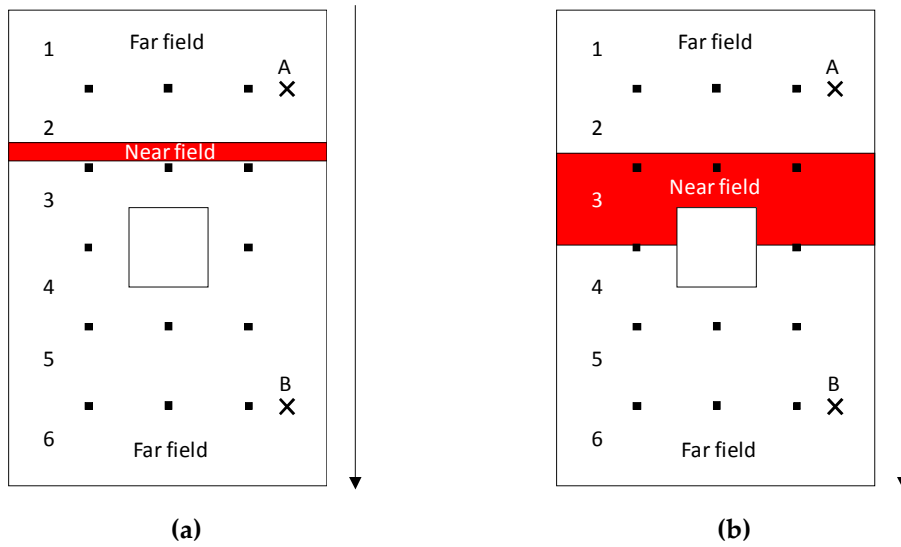
For the mechanical analysis, all of the material properties used in the model were temperature dependent and in accordance with Eurocode 2 and the yield criterion used for the concrete was the “damaged plasticity” model, based on the work of Lubliner [33]. A series of mesh sensitivity studies were conducted to find the optimum mesh density. The final mesh density used was 8 × 8 × 18 elements per floor per column, and an average element size of 0.4735m in the slab.

The base of each column was assumed to be fixed in translation and rotation, and the top of each column was fixed in all directions other than vertical. As the higher storeys of the structure were not modelled, the equivalent loads that would have been transferred into the column heads were calculated using a full-frame elastic model and applied to the remaining structure during the loading phase of the analysis. The central core of the building was not modelled explicitly but was assumed to provide rigid support to the adjoining structure.

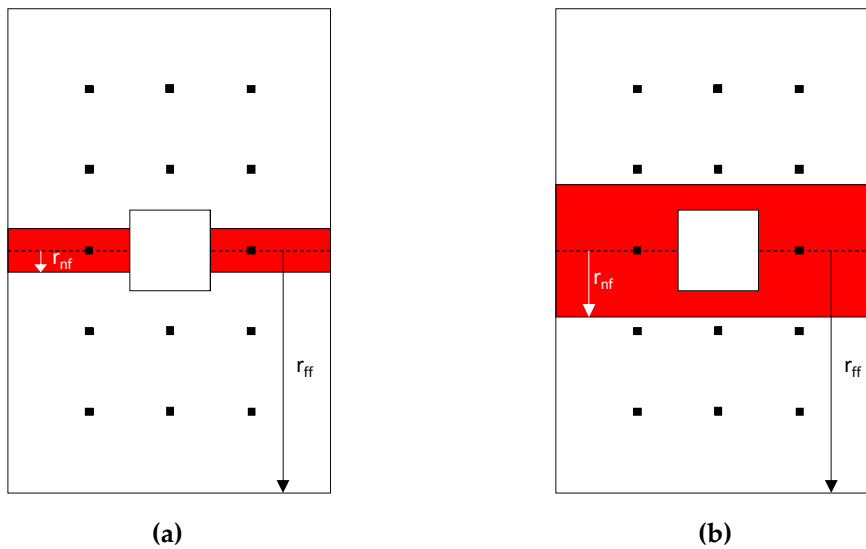
## 4.6 Base Case Fires

The base case family of fires were defined as fires that travelled linearly from one side of the structure to the other, as shown in Figure 4.3. The fire sizes considered were: 1%, 2.5%, 5%, 10%, 25%, 50% and 100% of the floor area. It was assumed that the fuel load,  $q_f$ , was 570MJ/m<sup>2</sup> and the heat release rate per unit,  $\dot{Q}''$ , was 500kW/m<sup>2</sup>. The distance to the far field for Alpert’s equation was measured from the centre of the fire at the mid-point of the building along the direction of fire travel, as shown in Figure 4.4. This creates the shortest far field distance, which in turns leads to the highest far field temperature possible for that specific scenario. This is done to err on the side of conservatism. Figure 4.4 shows the distances to the end of the near field and to the end of the far field for both the case where the near field is smaller

than the core and the case where it is larger. The near field distance is simply calculated from the geometry of the structure and the fire area for each case.



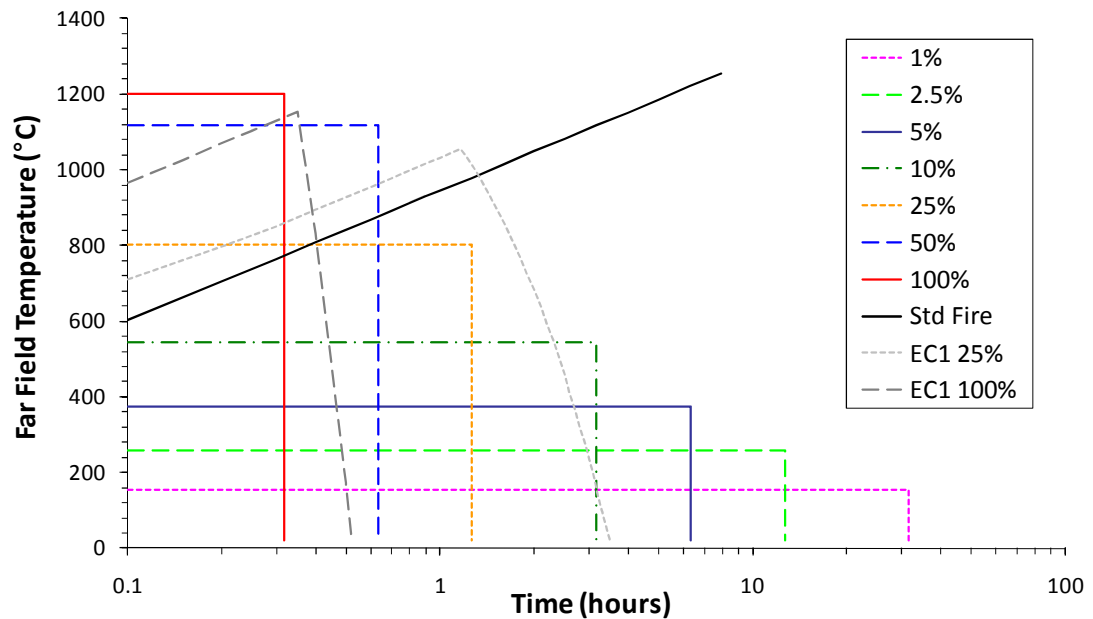
**Figure 4.3:** (a) Progression of the 2.5% fire across the floor plate; (b) Progression of the 25% fire across the floor plate. Bay numbers are indicated in both figures.



**Figure 4.4:** The measurement of  $r_{ff}$  and  $r_{nf}$  for two different indicative fire sizes: (a) small; and (b) large.

The fuel conditions above resulted in a local burning time of 19min for any single area. For example, as there were four phases in the 25% fire size, it lasted for a total burning duration of 76min, and had a far field temperature of 805°C. The near field

temperature is taken as the flame temperature, assumed to be 1200°C [13]. The 2.5% fire size, meanwhile, had a total burning duration of 760min and a far field temperature of 325°C. Figure 4.5 shows the total burning duration and far field temperatures for each of the base case fires. It should be noted for the 100% fire size, the far field temperature is the same as the near field temperature.

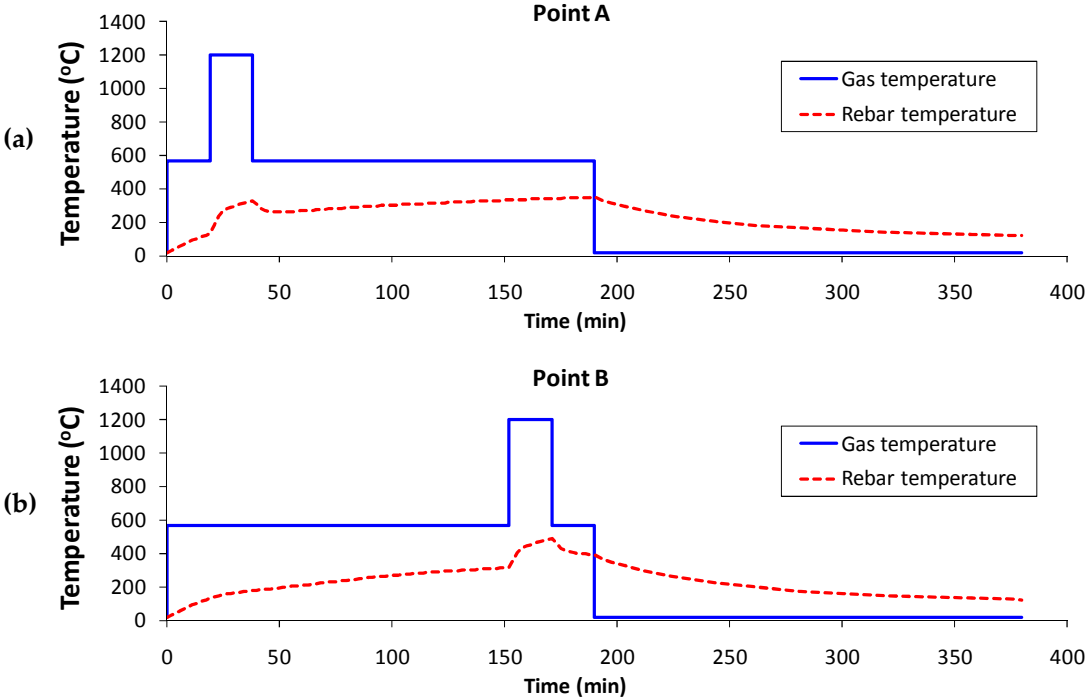


**Figure 4.5:** Far field temperatures vs. total burning durations for different fire sizes. Standard and two (“short hot” – EC1 100% and “long cool” – EC1 25%) parametric Eurocode fire curves are also shown for reference.

#### 4.6.1 Structural and Thermal Analysis

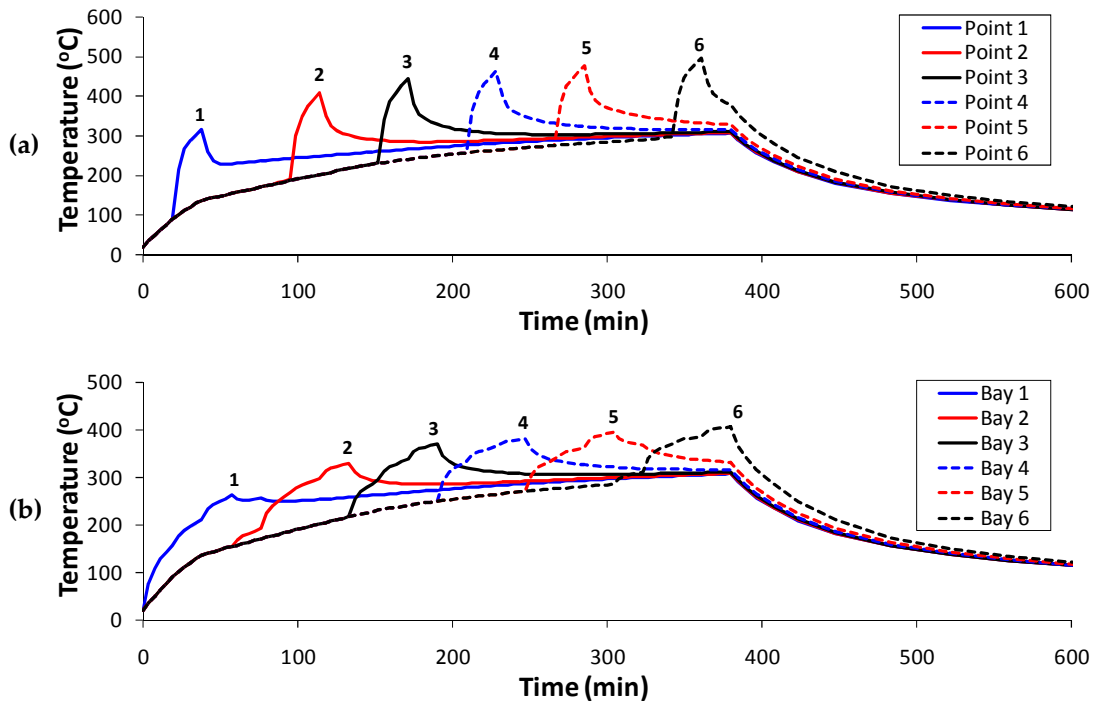
Thermal and structural analyses were conducted using the finite-element model described above. To allow meaningful conclusions to be drawn from the modelling, it should be noted that the analyses were intended to be comparative. Therefore, for the remainder of this paper, the metrics that will be used to quantify the response of the structure will be the three simple measures discussed above – temperature, strain in the tension steel, and central deflection of each bay.

Figure 4.3 shows the location of the near field part of the way through the 2.5% and 25% fire sizes. The heat transfer analyses allowed the temperature in the slab soffit rebar to be monitored. Figure 4.6 shows the gas temperatures and corresponding rebar temperatures for points A and B (indicated in Figure 4.3) during the 10% fire.



**Figure 4.6:** (a) Gas temperature and corresponding rebar temperature at point A; (b) Gas temperature and corresponding rebar temperature at point B for a 10% linearly travelling fire.

The influence of the near field on the rebar can be clearly seen as a temporary increase in temperature. The prolonged exposure of point B to the far field prior to the arrival of the near field causes the overall peak temperature to be higher than that at point A. Figure 4.7a shows a similar plot of the temperature profiles for the soffit rebar at the centre of bays 1 – 6 for the 5% fire size. It can be clearly seen that the final bay to be subjected to the near field experienced the highest temperature; the long pre-heat induced a higher maximum temperature in this bay which caused it to be most critical by this metric. This trend was the same with each of the base case fires.

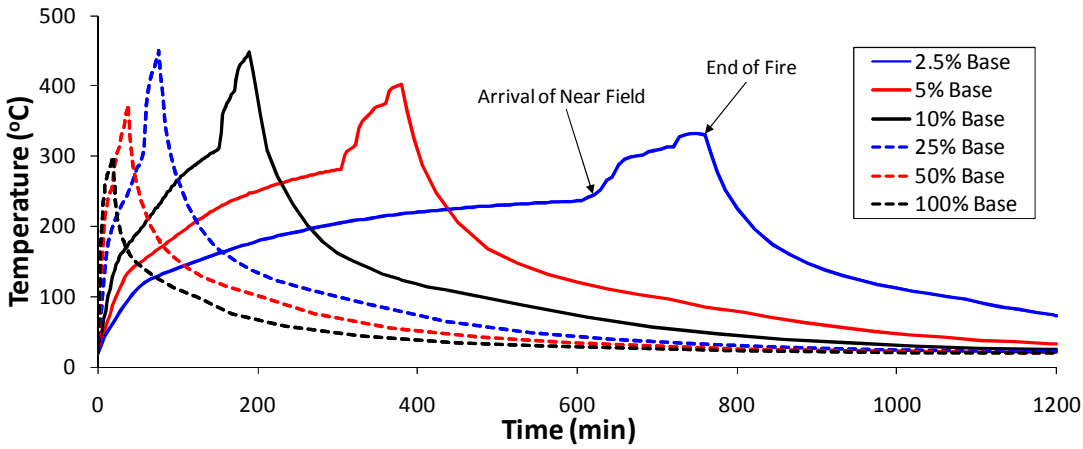


**Figure 4.7:** (a) Single point rebar temperature at the centre of bays 1 – 6 during the 5% base case fire; (b) Average rebar temperatures for the whole of bays 1 – 6 for the 5% base case fire.

Figure 4.7b shows the average temperature in the soffit rebar for each bay. Because the near field of the 5% fire size does not cover the whole area of any bay simultaneously, the average rebar temperatures are lower. The bay average rebar temperatures are a more representative measure of structural vulnerability as they will not be distorted by localized heating effects. For example, were a localized fire to heat only a tiny area of the bay, it would have minimal impact on the overall structural behaviour, but would induce high rebar temperatures. Thus, the bay average rebar temperatures will be used as the measure of rebar temperature for the remainder of this paper rather than point temperatures.

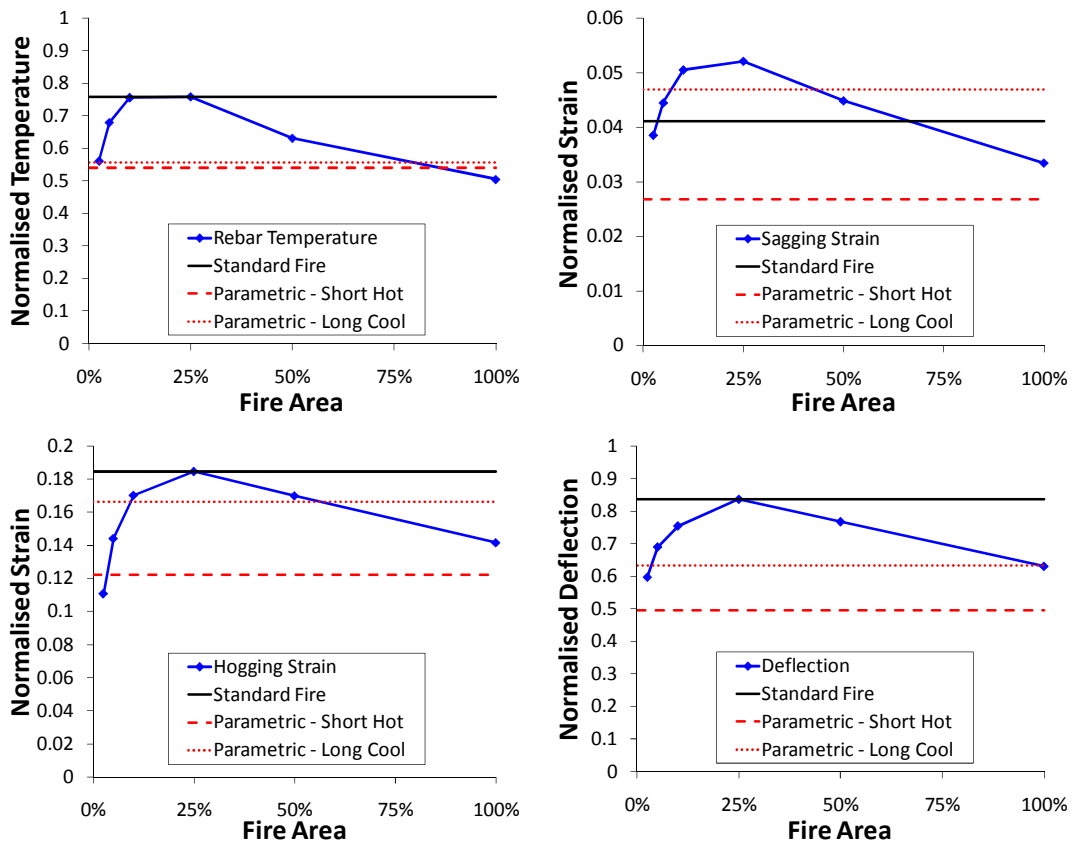
A comparison of the rebar temperatures induced in the final bay by the different fires in the family is given in Figure 4.8 and shows clearly that the highest

temperatures are caused by the medium duration fires: 10% and 25% fire sizes. For the 2.5% fire the arrival of the near field at bay 6 is labelled, as is the end of the fire.



**Figure 4.8:** Temperature profiles for the average rebar in the final bay to be heated during the base case fires.

A similar process was conducted for each of the structural measures. The absolute value of each measurement technique can be normalised with respect to the appropriate failure definition: 593°C for rebar temperature, span/20 for deflection, and 0.2 for rebar strain. It is possible therefore to observe how the level of structural distress varies with each curve in the family of fires. Figure 4.9 shows the trends for each of the measures against fire size. As a comparison the structure was also subjected to a Standard Fire, a “short hot” parametric fire and a “long cool” parametric fire. The “short hot” fire had a peak temperature of 989°C and a total fire time of 37min, and the “long cool” fire had a peak temperature of 915°C and a total fire time of 145min. Both curves were generated by the parametric temperature-time from Eurocode 1 [31] for the building being examined, varying the assumed glass breakage in the façade for the ventilation factor. The short hot fire assumed 100% glazing failure along the façade while the long cool fire assumed 25%.



**Figure 4.9:** Change in structural distress with near field area: (top left) rebar temperature, Standard Fire equivalent is 1h 37min; (top right) sagging tensile strain, value for Standard Fire given after 3h; (bottom left) hogging tensile strain, Standard Fire equivalent is 1h 18min; and (bottom right) deflection, Standard Fire equivalent is 1h 54min.

The 25% fire size induced the highest degree of structural distress in each of the failure metrics. The trend in every metric was the same: the medium sized fires (5%, 10% and 25%) caused a higher degree of structural distress than both the smaller and the larger sized fires. It is also notable, that the temperature and deflection measures show the structure as much closer to “failure” than the strain measures. For each measure, a comparison with Standard and parametric fires is also made. The parametric fires universally induced less extreme structural conditions than the medium fire size base case scenario. The worst case travelling fire was equivalent to 1hr 37min of the of a Standard Fire in terms of rebar temperature, 1hr 18min for hogging tensile stain and 1hr 54min for deflection. In contrast, the sagging strain was less than that obtained during most of the base case and “long cool” fires; this

was because there was no cooling phase during the Standard Fire so the structure was not pulled into tension.

The results of the base case fires, and their comparison with the codified fires, have shown that the traditional design methods do not necessarily produce the most onerous case for the structure. Indeed a travelling fire based on fundamental fire dynamics can induce a worse structural scenario. This is in agreement with previous work for steel structures [12, 34]. It has been shown that the medium size (and duration) fires induce the most extreme structural response; the short large fires and the long small fires are less severe for the structure. Specifically the 25% area fire produced the worst case for the structure. It has also been found that the lack of a cooling phase in the Standard Fire does not allow all the forces that are likely to develop over the course of a real fire to develop; it cannot, therefore be considered conservative [35].

## 4.7 Parametric Study

A parametric study was conducted to establish the effect of the various assumptions made in the travelling fire methodology on the predicted structural response. As the 25% fire was found to be the most severe by every metric for this structure, this fire size was used throughout the parametric study.

### 4.7.1 Far Field Definition

First, the method used to define the far field temperature was varied, and the response of the structure was monitored using the same metrics that were used in the previous section. The cases studied are described below and illustrated in Figure 4.10.

1. **Single far field (base case).** As with the previous analyses, Alpert's far field temperature profile was reduced to a single value by fourth power averaging. The progress of the fire was assumed to move suddenly, i.e. it

would jump from one quarter of the floor plate to the next after each burning time. This assumption means the fire is in four specific locations (for the 25% area fire) over the total burning duration.

2. **Two far fields.** Rather than reducing the far field to a single value for both sides of the burning area, two separate far fields were assumed, one on either side of the fire. Each far field had a unique temperature defined with the fourth power average.
3. **Alpert's temperature profile (sudden).** Rather than averaging the far field temperature as above, the continuous temperature profile defined by Alpert's equation (Eq. 4.1) was directly applied to the structure. As with the base case, the fire moved suddenly from area to area as the fuel was consumed.
4. **Alpert's temperature profile (gradual).** Alpert's temperature profile was used to define the far field, but the fire was assumed to progress gradually across the structure, rather than jumping suddenly from one area to the next.

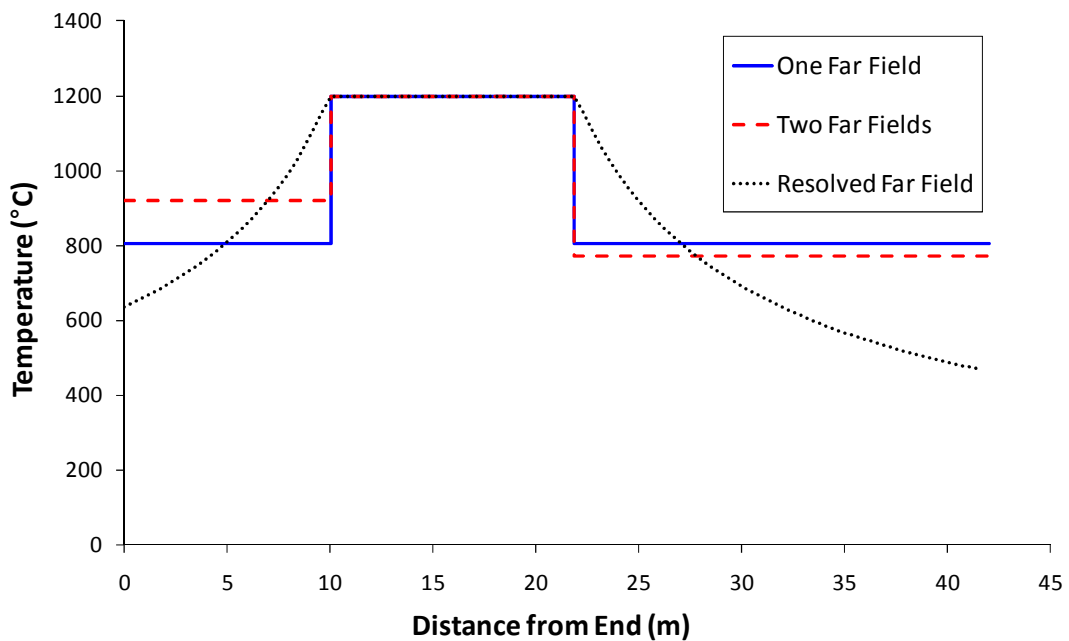


Figure 4.10: Example range of far field temperature definitions.

The results in Figure 4.11 show that there is little variation in the performance metrics between the different approaches of defining the far field temperature. As with the data in Figure 4.9 the results for each metric are normalised against the relevant failure definition: 593°C for rebar temperature, span/20 for deflection, and 0.2 for rebar strain. Of the different proposed profiles, the “Alpert – sudden” induced the greatest distress in terms of deflection (0.29m, normalised value of 0.84) and hogging tension strain (0.03, normalised value of 0.19). However, these values were only 0.5% and 3.6% in excess of the base case value, respectively. In terms of rebar temperature, the base case had the highest value (450°C) by a marginal amount (0.1% higher than the “two far fields” case) and the total variation between the largest and smallest temperature was 10.6%. The largest value in terms of the sagging tensile strain was obtained during the “Alpert – gradual” case (0.01). For this profile, the maximum strain measured was 4.7% larger than the base case equivalent.

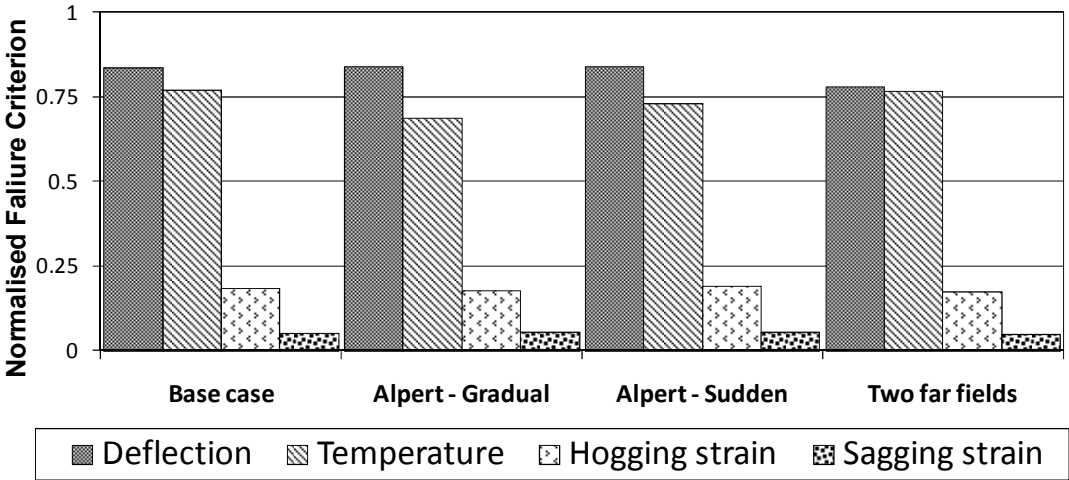


Figure 4.11: The effect of far field temperature definition on each failure metric.

This study shows that the variations induced by the different fires in the most critical structural measures are negligible. The variation in the less distressed measures was slightly larger, but still remained small (<5%). It therefore appears reasonable that the use of the simple, averaged, temperature profile, i.e. the base

case, for the whole of the far field temperature region provides appropriate results and a higher level of detail is not needed. This makes the temperature definitions in the heat transfer model significantly simpler to apply: a key consideration for the use of such an approach in a design context.

#### 4.7.2 Fire Shape and Path

The base case fire described above started at one end of the structure and then progressed linearly across the floor-plate. A real fire could follow a number of possible paths and it has long been recognised that to examine every possible fire scenario would be unfeasible due to the large number of analyses required [3]. However, since the advent of modelling techniques such as the finite-element method it has become possible to evaluate a number of different structural scenarios quickly. This paper has developed a number of fires and applied them to the same structure. In an attempt to quantify the impact that different fire paths and shapes have on the structure, this study analyses the effect of three other possible fire patterns with a fire size of 25% of the floor area. In addition to the linear base case, the different fire shapes are illustrated in Figure 4.12 and are described below:

- **Corner Fire.** Initiated in one corner of the structure and spread around the building's core. Due to symmetry, results are the same for clockwise and anti-clockwise fires.
- **Ring Fire, Outwards.** Initiated as a ring around the core, and spread concentrically outwards.
- **Ring Fire, Inwards.** Initiated in a peripheral ring around the edge of the structure, and spread concentrically inwards towards the core.

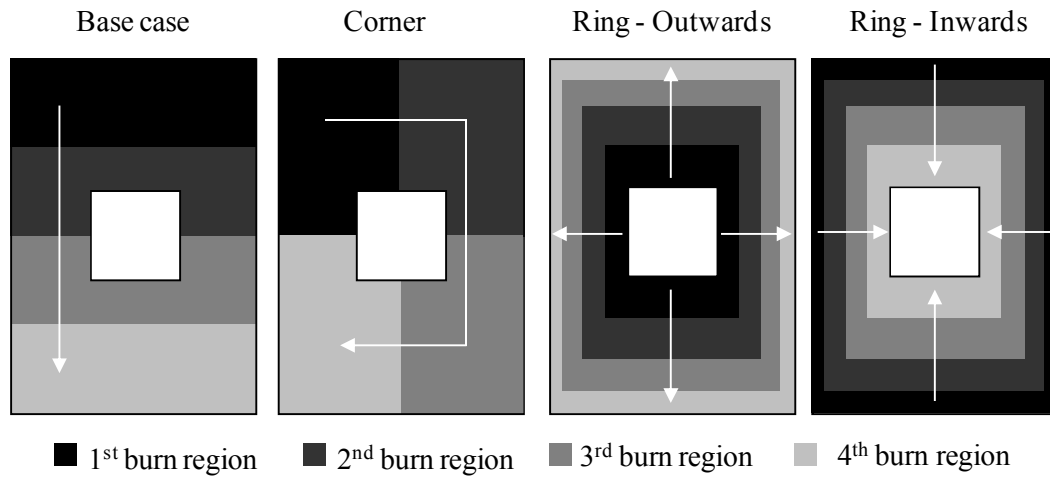


Figure 4.12: Illustration of different fire shapes and paths.

The results were broadly similar with some metrics showing an increase and some showing a decrease but there is some variation between the different fire paths. The corner fire was found to be the most severe scenario. The relative increase in comparison to the base case model was 8% for deflection (0.32m); 5% and 10% (0.04 and 0.01) for hogging and sagging strain respectively; and no change in the rebar temperature. Figure 4.13 shows the difference between the four fire shapes analysed. Therefore it can be concluded that the shape and path of the fire does have a small impact on the response of the structure.

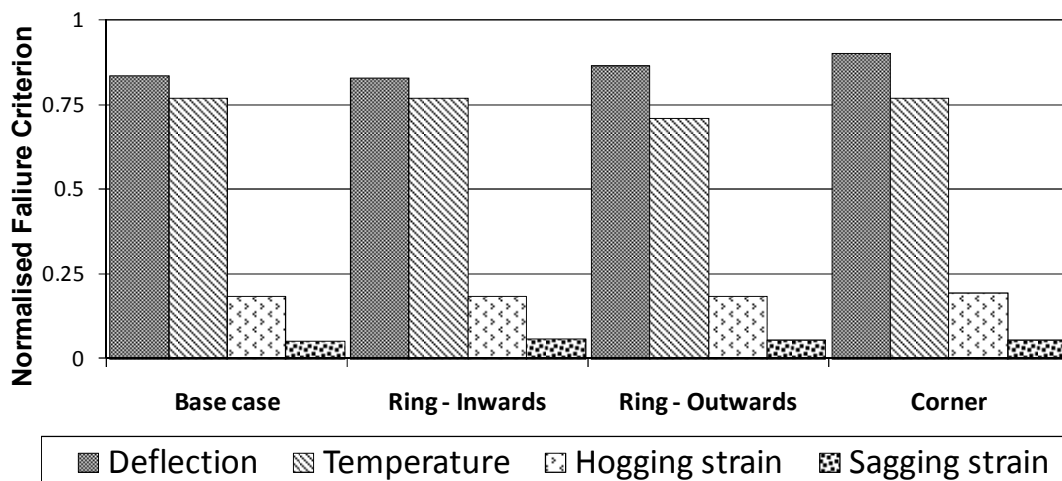


Figure 4.13: The influence of fire shape and path on the failure metrics.

## 4.8 Summary and Concluding Remarks

A comparative analysis of the impact of a number of different design fires on a concrete frame has been conducted. A new approach to defining temperature-time curves for design has been presented. The relative impact of the conventional codified curves and the new travelling fire methodology has been studied.

The travelling fire approach is based on observations from real, large building fires, and founded on the fundamental fire dynamics of a large open plan floor plate. It allows a range of realistic fires to be considered and, thus, allows structural engineers to better understand how different fires might affect the behaviour of a building. Though based on complex temperature distribution data, a simplified approach allows a single value far field temperature distribution. It has been demonstrated that this simplification is a good approximation to more complex temperature fields obtained from fundamental fire dynamics. The simplified far field approach is easily implemented in finite-element codes.

The generic concrete frame which was subjected to the various fires was the same in each of the analyses. It has thus been possible to draw strong comparative conclusions, particularly given the variety of measures used to assess the structure, which include:

- Travelling fires have a more severe impact on the performance of this structure than the Eurocode parametric fires. The Eurocode fires cannot, therefore, be considered conservative.
- The fires of medium duration and fire size are the most severe in terms of their impact on the structure.
- The 25% fire size fire was conclusively found to be the most severe by every measure used.

- The assumption of a simplified far field temperature was valid: more complex and realistic temperature profiles had little impact on the overall structural behaviour.

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# 5

## Refinement and Application of the Travelling Fires Methodology

### 5.1 Introduction

Close inspection of accidental fires in large, open-plan compartments reveals that they do not burn simultaneously throughout an entire enclosure. Instead, these fires tend to move across floor plates as flames spread, burning over a limited area at any one time. These fires have been labelled “travelling fires”.

Despite these observations, fire scenarios currently used for the structural fire design of modern buildings are based on one of two traditional methods for specifying the thermal environment; the standard temperature-time curve (which has its origins in the late 19<sup>th</sup> century [1]) or parametric temperature-time curves,

such as that specified in Eurocode 1 [2]. These methods assume uniform burning and homogeneous temperature conditions throughout a compartment, regardless of its size. These two assumptions, which have never been confirmed experimentally, lead to limitations in the use of the traditional methods in large compartments. Details of the limitations and their implications are given in the literature [3, 4, 5].

Accidental fires that have led to structural failure [6, 7, 8, 9] have been observed to travel across floor plates, and vertically between floors, rather than burn uniformly. Travelling fires have also been observed experimentally in compartments with non-uniform ventilation [10, 11, 12].

Even though the traditional methods have inherent assumptions of fire behaviour different from that observed in accidental and experimental fires, in the past they were generally deemed to be conservative, and therefore appropriate for engineering design. However, recently travelling fires have been shown to be more challenging to structures than the design fires from traditional methods [4, 13]. Moreover, recent advances in structural analysis and modelling techniques are aimed at determining the true performance of a building exposed to fire. Therefore, there is a need for a more realistic definition of fire scenarios to obtain a more accurate characterisation of building performance. Because current engineering analysis of the structural response often involves the use of sophisticated computer modelling, it is also important to ensure a consistent level of crudeness across the whole analysis [14, 15].

To address this need, a methodology that utilises physically-based fire dynamics for large enclosures, based on travelling fires, has been developed. It has been formulated to enable collaboration between fire safety engineers to define the fire environment and structural fire engineers to assess the subsequent structural behaviour, which is an identified need within the structural fire community [14, 15].

This paper presents the general framework and analytical details of this travelling fires methodology, which produces temperature fields for a range of fire sizes. These results are used to calculate the heating of a generic concrete structure. A sensitivity study is conducted to determine the relative impact of the methodology's numerical, physical and building parameters on the structure.

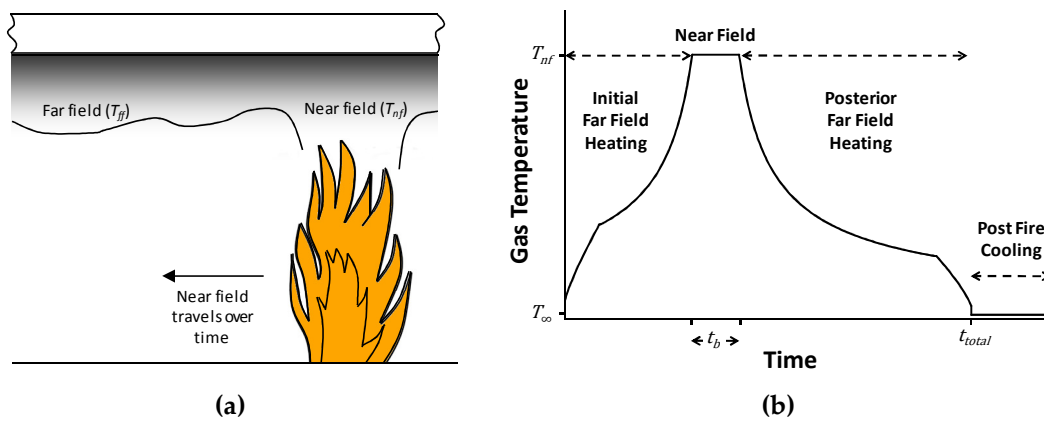
## 5.2 Travelling Fires Framework

The goal of the methodology developed in this paper is to calculate the fire-induced thermal field such that it is physically-based, compatible with the subsequent structural analysis, and accounts for the fire dynamics relevant to the specific building being studied. In order to achieve this, a fire model must be selected that provides the spatial and temporal evolution of the temperature field. This model is then applied to the particular compartment of interest.

The fire-induced thermal field is divided in two regions: the near field and the far field. These regions are relative to the fire, which travels within the compartment, and, therefore, move with it. The near field is the burning region of the fire and where structural elements are exposed directly to flames and experience the most intense heating. The far field is the region remote from the flames where structural elements are exposed to hot combustion gases (the smoke layer) but experience less intense heating than from the flames. The near and far fields are illustrated in Figure 5.1.

Because the initiation and end of the fire results in a very fast rise and decrease in gas temperature relative to the structural heating, these phases can be assumed to be instantaneous for the temperature field (see Figure 5.8 for a fast return to ambient). This is because the larger an enclosure is, the lower the importance of the thermal inertial of its linings, thus the faster the growth and decay phases will be. In other words, the transport of the hot gases in the smoke layer is faster than the heat

transfer to the surfaces. Note that the cooling of the structure is not neglected; only the brief decay phase of the fire environment is shortened.

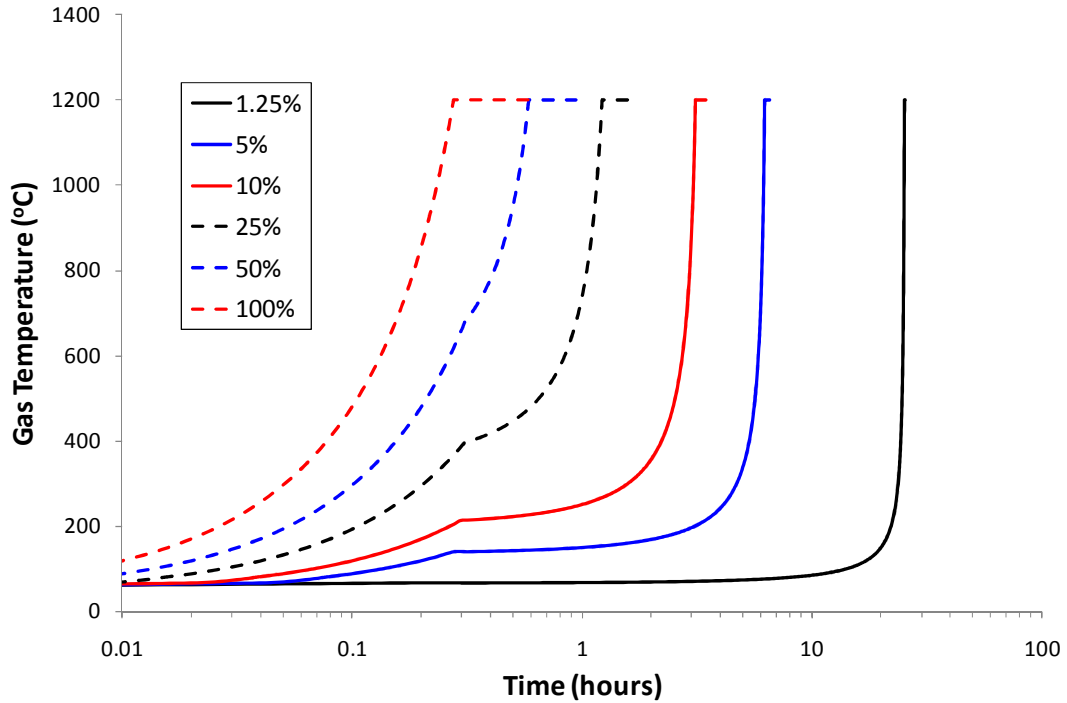


**Figure 5.1:** (a) Illustration of a travelling fire; (b) Near field and far field exposure durations at an arbitrary point within the fire compartment.

For most large compartments, travelling fires are likely to be fuel bed controlled. In fact, a recent review by Majdalani and Torero [16] of early CIB tests and the resulting analyses of compartment fire behaviour done by Philip Thomas and others highlights that ventilation controlled fires are unlikely in large enclosures and that they are not necessarily more conservative for structural analysis than fuel bed controlled fires. Majdalani and Torero note that while the different burning behaviour between ventilation and fuel bed controlled fires was clearly stated in the original studies, ventilation controlled fires have nonetheless been assumed to be the most severe case for design. Therefore traditional methods of calculating the burning rate, based on correlations for ventilation limited fires in relatively small compartments, are inappropriate for use with travelling fires.

The methodology does not assume a single, fixed fire scenario but rather accounts for a whole family of possible fires, ranging from small fires travelling across the floor plate for long durations with mostly low temperatures to large fires burning for short durations with high temperatures. Temperature-time curves for a family of fires are shown in Figure 5.2. Using the family of fires enables the methodology to

overcome the fact that the exact size of an accidental fire cannot be determined a priori. This range of fires allows identification of the most challenging heating scenarios for the structure to be used as input to the subsequent structural analysis.



**Figure 5.2:** Temperature-time curves on a log x-axis for a family of fires at the final location along the fire path. Cooling to ambient temperature starts after the last point in each curve.

Each fire in the family burns over a specific surface area, denoted as  $A_f$ , which is a percentage of the total floor area,  $A$ , of the building, ranging from 1% to 100%. Compared to this approach, the conventional methods only consider full size fires, which are analogous to the 100% fire size in this methodology. All other burning areas represent travelling fires of different sizes which are not considered in the conventional methods.

The methodology is independent of the fire model selected and can utilise simple analytical expressions or sophisticated numerical simulations. The first version of this methodology used the Computational Fluid Dynamics (CFD) code Fire

Dynamics Simulator (FDS) as the fire model [17]. Later work was developed using an analytical correlation [4, 13, 18]. The work in this paper is developed further from the earlier analytical work of Law et al. [4]. Details of each step of the methodology are given in the following section.

## 5.3 Analytical Model

The analytical correlation used, in lieu of CFD modelling, was selected for several reasons. The analytical model is simple and easy to use, while still providing the correct dynamics (see Section 5.3.2). It also provides a consistent level of crudeness with the heat transfer calculations performed to assess structural performance. And it does not have the high computational cost of CFD (which is on the order of days to calculate one fire scenario) associated with it and, therefore, enables consideration of many more scenarios and sensitivity studies than would have been practical with CFD models.

It is noted, however, that the correlation used is a simplification of the actual fire dynamics of the cases being examined and is only applicable to a limited set of scenarios where it is valid, such as a single floor without interconnection to other levels. However, given the benefits of the points listed above, the analytical correlation was deemed sufficient to progress development of the methodology.

The following sections present the details needed to calculate the temperature field for the family of fires, using the analytical correlation selected.

### 5.3.1 Burning Times

As the exact size of a potential fire in a building cannot be determined a priori, and the calculation methods for burning rates are inappropriate for large compartments, this methodology assumes the heat release rate of a fire by considering a wide range of possible sizes. It is assumed that there is a uniform fuel load across the fire path

and that the fire will burn at a constant heat release per unit area typical of the building load under study. From this, the total heat release rate is calculated by Eq. (5.1).

$$\dot{Q} = A_f \dot{Q}'' \quad (5.1)$$

where  $\dot{Q}$  is the total heat release of the fire (kW)

$A_f$  is the floor area of the fire (m<sup>2</sup>)

$\dot{Q}''$  is the heat release rate per unit area (kW/m<sup>2</sup>)

The local burning time of the fire over area,  $A_f$ , is calculated by Eq. (5.2).

$$t_b = \frac{q_f}{\dot{Q}''} \quad (5.2)$$

where  $t_b$  is the burning time (s)

$q_f$  is the fuel load density (MJ/m<sup>2</sup>)

For the case study presented below, the fuel load density,  $q_f$ , is assumed to be 570MJ/m<sup>2</sup>, as per the 80<sup>th</sup> percentile design value [19] for office buildings. The heat release rate per unit area,  $\dot{Q}''$ , is taken as 500kW/m<sup>2</sup> which is deemed to be a typical value for densely furnished spaces, as design guidance [20] gives this value for retail spaces. Based on these two values, the characteristic burning time,  $t_b$ , is calculated by Eq. (5.2) to be 19min. This time correlates well to the free-burning fire duration of domestic furniture, which Walton and Thomas [21] note is about 20min. It is also in line with Harmathy's [22] observation that a fully developed, well ventilated fire will normally last less than 30min.

Note that the burning time is independent of the burning area. Thus the 100% burning area and the 1% burning area will both consume all of the fuel over the specified area in the same time,  $t_b$ . However, a travelling fire moves from one

burning area to the next so that the total burning duration,  $t_{total}$ , across the floor plate is extended (see Eq. (5.9) in Section 5.3.3). This means that there is a longer total burning duration for smaller burning areas.

The total burning duration for a single fire size can reach a theoretical maximum, denoted as  $t_{total}^*$ , which is equal to the local burning time multiplied by the ratio of floor area to the fire size, plus one additional local burning time. For example, a 25% fire has a ratio of floor area to fire area of four, so adding one local burning time to this gives five times the local burning time, or 95min, for the total burning duration. Similarly, the maximum total burning duration for a 1% fire is 1919min. For full details of the derivation of  $t_{total}^*$ , see Eq. (5.10) in Section 5.3.3.

### 5.3.2 Near Field vs. Far Field

The near field is dominated by the presence of flames. The maximum possible structural heating would result from direct contact of the flames and a structural element. Hence it is assumed that there is direct contact and peak flame temperatures are used in this methodology. These temperatures have been measured in small fires in the range of 800 to 1000°C [23] and up to 1200°C in larger fires [24]. The maximum value of 1200°C is chosen here for the near field temperature to represent worst case conditions. A sensitivity study on the effect of this parameter value over the experimental range of peak flame temperatures is presented in Section 5.5.7.

The far field temperature decreases with distance from the fire. The maximum exposure to hot gases results when the structural element is on the exposed side of the ceiling. Therefore temperatures at the ceiling are used in this methodology. An analytical expression capturing the decrease of temperature with distance as a function of the fire heat release rate would take the general form given in Eq. (5.3).

$$T_{ff}(x) = c\dot{Q}_c^\alpha x^{-\beta} \quad (5.3)$$

where  $T_{ff}$  is the far field temperature ( $^{\circ}\text{C}$ )

$\dot{Q}_c$  is the convective heat release rate (W)

$c$  is a constant parameter related to geometry and physical properties (-)

$x$  is the horizontal distance from the fire (m)

$\alpha$  is the power law coefficient for heat release rate (-)

$\beta$  is the power law coefficient for distance (-)

The decrease with distance is due to the incremental mixing of hot gases with fresh air as they flow away from the fire source. This is a similar mixing process that takes place in a vertical turbulent fire plume. The scale analysis of an inert mixing plume [24, 25] gives  $\alpha$  of 2/3 and  $\beta$  of 5/3.

The experimental and theoretical work by Alpert [26] provides the full expression and the coefficients valid for an axi-symmetric, unconfined ceiling jet as a function of radial distance from the fire centre. The correlation is given below in Eq. (5.4). Alpert found experimentally that  $\alpha$  and  $\beta$  are both 2/3, and that there is a dependence on the inverse of the ceiling height (thus yielding a combined power law coefficient for the spatial distance of 5/3 as predicted by the scale analysis).

$$T_{max} - T_{\infty} = \frac{5.38(\dot{Q}/r)^{2/3}}{H} \quad (5.4)$$

where  $T_{max}$  is the maximum ceiling jet temperature( $^{\circ}\text{C}$ )

$T_{\infty}$  is the ambient temperature ( $^{\circ}\text{C}$ )

$\dot{Q}$  is the total heat release rate (kW)

$r$  is the distance from the centre of the fire (m)

$H$  is the floor to ceiling height (m)

The Alpert correlation uses the total heat release rate, rather than its convective portion which is related to buoyancy. This is due to the fact that the heat release rates of pool fires, which were the basis of the correlation, are often reported as total values and not convective [27]. The specific pool fires used for the development of the Alpert correlations were alcohol pool fires, in which the radiative fraction is negligible. Therefore, for application in this methodology, the heat release rate is assumed to be purely convective, i.e. the radiative fraction is taken to be zero.

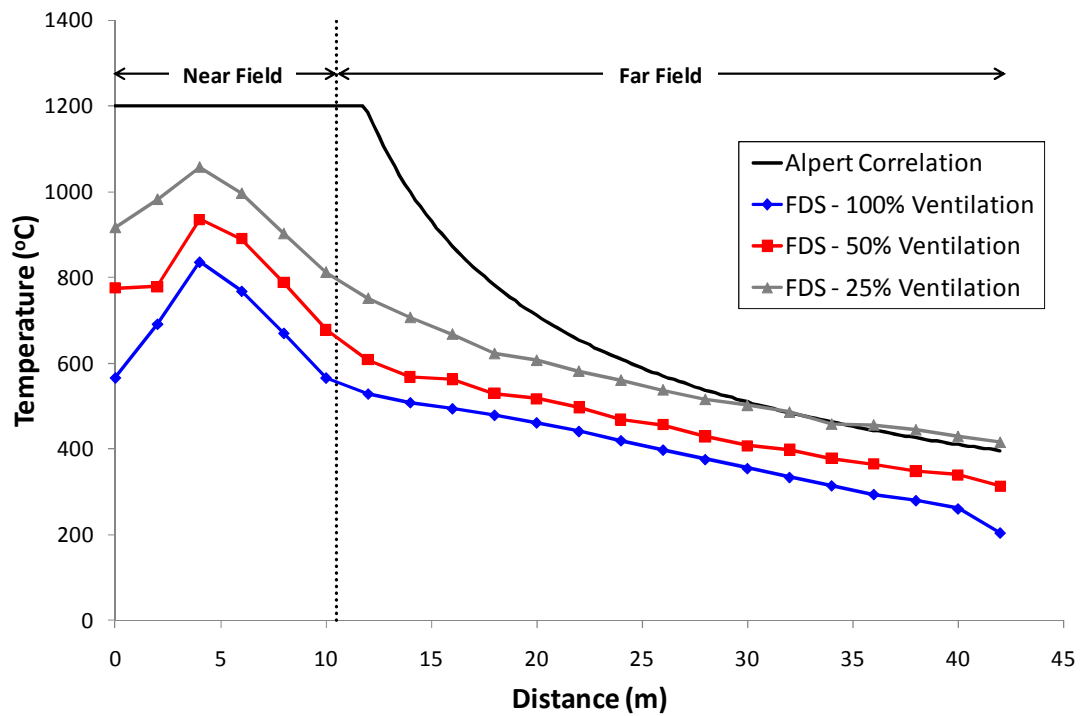
Alpert gives a piecewise equation for maximum ceiling jet temperatures to describe the near field ( $r/H \leq 0.18$ ) and far field ( $r/H > 0.18$ ) temperatures. But only the far field equation is used here. The methodology assumes the near field to be the flame temperature and does not use the expression given by Alpert. If the results of Eq. (5.4) exceed the specified near field temperature at any point, they are capped at the flame temperature.

This correlation was used in previous work of this methodology [4, 13, 18]. Its use for horizontally travelling fires requires the further assumption that the coefficient,  $c$ , does not change significantly when the linear distance,  $x$ , replaces the radial distance,  $r$ , given by Alpert (planar vs. axi-symmetrical configurations). Therefore, the linear distance,  $x$ , is used in the methodology.

It is also noted that the correlation assumes an unconfined ceiling with no accumulated smoke layer. However, these strict limitations are ignored in the application to this methodology. This has been done as it is a simple correlation and was chosen to provide an approximate and straightforward calculation of the temperature field that is sufficient to progress the development of the methodology. Further sophistication and accuracy could be added to this framework as needed.

As a point of comparison between the axi-symmetric ceiling jet correlation and a planar case, a set of CFD simulations were run using FDS v5.5.3. The simulations examined the temperature decrease with linear distance from a 147MW fire (the

25% fire size examined in Section 5.5) over a 28m wide strip located at one end of a large compartment 42m long by 28m wide and 3.6m high (see Section 5.4 for details). A grid sensitivity study was conducted to ensure good resolution and the final cell size was set at 40cm. Three cases were investigated: 100% ventilation opening (the whole façade is open), 50% ventilation opening, and 25% ventilation opening. While very different ventilation scenarios were investigated, Figure 5.3 shows that the ceiling jet correlation provides a similar decay with distance (similar  $\beta$  value) to the FDS models. The temperature agreement is better at larger distances.



**Figure 5.3:** Comparison of Alpert’s ceiling jet correlation with three FDS models of varying ventilation for a 147MW, 28m wide fire (25% fire size) burning at one end of the compartment (see Section 5.4 for details).

The values of  $\beta$  for the three FDS curves are 0.605 for 100% ventilation, 0.502 for 50% ventilation, and 0.463 for 25% ventilation. These values are similar to the  $2/3 \beta$  value from Alpert’s correlation. The modelling results provide confidence that the ceiling jet correlation, while not exactly capturing the fire dynamics of each scenario of interest here, gives appropriate and conservative results.

The previous work of this methodology [4, 13, 17, 18] took a single representative temperature for the far field for each fire size, independent of distance. The work in this paper, however, relaxes this simplification and allows for spatially varying far field temperatures to be carried into the heating calculations. While this creates more information to pass to the structural analysis, it provides a more accurate representation of the fire dynamics for each scenario, which may be particularly important for analyses of whole frame behaviour.

### 5.3.3 Spatial Discretisation

It is assumed that the fire extends the whole width of the building and travels in a linear path along the structure's length. Other fire paths are possible but results shown in [4] demonstrate that they do not greatly alter the structural response. Thus a single linear path is chosen for this further development of the methodology. As the far field temperature is assumed uniform along the width of the building but varies along its length for the assumed linear path, the problem is treated as one-dimensional. Thus the far field temperature for any given fire size can be calculated at any position in the structure by its linear distance from the fire. This discretisation is similar to the strips examined by Clifton in his Large Firecell Model [28].

The fire is assumed to travel at a constant spread rate,  $s$ , across the floor plate. This is calculated by Eq. (5.5) and is related to burning time and fire size.

$$s = \frac{L_f}{t_b} \quad (5.5)$$

where  $s$  is the spread rate (m/s)

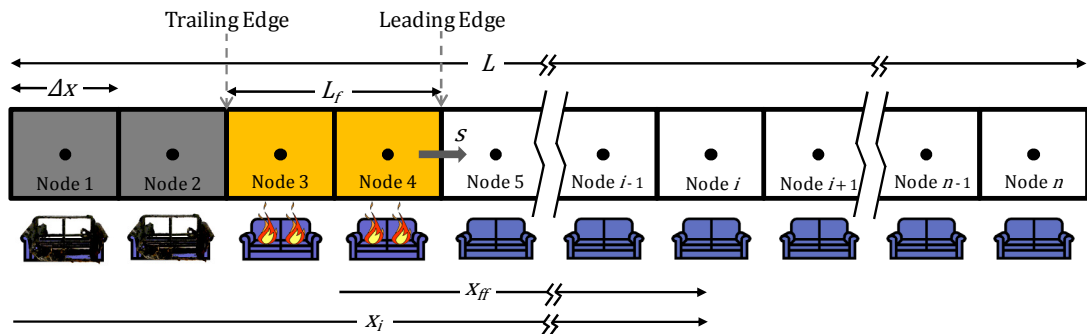
$L_f$  is the length of the fire (m)

Given that there is a fixed local burning time (based on the assumption of a uniform fuel load density and a constant heat release rate per unit area, as explained in Section 5.3.1), there is a one-to-one relationship between fire size and spread rate.

This corresponds with the logic that the bigger the fire, the faster it moves. For example, a fire that is 50% of the floor area ( $L_f = 0.5L$ ) would have a spread rate five times faster than a 10% fire ( $L_f = 0.1L$ ), as the local burning time is the same for both.

To track the fire location over time and enable calculation of the far field temperature at various distances, the building is broken up into numerous nodes, each with a fixed width  $\Delta x$  (also referred to as the grid size). Each node has a single far field temperature at any given time. Therefore the more elements that are used, the better resolved the far field temperature is (see Section 5.5.2). As the fire travels across the floor plate, nodes go from being unburnt, to on fire, to burnt out.

Figure 5.4 illustrates the one-dimensional discretisation of the building showing the grid size ( $\Delta x$ ), total length ( $L$ ), fire length ( $L_f$ ), far field distance ( $x_{ff}$ ), node references, and the leading and trailing edges of the fire. The near field distance is half the fire length, while the far field distance ( $x_{ff}$ ) is taken from the fire centre to the node being examined (node  $i$ ).



**Figure 5.4:** Illustration of spatial discretisation, showing the nodes of grid size,  $\Delta x$ , and the characteristic lengths of the problem. The fire (orange) travels at spread rate,  $s$ , towards the unburnt nodes (white), leaving burnt-out nodes (grey) behind.

Each node can be described by its index, varying from 1 to  $n$ . The distance,  $x_i$ , from a fixed reference point, taken here as the left end of the structure where the fire is assumed to start, to another point can be described by Eq. (5.6).

$$x_i = (i - 0.5)\Delta x \quad (5.6)$$

where  $x_i$  is the position relative to the end of the structure (m)

$i$  is the node reference (-)

$\Delta x$  is the grid size (m), also given by  $L/n$

The relative positions of the fire location and the node can be tracked over time to give a full transient evolution of the temperature field, including the passage of the near and far fields (see Figure 5.1b and Figure 5.2). In order to adequately resolve the movement of the fire, the time step,  $\Delta t^*$ , is determined by Eq. (5.7).

$$\Delta t^* = \frac{\Delta x}{s} \quad (5.7)$$

This definition allows the time step to capture the movement of the fire from one node to the next. If the time step is longer than that calculated by Eq. (5.7), then important information is lost. However, note that there is no benefit in making a smaller time step. This is because a node cannot be partially occupied by the fire, and thus each node has only one temperature for each time step. A finer time step would yield consecutive times with the same temperature. Therefore the time step in this work is always set by Eq. (5.7).

The time the fire spends at one node location,  $t_i$ , is the sum of the travel time across the node plus one local burning time. The whole node is assumed to start burning when the leading edge of the fire enters from the near side. Then the whole node is burnt out when the trailing edge of the fire passes the far side. This is given by Eq. (5.8).

$$t_i = \frac{\Delta x}{s} + t_b \quad (5.8)$$

As the fire travels across  $n - 1$  nodes (the initial condition has node 1 burning at  $t = 0$ ), the total burning duration,  $t_{total}$ , is the travel time across the rest of the floor plate plus one burning time. This fact, plus noting that  $n = L/\Delta x$ , means the total burning duration is given by Eq. (5.9).

$$t_{total} = t_b \left( \frac{L - \Delta x}{L_f} + 1 \right) \quad (5.9)$$

As can be seen from Eq. (5.9), the total burning duration is a multiple of the local burning time. This multiple of the local burning time is greater for smaller fire sizes, meaning longer total burning durations. This explains why travelling fires account for the longest burning fires that can take place in a large compartment and, indeed, corresponds well to those observed in accidental fires [3].

Note that the total burning time also depends on the grid size (due to the initial condition). The largest grid size that can be used to ensure that a given fire size is fully resolved is  $\Delta x = L_f$ . A larger grid size would lead to the fire only occupying a portion of any node, which is inconsistent with the assumptions of this methodology. Placing this maximum grid size in Eq. (5.9), gives a total burning time of  $t_{total} = t_b(L/L_f)$ . For example, the total burning duration for a 25% fire is 76min, which is four times the local burning time (19min). The approach taken in earlier work [4, 13, 18] used the largest grid size only and therefore had total burning durations along these lines. However, as the grid size is reduced, the total burning duration increases. The longest possible total burning duration, denoted as  $t_{total}^*$ , is the limit of  $t_{total}$  as the grid size approaches zero (the smallest possible grid size), as given in Eq. (5.10).

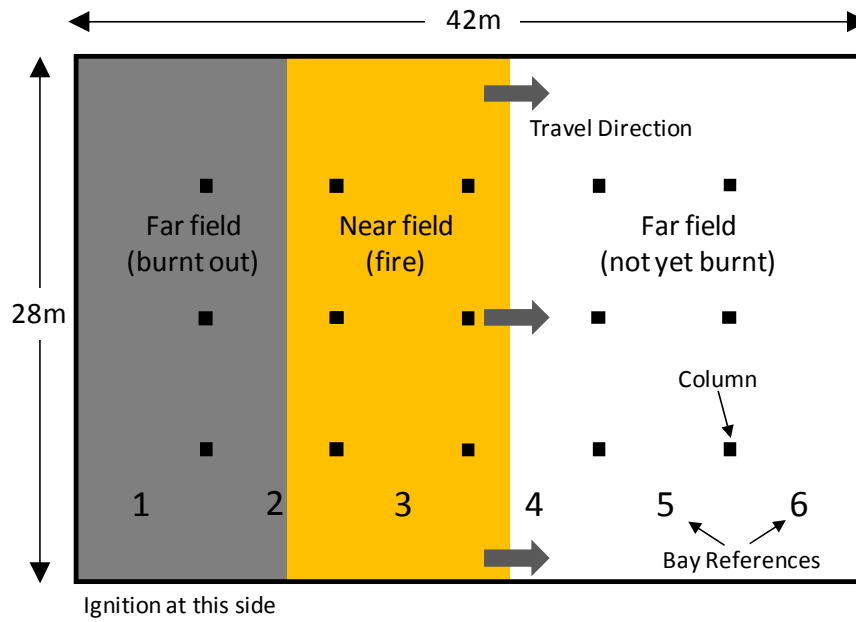
$$t_{total}^* = \lim_{\Delta x \rightarrow 0} t_{total} = t_b \left( \frac{L}{L_f} + 1 \right) \quad (5.10)$$

This means that the total burning duration is up to one local burning time longer with a fine grid resolution than with a coarse one. For the same 25% fire size example, the total burning duration with a very well resolved grid would approach five times the local burning time, or 95min. This additional burning time, which was not considered in previous versions of the methodology, represents the time period of initial fire growth before the fire reaches its full size and the final stages of the fire as it burns out and is again smaller than its full size. This is not accounted for in the coarse grid case, which assumes the fire initialises and burns out at its peak size.

## 5.4 Application to a Generic Structure

The travelling fires methodology presented here is applied here to a case study of a generic concrete frame, shown in Figure 5.5. The structure is based on that used in Law et al. [4], but without the central core. The compartment is 42m long, 28m wide and 3.6m high. There are six structural bays along the length of the building, and four across its width. Each bay is 7m x 7m. The fire is assumed to ignite at one end of the structure, occupy the full width and burn along its length over time as illustrated in Figure 5.5.

A family of fires was investigated with sizes ranging from 1% to 100% of the floor plate. A selection of fires is given in Table 5.1, showing the fire size and area, the heat release rate calculated from Eq. (5.1), the maximum total burning duration from Eq. (5.10), and the spread rate from Eq. (5.5).



**Figure 5.5:** The generic concrete structure used for the case study.

Fire size	$A_f$ (m <sup>2</sup> )	$\dot{Q}$ (MW)	$t_{total}^*$ (min)	$s$ (m/min)
1%	11.8	5.9	1919	0.02
2.5%	29.4	14.7	779	0.06
5%	58.8	29.4	399	0.11
10%	117.6	58.8	209	0.22
25%	294	147	95	0.55
50%	588	294	57	1.1
75%	882	441	44.3	1.7
100%	1176	588	38	2.2

**Table 5.1:** A selection from the family of fires.

The burning durations of the larger fire sizes are of the same order of magnitude as those predicted by the traditional methods [2]. The smaller fire sizes have burning durations on the order of those observed in large, accidental fires [7, 8, 9]. For example, the One Meridian Plaza fire in Philadelphia in 1991, which had horizontal and vertical flame spread, lasted for almost 19 hours [29]. The range of spread rates from the family of fires also corresponds well with physical values. Quintiere [30] gives the rough order of magnitude of lateral fire spread on thick solids as 0.1cm/s

(0.06m/min) and of “forest and urban fire spread” between 1 and 100cm/s (0.6 to 60m/min). This again highlights the advantage of considering a range of fire sizes in this methodology, as the burning duration and spread rate of an accidental fire cannot be calculated a priori.

The family of fires created was used to generate transient gas phase temperature fields across the structure. The temperature fields were then used as input to calculate the resulting in-depth concrete temperature at the rebar location as a simple measure of structural performance. The hotter the rebar temperature, the poorer the structural performance is deemed to be. One-dimensional conductive heat transfer inside the material was considered with boundary conditions for convective and radiant heating from the gas phase as well as reradiation. The heat transfer was solved by means of finite differences, as detailed in Appendix A. Law et al. [4] showed that the average rebar temperature across a bay is a more critical parameter for the structural response than that of a single point. Therefore to obtain the bay average rebar temperatures (referred to as the bay temperature), the average across the whole bay is calculated from results of the one-dimensional, in-depth heat transfer method at each node.

An alternative to this approach would be to use a three-dimensional heat transfer method and then calculate the full structural response by use of a detailed Finite Element Model (FEM). This was the approach taken in the work done by Law et al. [4]. For comparison, the bay average temperature results of the method used in this paper were found to be between 7 to 15% higher than that calculated by Law et al. Therefore this method is deemed appropriate, especially considering the differences in comparison to a FEM approach (one vs. three-dimensional heat transfer, constant vs. temperature dependent concrete properties, and varying heat transfer formulations). The simple approach used here allows for rapid calculation of a large variety of parameters which would be computationally restrictive to do with full FEM analyses.

## 5.5 Parameter Sensitivity Study

One aim of this methodology is to allow fire safety engineers to interface with structural fire engineers to determine the most appropriate design fire scenarios prior to the detailed structural analysis. It is the intent of this sensitivity study to highlight the important parameters that should be considered in design.

The parameter values for the base case scenario and the ranges investigated are given in Table 5.2. Unless specified otherwise, the base case values are used. The study includes building, physical, and numerical parameters. Building parameters are the actual quantities related to the building structure and its contents. Changes in these parameters come from differing building designs or uses. Physical parameters are those related to the temperature field and heat transfer mechanisms. Numerical parameters are those required to generate the temperature fields and heating but without physical meaning, such as the grid size. These last two sets of parameters do not depend on the building design or its use, but on the theoretical or numerical aspects of the methodology. As the fire size is the fundamental input variable to the methodology, it is not classified as a parameter but a variable.

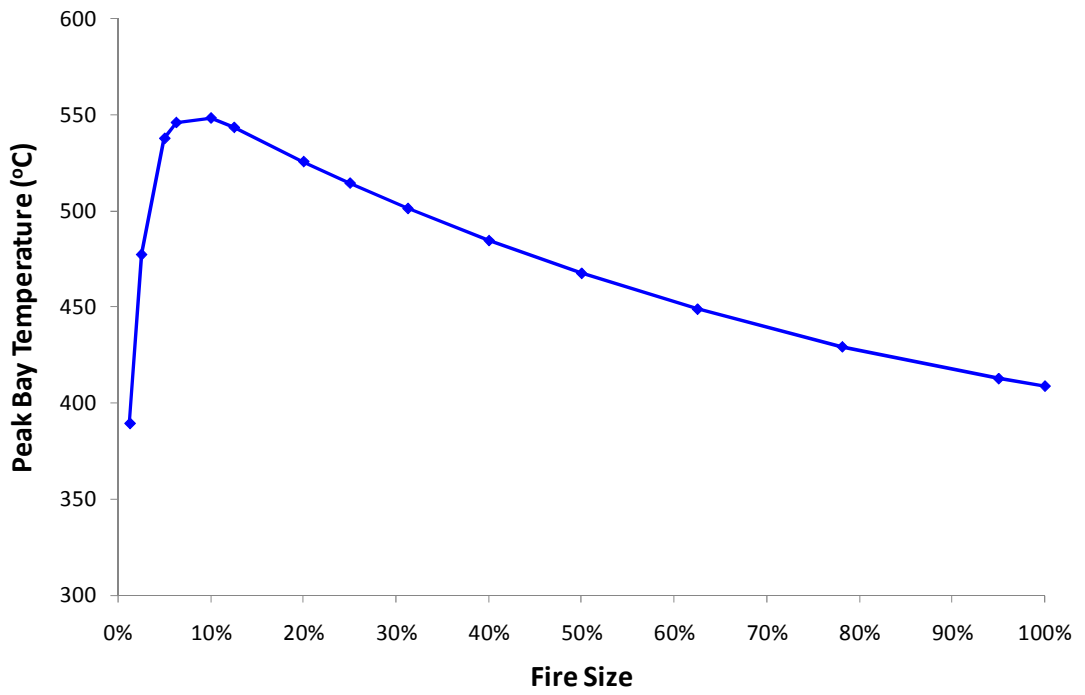
The following sections present the sensitivity of each of the parameters in Table 5.2.

### 5.5.1 Fire Size

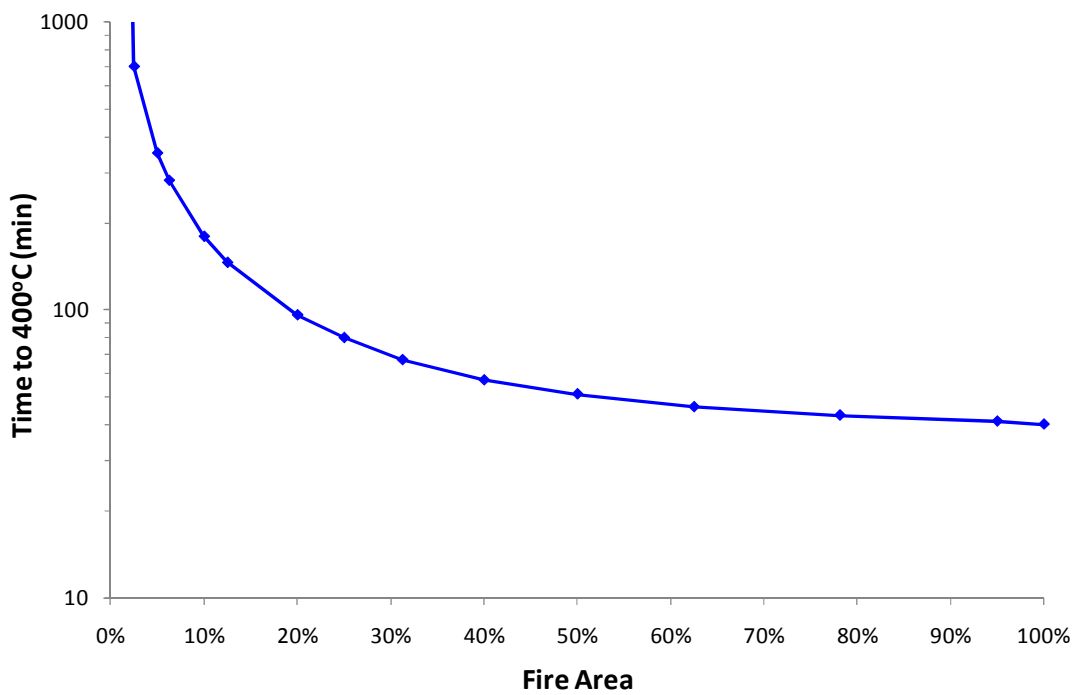
Figure 5.6a shows the variation of peak rebar temperature with fire size ranging from 1.25% to 100% for a grid size of 0.2625m. This grid size was selected as it divides evenly amongst a large number of fire sizes.

Parameter	Range	Base Case	Parameter Type	Comment
Fire Size ( $A_f$ )	1% – 100% of floor plate	10%	Main variable	Range is parametrically generated to cover all possibilities. Base case value determined by analysis in Section 5.5.1.
Grid Size ( $\Delta x$ )	0.21 – 42m	1.05m	Numerical	Range is to have a well resolved grid for the smallest fire (1%) to the coarsest possible for the largest fire (100%). Base case value determined by analysis in Section 5.5.2.
Rebar Depth ( $d_r$ )	20 – 50mm	42mm	Building	Range taken to be representative of typical range in real buildings. Base case value as per the design of the case study building [4].
Bay Location	1 <sup>st</sup> – 6 <sup>th</sup> bay	6 <sup>th</sup> bay	Building	Range is all six bays of the structure. Base case value selected as it is the most onerous for the structure as shown in Section 5.5.4.
Bay Size ( $L_b$ )	1.05 – 21m	7m	Building	Range is from the bay being the base case grid size (1.05m) to half the structure's length (21m). Base case value as per the design of the case study building [4].
Fuel Load Density ( $q_f$ )	285 – 1500MJ/m <sup>2</sup>	570MJ/m <sup>2</sup>	Building	Range covers sparsely furnished (classroom) to densely loaded (library) spaces. Base case value is taken as the 80 <sup>th</sup> percentile design value [19] for office buildings.
HRR per Unit Area ( $\dot{Q}''$ )	200 – 800kW/m <sup>2</sup>	500kW/m <sup>2</sup>	Building	Range taken for representative values of real fuels in non-industrial buildings [31]. Base case value is taken as densely furnished office [20].
Emissivity ( $\epsilon$ )	0.2 – 1	0.7	Physical	Range taken to test sensitivity; however values in an accidental fire are expected to be above 0.5. Base case value is taken from Eurocode guidance [2].
Convective Coefficient ( $h_c$ )	10 – 100W/m <sup>2</sup> K	35W/m <sup>2</sup> K	Physical	Range taken to represent bounds in a fire condition [32]. Base case value is taken from Eurocode guidance [2].
Near Field Temperature ( $T_{nf}$ )	800 – 1200°C	1200°C	Physical	Range taken to represent bounds of compartment flame temperatures [23, 24]. The base case is taken as the upper end of the range to represent worst case conditions and provide similarity to earlier work [4].
Structural Material	Concrete or Steel	Concrete	Building	Two structure types have been considered: concrete and steel. This paper predominately focuses on concrete, but some comparison is made for three steel beams: unprotected, 60min fire rated, and 120min fire rated.

**Table 5.2:** Parameter values for the base case and ranges investigated.



(a)



(b)

**Figure 5.6:** (a) Peak bay temperatures vs. fire size for  $\Delta x = 0.2625\text{m}$ ; (b) Time for bay rebar temperatures to reach  $400^\circ\text{C}$  on a log scale for time.

Fire sizes between 5% and 20% result in the largest bay temperatures (between 538 and 548°C) and thus are the most challenging for the structure. The maximum peak bay temperature is 548°C for a 10% fire. Note that both a very small fire (2%) and a very large fire (100%) result in the same peak bay temperature of 410°C. The smaller fire sizes have long durations, but relatively low far field temperatures. The larger fire sizes have higher far field temperatures, but for shorter durations. The maximum rebar temperature found for the 10% fire size results from an optimum heating balance between far field temperature and duration. These results are similar to conclusions of work previously reported [4, 13].

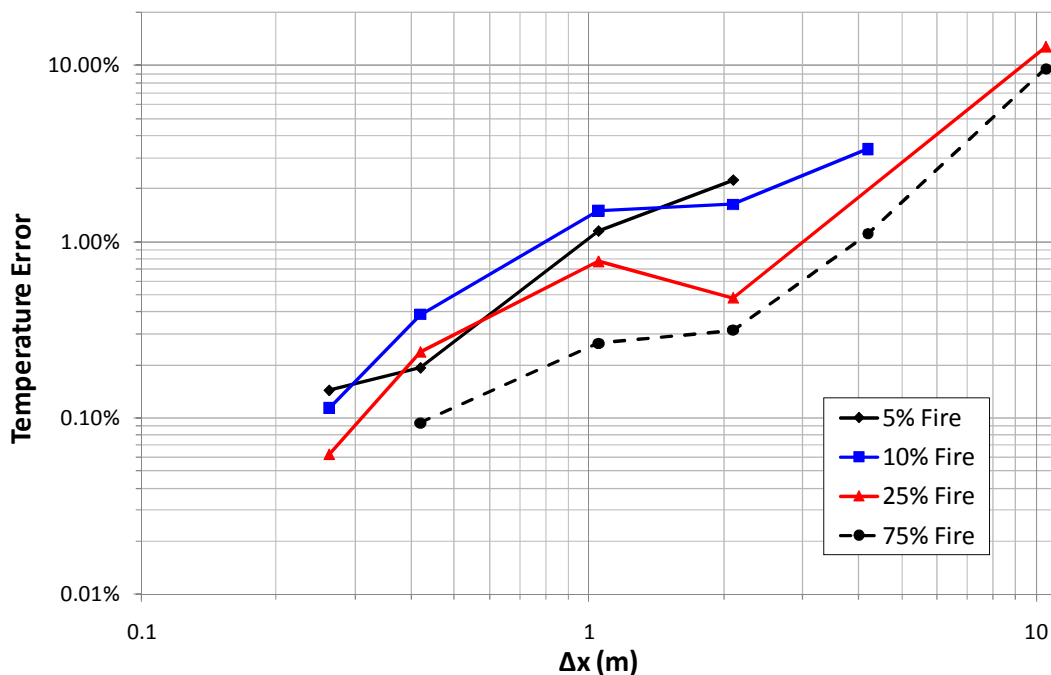
Because the most challenging scenario is the 10% fire size, it is used as the base case for the rest of this sensitivity study.

Figure 5.6b gives the time for bay temperatures to reach 400°C. A reference value of 400°C was selected for comparison of heating times as this was a bay rebar temperature reached by all but the smallest fire size (1.25%). It shows that the larger fire sizes reach this temperature more quickly than the smaller ones, even though they ultimately do not reach the same peak temperature. Note, however, that the time for the bay rebar to reach a specified temperature for a travelling fire is dependent on the location of the bay relative to the fire's path, i.e. the time of near field arrival relative to the total burning duration. This is explored further in Section 5.5.4.

### 5.5.2 Grid Size

The grid size was varied in a series of cases to ensure that the number of nodes in the discretisation scheme is high enough to properly resolve the dynamics of the problem. The grid size has an impact on three parts of the methodology: the resolution of the far field temperature in Eq. (5.4), the total burning duration in Eq. (5.9), and the resolution of a bay ( $L_b/\Delta x$ ). The impacts of these parameters are explored below.

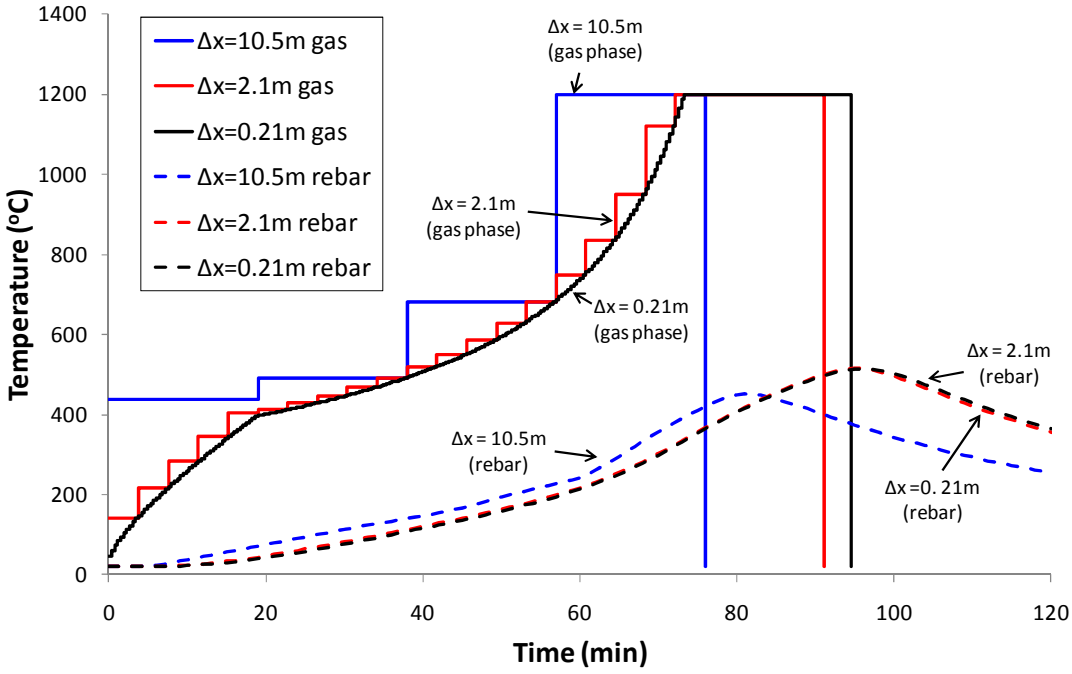
Figure 5.7 shows the error of the peak bay temperature relative to the finest grid against varying grid sizes. The finest grid size used for any calculation was 0.21m, which is fine enough to include more than one node across the smallest fire size (1%). The smaller the grid size, the lower the error, thus proving the grid independence of the model. A grid size of 1.05m gives an error of approximately 1% for several fire sizes, including the base case 10% fire size, and therefore has been selected as the base case grid size.



**Figure 5.7:** Error in the peak bay temperature relative to finest grid ( $\Delta x = 0.21\text{m}$ ) vs. grid size for a range of fire sizes.

The evolution of the gas temperature and the resulting bay temperatures for the last bay (Bay 6) at the far end of the structure (node n) are shown in Figure 5.8 for three grid sizes: coarse ( $\Delta x = 10.5\text{m}$ ), medium ( $\Delta x = 2.1\text{m}$ ), and fine ( $\Delta x = 0.21\text{m}$ ). For the course grid, the peak bay temperature was lower (by  $63^\circ\text{C}$ , difference of 12.7%) and arrived earlier (by 15min, difference of 15.6%) than for the fine grid which resulted in a peak bay temperature of  $514^\circ\text{C}$  at 96min after ignition. The results of the medium grid are very similar to the fine grid ( $517^\circ\text{C}$  peak bay temperature at

95min). Given the differences in structural heating resulting from the coarse and fine grids, and the similarities of heating from the medium and fine grids, the model is concluded to be grid independent for grid sizes of 2.1m and finer.



**Figure 5.8:** Gas phase and resulting bay temperatures vs. time at the far end of the structure (Bay 6) for coarse ( $\Delta x = 10.5\text{m}$ ), medium ( $\Delta x = 2.1\text{m}$ ), and fine ( $\Delta x = 0.21\text{m}$ ) grids.

The change of slope in the gas phase curves at 19min is due to the growth of the fire to its full size prior to that time. Note that these bay temperature results are for the last bay in the compartment. Thus when the fire ends, the gas temperature returns immediately to ambient. After that, the rebar is still heated from the thermal wave passing through the slab but then slowly cools at a rate controlled by the heat transfer in the concrete. This cooling phase and its relationship to whole frame response during a fire are of great importance to structural engineering [33, 34, 35].

The more well resolved the compartment, the longer the total burning duration is, eventually approaching  $t_{total}^*$  as can be seen from Eqs (5.9) and (5.10). For the gas

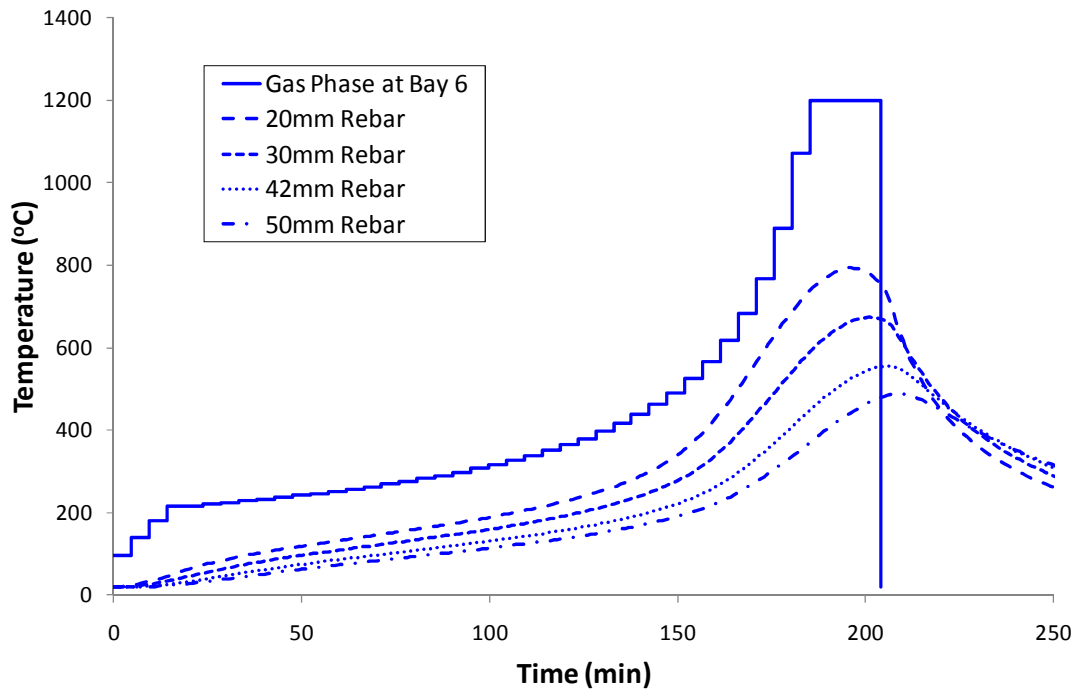
phase temperatures shown in Figure 5.8,  $t_{total}$  is 80% of the theoretical limit,  $t_{total}^*$ , for the coarse grid ( $t_{total}$  is 76min compared to  $t_{total}^*$  which is 95min), 96% for the medium grid ( $t_{total}$  of 91.2min), and 99.6% for the fine grid ( $t_{total}$  of 94.6min). This is one reason for the earlier and lower peak bay temperature seen for the coarse grid. As an additional check on the impact of this fraction of the theoretical maximum burning duration, one local burning time was added to the coarse grid case (spread evenly amongst the three far field components of the gas phase temperature-time curve), bringing  $t_{total}$  to 95min and equal to  $t_{total}^*$ . The peak bay temperature from this check was 477°C (7.5% lower than that from the finest grid) at 100min (4.2% later), instead of the previous 451°C peak and 15min time difference. Thus, the impact of the temporal delay introduced by coarse grids can be easily quantified.

Coarse grids that are on the same order of length as a structural bay could also affect the bay temperatures. This is explored in Section 5.5.4.

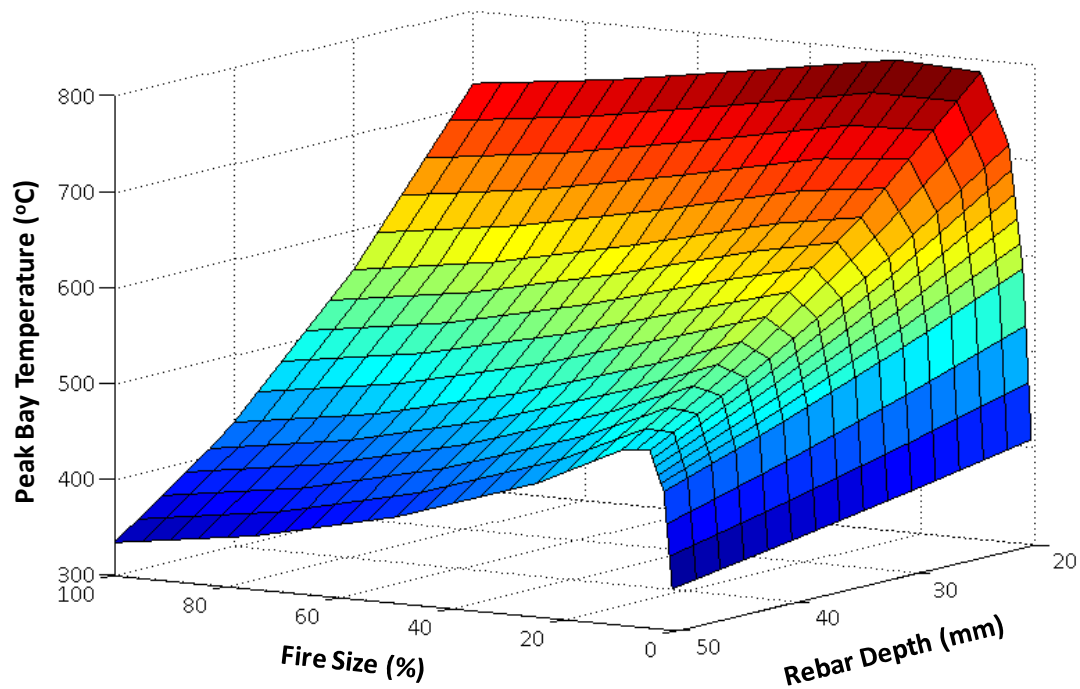
### 5.5.3 Rebar Depth

The depth of rebar is a fundamental design variable for any concrete structure. Typical rebar depths are between 20 and 60mm. A structural engineer would usually establish the rebar depth of a structure before its fire performance is analysed in detail. However, it is worth understanding the impact of rebar depth on peak bay temperatures, as it could make a significant difference in the design and, subsequently, the performance and cost of the structure.

Figure 5.9a shows the gas phase and resulting bay temperature vs. time for various rebar depths for the base case. Figure 5.9b shows the peak bay rebar temperature for varying rebar depth and fire size, for a grid size of 0.21m. The results show the logical result that the shallower the rebar, the higher its temperature.



(a)



(b)

**Figure 5.9:** (a) Gas phase and bay temperatures for rebar depths of 20, 30, 42 and 50mm; (b) Peak bay temperature vs. fire area and rebar depth for  $\Delta x = 0.21\text{m}$ .

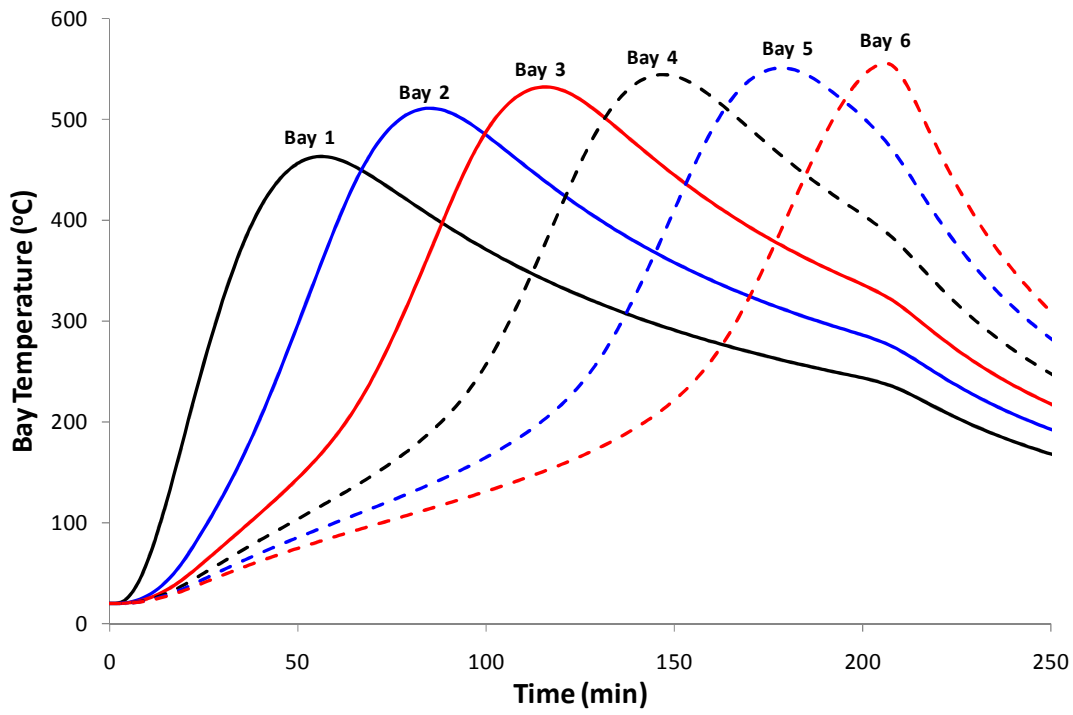
The 10% fire size results in the maximum peak bay temperature for all rebar depths except the 50mm depth, which has its maximum at the 5% fire size. This is due to the increased importance of the pre-heating and post-heating of the rebar from the far field, which is longer for smaller fires. A rebar depth of 42mm is used for the base case as this was the design value for the similar structure in [4].

#### **5.5.4 Bay Location and Bay Size**

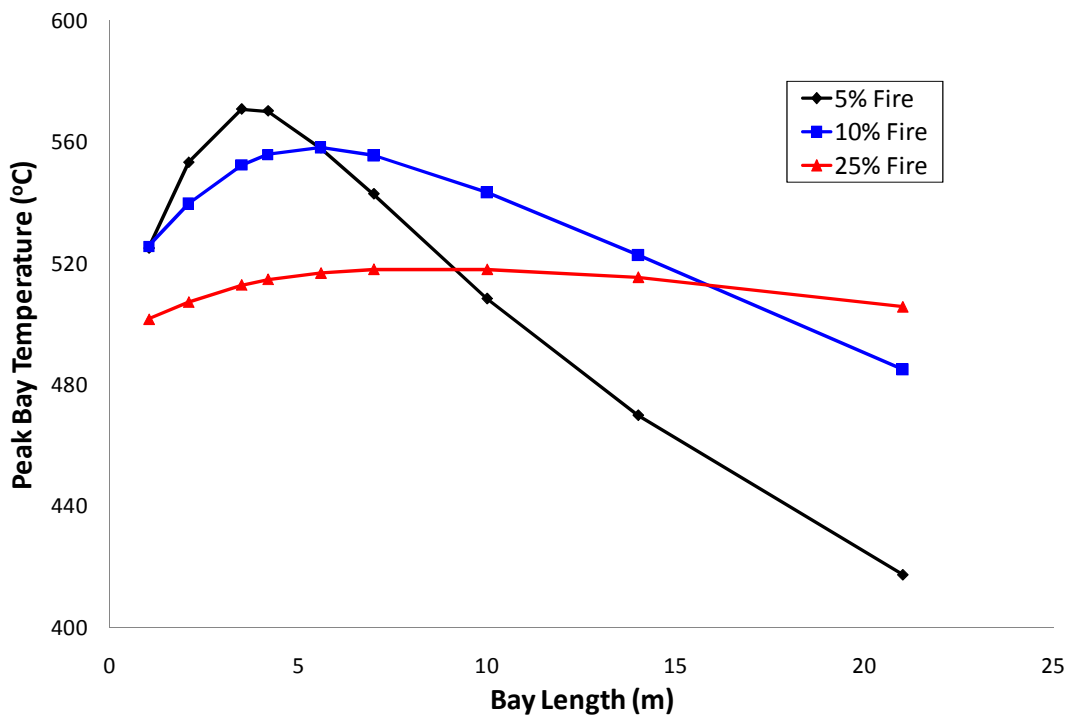
As discussed above, the bay temperature is a critical parameter for structural response. Figure 5.10 shows the sensitivities of the bay location and bay size. Figure 5.10a gives the temperature-time curves for each bay in the compartment (see Figure 5.5 for bay numbering). Figure 5.10b gives the peak bay temperature as a function bay length for three fire sizes (5%, 10%, and 25%). The fire begins in Bay 1 and travels across the structure, eventually ending in Bay 6.

Figure 5.10a shows that the peak bay temperature increases with distance from the ignition location. This is because the peak temperatures are always caused by exposure to the near field, but are also dependent on the bay temperature at the time of near field arrival. The bay temperature at the time the fire arrives is dependent on the exposure duration and temperatures of the far field. As each subsequent bay along the structure is exposed to longer pre-heating times prior to the arrival of the near field, the hottest peak bay temperature is found in the final bay (Bay 6).

This conclusion can be generalised, stating that the peak rebar temperature in a structure will occur at the final burning location of the fire. This is a significant result, as it means that the exact travel path of a fire does not need to be known if the peak rebar temperature is the variable of interest for the structural analysis. This is beneficial for design, as the path cannot be known a priori as there are many possible paths of fire travel depending on ignition location, early fire development and subsequent glazing failure.



(a)



(b)

Figure 5.10: (a) Bay temperatures vs. time for each bay in the structure along its length; (b) Variation of peak bay temperature with bay length for 5%, 10% and 25% fire sizes.

Thus for design, if the structural engineer can identify particular areas of the structure that are most vulnerable to the effects of elevated rebar temperature, then it can be conservatively assumed that the fire reaches this location last, thereby producing the most onerous fire environment for that part of the structure. Note that other structural variables are important in travelling fires (see [4]) and that the role played by the heating and cooling phases, for example, are not directly captured by the peak bay temperature alone.

Figure 5.10b shows the impact of bay size on bay temperature. The bay size was varied from 1.05m (the smallest possible bay size for the base case grid size, as there is only a single node per bay) to 21m (half the length of the structure, which is deemed to be beyond a realistic upper bound). The results indicate that the larger the fire, the less impact the bay size has on the peak bay temperature. This is due to the ratio between fire size and bay size. For bay sizes that are smaller than the fire size, the full bay is exposed to the near field at once. Given that much of the range in bay size variation is less than the fire size for the 25% case (the largest fire examined here, with  $L_f = 10.5\text{m}$ ), little impact on peak temperatures is expected from variation of bay size. However, for the smaller fire sizes, many of the bay lengths examined are greater than the fire lengths (2.1m for the 5% fire and 4.2m for the 10%). Therefore impact of bay size is to be expected in these cases.

The results also show that the maximum peak bay temperatures occur nearly, but not exactly, when the bay size is equal to the fire size. This is due to the balance of higher far field temperatures prior to the fire arriving and lower far field temperatures after the fire passes. There is a small effect of the grid size on the peak value, but as the temperature differences are small (on the order of  $10^\circ\text{C}$ ) it is not deemed significant.

### 5.5.5 Fuel Load Density and Heat Release Rate per Unit Area

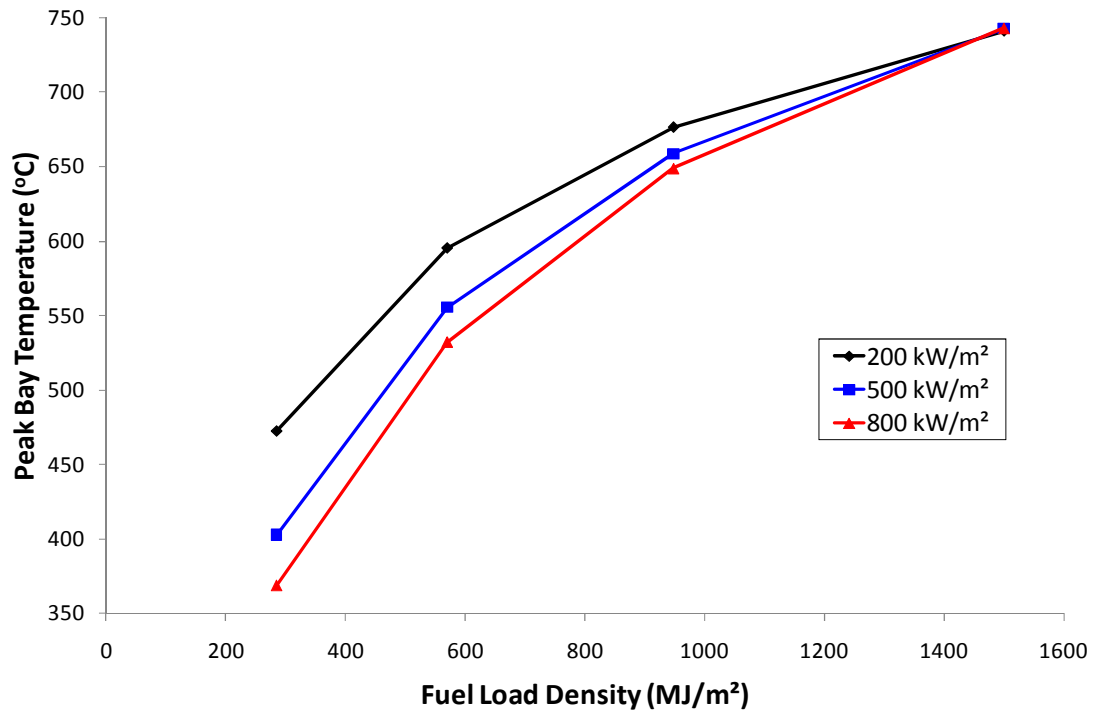
Eq. (5.2) gives the local burning time as a function of the fuel load density and heat release rate per unit area. The local burning time, in turn, affects the total burning duration. The higher the fuel load, the longer the local burning time and, thus, the longer the total burning duration. The heat release rate per unit area also impacts the burning times. The higher the heat release rate per unit area, the shorter the local burning time and total burning duration of a fire. However, the heat release rate per unit area also has an impact on the total heat release rate for a given fire size and, therefore, the far field temperatures. This means that as it reduces the total fire duration, it also increases the gas phase temperatures to which the structure is exposed.

The amount of fuel in a building significantly alters the dynamics of a fire. The fuel load varies greatly for building types and guidance exists to provide typical ranges [2]. The base case fuel load was taken as the 80<sup>th</sup> percentile value for office buildings [19]. The range of values for the sensitivity study varies from sparsely furnished (classroom) to densely loaded (library) spaces according to [2]. The heat release rate per unit area is a fundamental characteristic of a fire. The range selected here corresponds to that measured for a variety of fuels that could be expected in a typical office building [31], but excludes very high values that might be associated with rack storage or other industrial usages. The base case value is taken from [20] and is the same used in earlier work [4].

Figure 5.11 shows the variation of peak bay rebar temperature with fuel load density for heat release rates per unit area of 200, 500, and 800kW/m<sup>2</sup>.

Denser fuel loads result in higher peak bay rebar temperatures. The opposite trend is observed for the heat release rate per unit area, i.e. the lower the heat release rate per unit area, the higher the peak bay rebar temperatures. Both of these trends can be explained by the increase in time that results from an increase in fuel load or

decrease of the heat release rate per unit area. While the total heat release rate increases for a higher heat release rate per unit area, these results suggest that the effect of the reduction in fire duration is more important than the effect of the far field temperature on the structural heating. This is due to the linear relationship between heat release rate per unit area and time and the 2/3 power relationship between heat release rate and far field temperature.



**Figure 5.11:** Peak bay temperature vs. fuel load density for a range of heat release rates per unit area.

### 5.5.6 Heat Transfer

Because it is difficult to quantify specific values of the overall heat transfer coefficient and emissivity in a fire, the sensitivity of these parameters has been examined here. The convective heat transfer coefficient of the exposed side of the concrete slab was varied from 10 to 100W/m<sup>2</sup>K to represent the bounds typically expected in a compartment fire [48]. The material emissivity was varied from 0.2 to 1. For typical concrete reradiation at high temperatures, the effective emissivity is likely to be high, but 0.2 has been examined as a lower bound. The gases are

assumed to have an emissivity equal to 1, and the material absorptivity is assumed to be equal to the emissivity. The base case values of both heat transfer parameters were taken according to Eurocode 1 guidance [2].

Figure 5.12 plots peak bay temperature against the convective heat transfer coefficient for varying values of emissivity and two rebar depths. A shallow rebar depth (20mm) was examined, in addition to the base case value, to include a scenario of reduced importance of the conductive heat transfer.

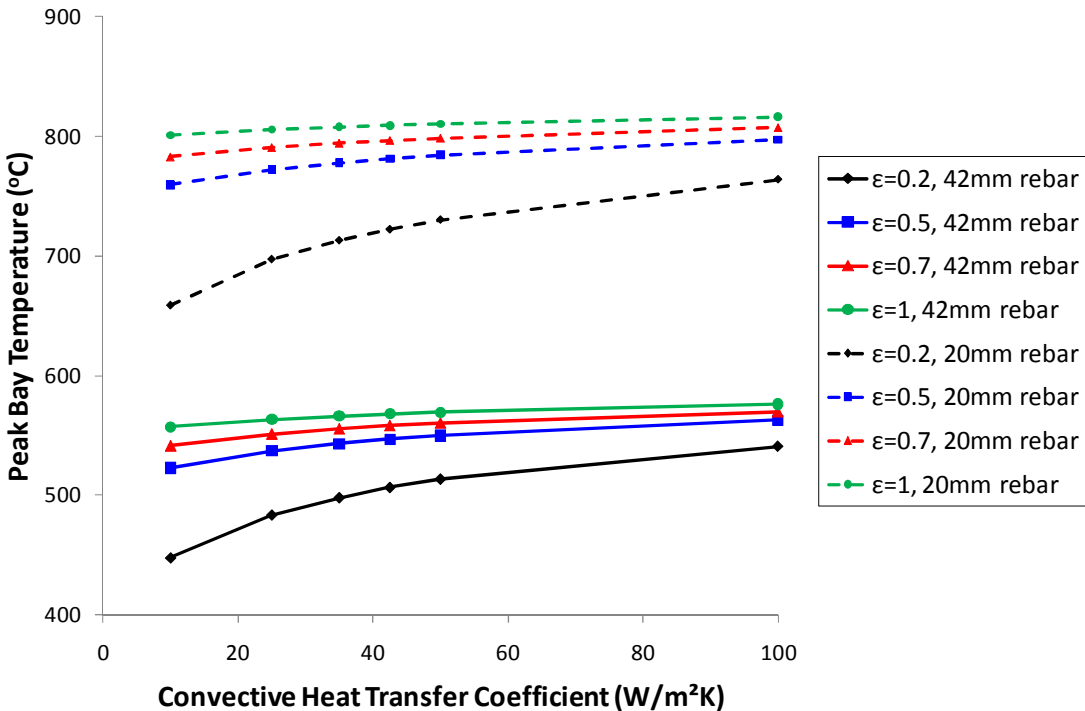


Figure 5.12: Peak bay temperature vs. convective heat transfer coefficient for a range of material emissivities and rebar depths.

The results indicate that the peak bay temperatures are only marginally affected by the heat transfer parameters at either of the two rebar depths studied. The lower temperatures that result from the lower emissivities indicate that concrete heating is dominated by radiation in the base case.

### 5.5.7 Near Field Temperature

For the sake of conservatism, the methodology assumes that the near field temperature is the peak flame temperature measured in large fires. The sensitivity of bay temperatures to this assumption is studied here. Peak temperatures in small fires have been measured in the range of 800 to 1000°C [23], while those in larger compartments have been found to be up to approximately 1200°C [24]. The FDS simulations of a localised 147MW fire in a large compartment shown in Figure 5.3 agree with this range and predict peak near field temperatures ranging from 800 to 1050°C, depending on the ventilation scenario. Therefore the near field temperature has been varied from 800 to 1200°C, with the base case value at the upper end of the range to account for worst case conditions and overcome the uncertainty associated with the prediction and measurement of flame temperatures. Figure 5.13 shows the bay temperature evolution over time for varying near field temperatures at Bays 2 and 6.

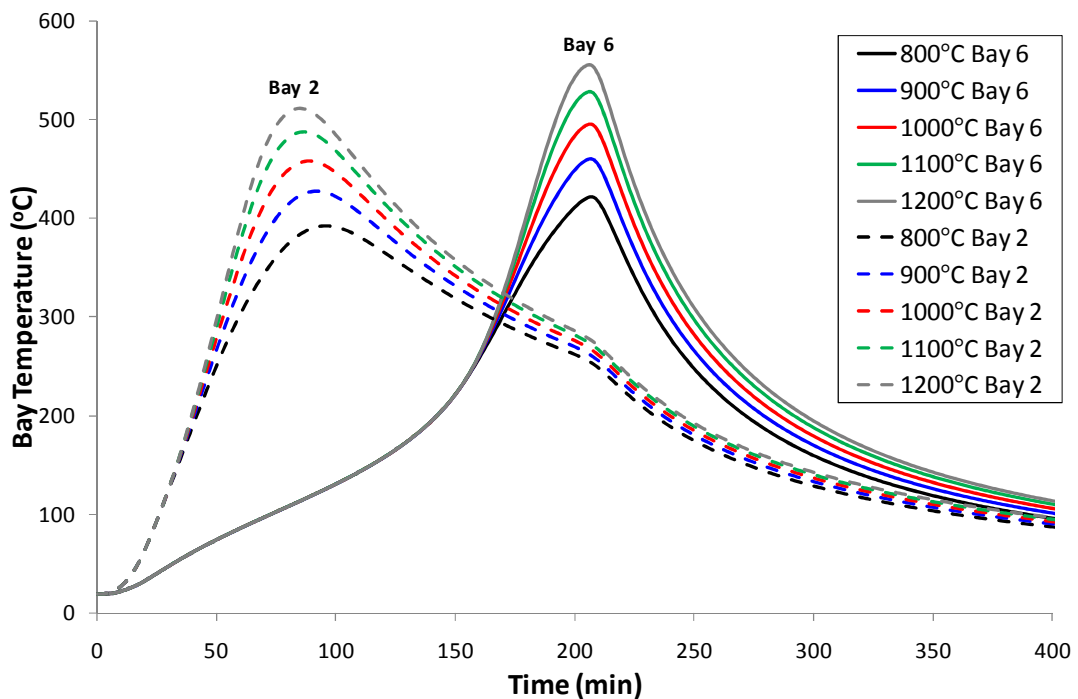


Figure 5.13: Bay temperature vs. time for near field temperatures between 800 and 1200°C at Bays 2 and 6.

The results show that a near field temperature variation of 400°C (from 800 to 1200°C) produces a peak bay temperature range of approximately 130°C. The results are similar for both bays. The near field temperature assumed has no impact on the structural heating in the far field region, but does have an important overall effect on the predicted fire resistance of the structure. However, given that the design value is taken at the upper end of the physical range, it means results from this methodology can be deemed conservative.

### 5.5.8 Steel Structure

In addition to the base case concrete structure, the heating of a typical steel beam is also examined. The steel beam studied was selected to be representative of typical section sizes used in real buildings. Dimensions of the beam are given in Figure 5.14. The beam has been assessed with three levels of fire protection: unprotected, fire rated to 60min, and fire rated to 120min. For quantification of its heating, it is assumed that there is a slab above the top flange of the beam and thus it is only heated on three sides.

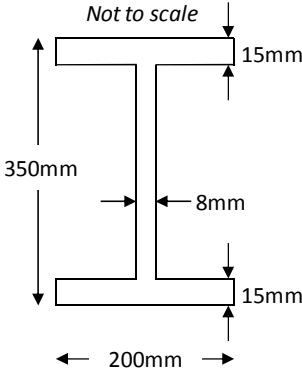


Figure 5.14: Dimensions of the steel beam section analysed.

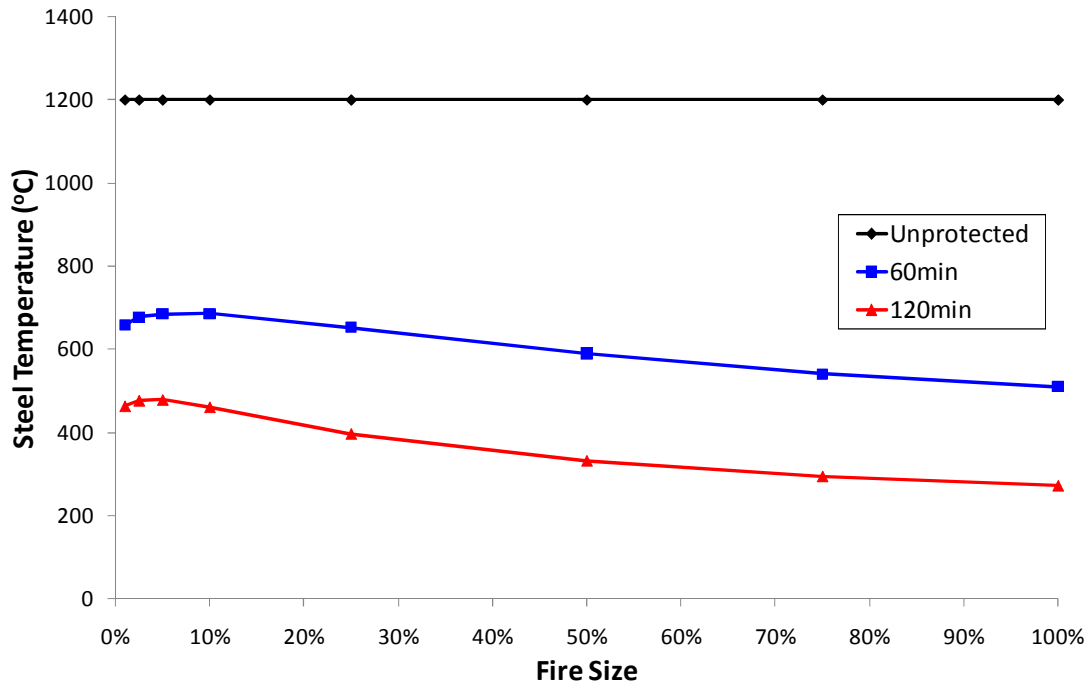
The heat transfer to the beam was calculated utilising a lumped mass approach and is given in Appendix A. For the purposes of this analysis, it is assumed that the steel beam is perpendicular to the direction of fire propagation and thus is exposed to the same gas temperature along its full length at any given time. This is done because

using a single beam temperature as a surrogate for structural response is not valid for a beam exposed to a varying temperature along its length. This methodology could also be used for calculation of the distribution of steel temperatures along the beam axis as the rate of conductive heat transfer in steel is much lower than the spread rate of a travelling fire. However determining the structural response of that scenario would require the adoption of a two or three-dimensional structural analysis method. Nonetheless, the single point heat transfer calculations used here provide insight into the differences in heating of the three types of steel beam, as compared to the concrete slab.

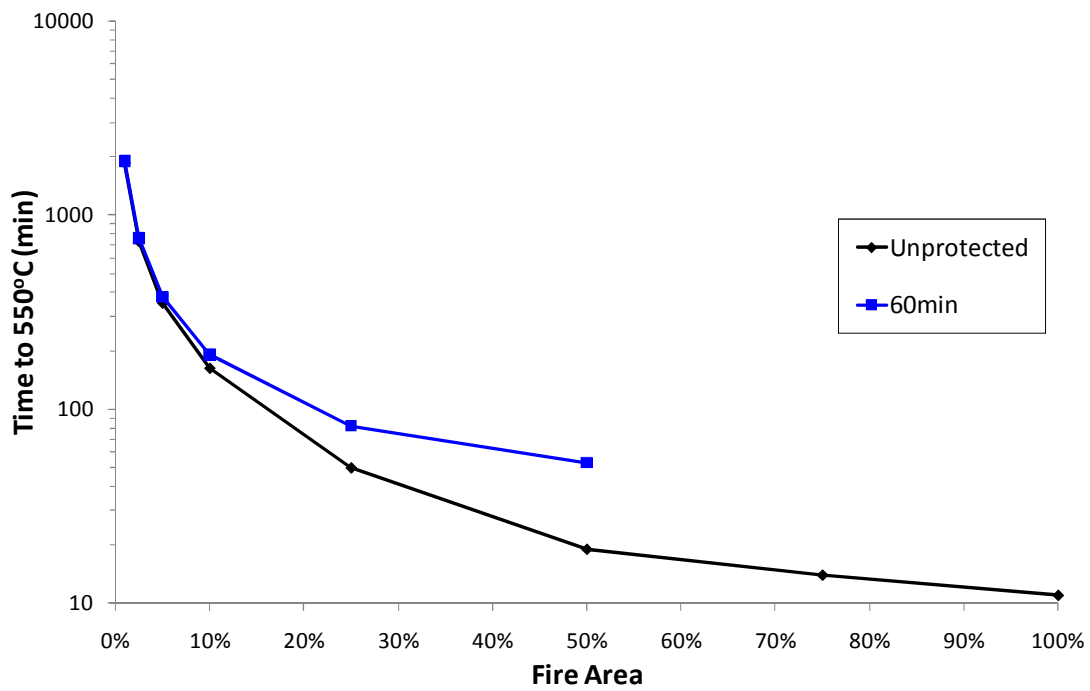
Figure 5.15a shows the resultant peak steel temperatures for the three beam types at the far end of the final structural bay (Bay 6) for a grid size of 0.21m. The fine grid resolution was used to best match the node size to the physical size of the steel beam.

It can be seen that the steel temperatures of the unprotected beam reach the near field temperature for all fire sizes. This is due to the low thermal inertia and high conductivity of the unprotected steel. The protected beam temperatures follow a similar trend to that of the concrete structure. The maximum temperature recorded for the 60min rated beam is from a 10% fire size and for the 120min beam from a 5% fire size.

Figure 5.15b gives the time for the unprotected and 60min rated beams to reach 550°C (the 120min rated beams do not reach this temperature and are therefore not shown). A critical value of 550°C was selected here as this is normally considered an approximate temperature above which steel loses sufficient strength such that failure of a typical simply-supported beam could occur under the loads assumed to be applied during a fire [24]. As with the concrete bay temperatures, it shows that the larger fire sizes reach the specified temperature more quickly than the smaller ones, even though they ultimately do not reach the same peak temperature.



(a)

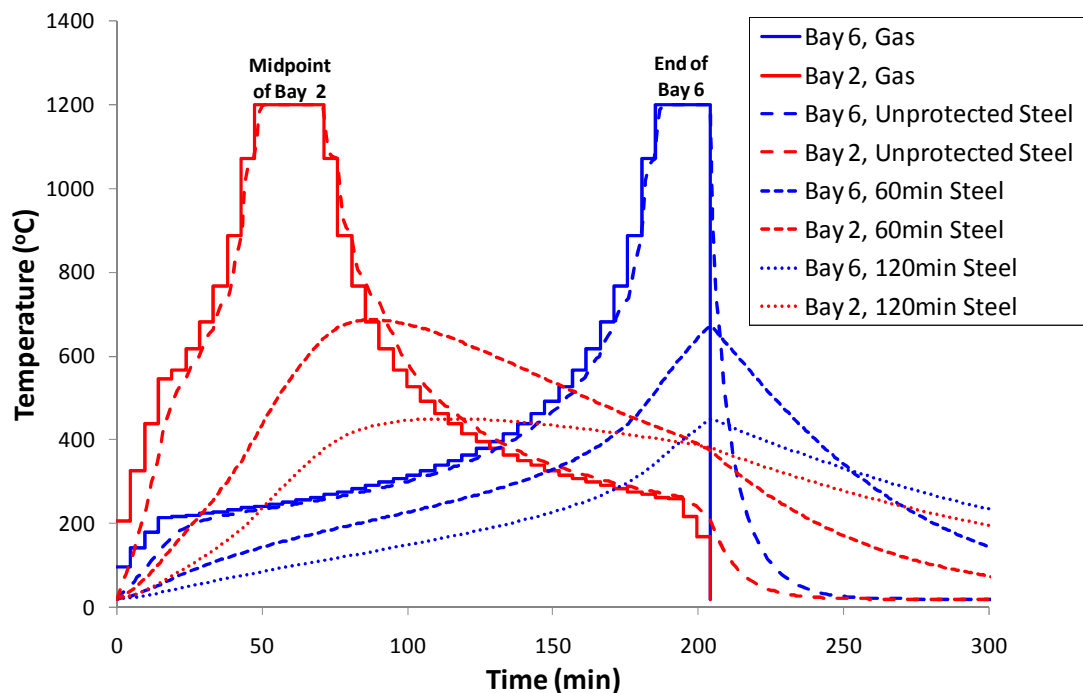


(b)

**Figure 5.15:** (a) Peak steel temperature vs. fire size for unprotected, 60min rated, and 120min rated steel beams at the far end of Bay 6 for a grid size of  $\Delta x = 0.21\text{m}$ ; (b) Time for the unprotected and 60min rated steel beams to reach  $550^\circ\text{C}$  on log scale for time.

The unprotected beam reaches 550°C between 29min (for the 1% fire) and 34min (for the 50% fire) faster than the 60min rated beam. Fire sizes above 50% do not reach 550°C for the 60min rated beam. Note, however, as with the concrete rebar heating, the time for the steel beam to reach a specified temperature for a travelling fire is dependent on the location of the bay relative to the fire's path, i.e. the time of near field arrival relative to the total burning duration.

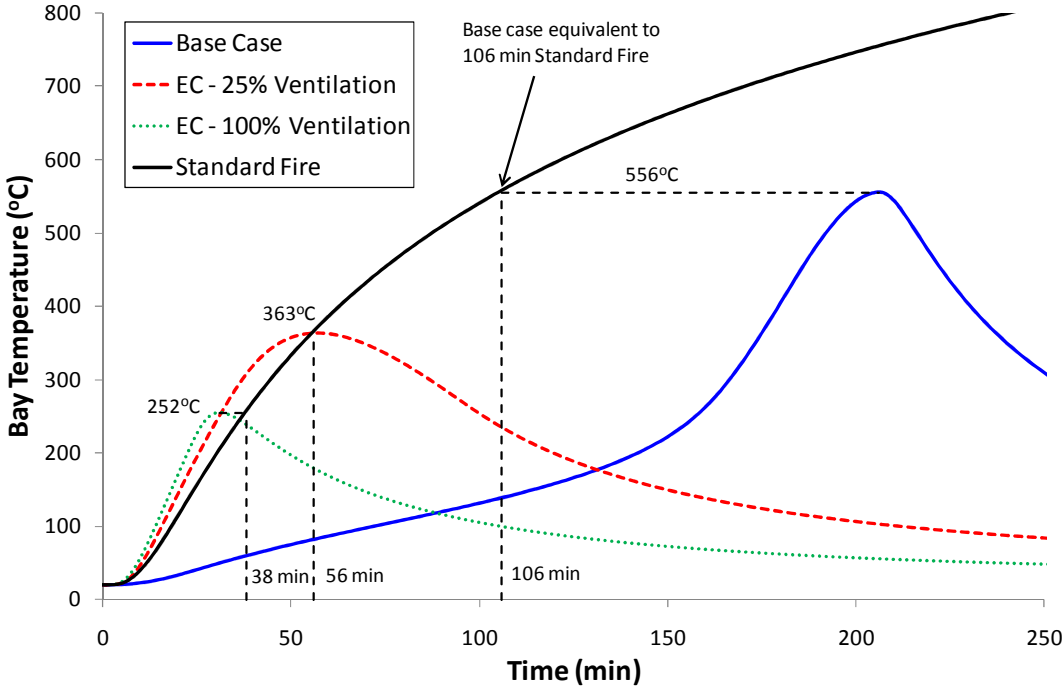
Figure 5.16 shows temperature-time curves for the gas phase and steel for all three beam types considered at two different locations in the structure. The unprotected steel temperature follows the gas phase temperature very closely, for the reasons given above. The peak steel temperatures are very similar for both locations, with a slightly higher peak reached for the midpoint of Bay 2 for the 60min rated beam. This lack of sensitivity to steel location is different from that observed in concrete (see Figure 5.10a)



**Figure 5.16:** Temperature vs. time for the gas phase plus all three steel beam types at the midpoint of Bay 2 and the far end of Bay 6.

## 5.6 Comparison to Conventional Methods

Figure 5.17 compares the bay temperature-time curves resulting from the base case fire scenario with those calculated from the standard fire and two Eurocode parametric temperature-time curves [2]. One parametric temperature-time curve assumes 100% glass breakage on the façade and the other 25%. The parametric curves use the same thermal properties of concrete (see Appendix A for values) and fuel load density as the base case.



**Figure 5.17:** Comparison of bay temperatures calculated using the base case, the standard fire, and two Eurocode parametric temperature-time curves.

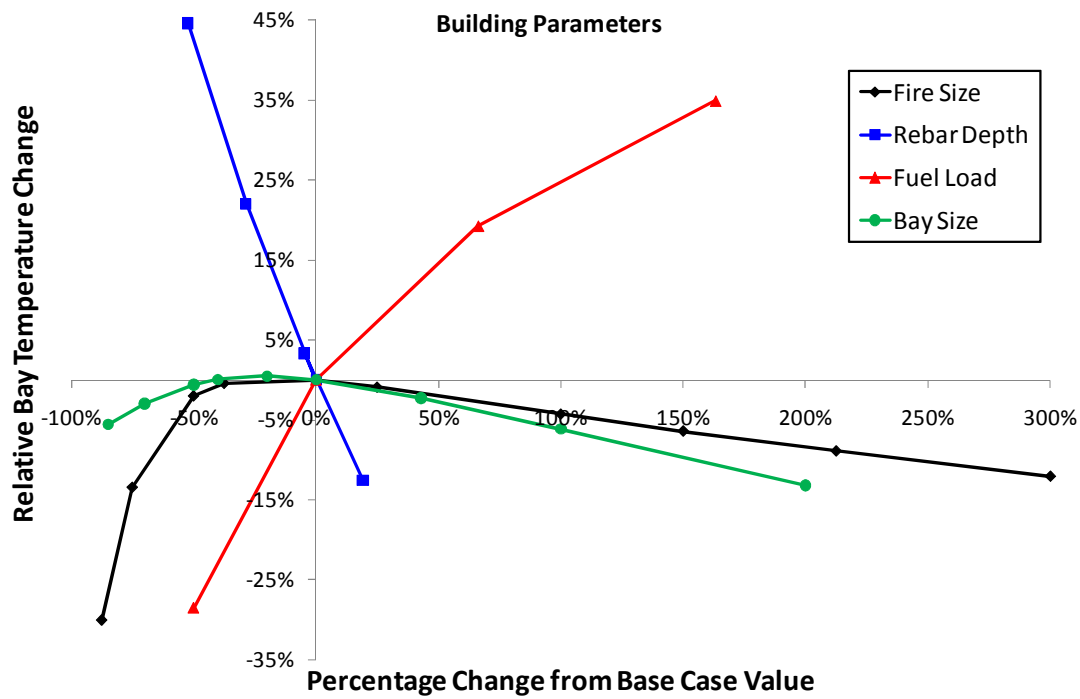
The comparison shows that the base case, which is the most onerous fire size in the family of fires, is a more challenging scenario for the structure in terms of peak bay temperature reached than the two parametric curves. In terms of the peak bay temperature, the travelling fire is equivalent to 106min of the standard fire, which is similar to the conclusions of Law et al. [4]. This is compared to the two parametric curves, which are equivalent to 38min and 56min of the standard fire.

The results presented here should be explored in more detail by a structural engineer, as a travelling fire may lead to different structural behaviour than that indicated by examining the peak bay temperature alone [4]. For example, whole frame behaviour resulting from exposure to a travelling fire with portions of the structure being heated while other areas are cooling may be different than that suggested by the bay average results given here.

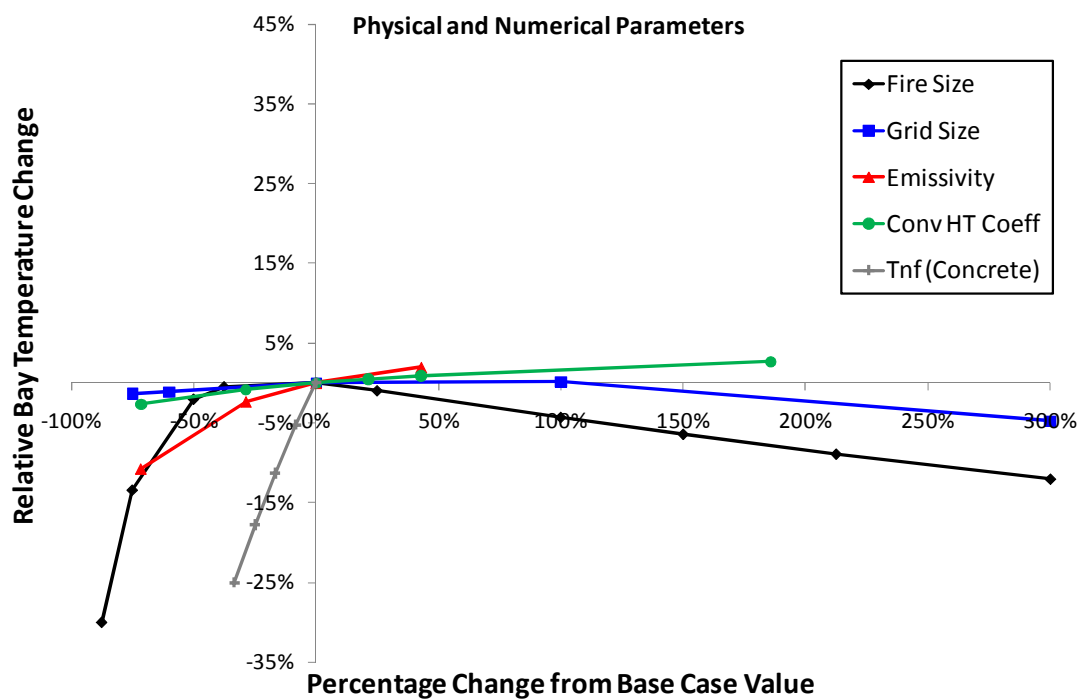
## 5.7 Final Remarks

Comparisons of the relative impact of all the parameters varied in the methodology are shown in Figure 5.18. The percentage variation of each parameter from the corresponding base case value has been plotted against the resultant percentage change of the peak bay temperature calculated. Figure 5.18a shows the results for the building parameters, and Figure 5.18b the physical and numerical parameters. Fire size has been shown on both plots as it is the main variable in this methodology.

Steeper slopes on the curves in Figure 5.18 correspond to the more sensitive parameters. Positive values in the bay temperature change mean conditions are more onerous on the structure than the base case and negative values less onerous. The largest changes in bay temperature come from rebar depth, fuel load density, fire size, and near field temperature, in this order. These are the most sensitive parameters.



(a)



(b)

Figure 5.18: Relative change in bay temperature vs. percentage change in (a) building parameters and; (b) physical and numerical parameters.

The rebar depth, the most sensitive parameter, is likely to be a fixed value early in the design, but its sensitivity is worth noting for the design process of a building. The exact fuel load density cannot be known exactly, as it is inherently variable and may change over the lifetime of a building. Therefore a reasonable assessment of the likely values should be made during design. It is noted that both of these parameters would be used by many forms of structural fire assessment, whether that be the travelling fires methodology presented in this paper or the conventional methods.

Fire size is the main variable of this methodology, so the full range should always be explored in a design case. While the near field temperature has a marked impact on the bay temperatures, it is not necessary to vary this parameter for design, as the methodology assumes the most onerous condition.

The methodology presented in this paper offers a paradigm shift in defining fire scenarios for structural fire engineering and compliments the traditional methods. This paper has explored the details of the method and concluded on the more sensitive parameters that ought to be considered in design. The methodology provides a robust platform for collaboration between fire engineers and structural fire engineers to jointly understand a building's structural performance in fire.

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# 6

## Conclusions and Future Work

### 6.1 Conclusions

Most fire tests that have been used in the development of traditional tools to define the thermal environment for structural fire design had floor areas on the order of only a few square metres. Larger tests, which are far fewer in number, extend to floor areas on the order of one hundred square metres. Given that floor to ceiling heights of real buildings are typically only a few metres regardless of floor area, the relative impact of a compartment's walls on the heat transfer with hot fire gases is greatly reduced in larger compartments. In addition it is well known that radiation, which governs flame spread in compartment fires, does not scale well. Therefore applying data that are extrapolated from small compartment tests to large buildings is inappropriate and has led to design methods that do not replicate real fire behaviour at that scale.

One common conclusion from these small scale tests that is manifest in the traditional structural fire design methods is that of a uniform gas phase temperature. This homogeneous temperature assumption has been reviewed in Chapter 2. It is shown that this assumption does not hold well. The temperature heterogeneity that actually exists in real compartment fires does have an impact on the heating of a structure.

However, conducting fire tests in enclosures of a size of interest for modern building design (floor plates on the order of thousands of metres squared) is difficult and costly. Therefore practical solutions to characterising fire behaviour in large enclosures are needed. To this end, this thesis has developed a methodology to characterise travelling fires for structural design. The approach used is a dramatic departure from the traditional methods and suggests a paradigm shift in structural fire engineering. It has been shown that this methodology both addresses limitations in the existing methods and enables innovation in design by providing a more realistic characterisation of the fire environment of a building.

The methodology, which is presented in Chapters 3, 4 and 5, addresses the lack of large scale test data by examining a full range of fire sizes rather than trying to calculate one. This fits well with the uncertainty associated in fire growth and development in large, real buildings. The family of fires approach ensures that the most challenging, physically possible fire scenario is considered for structural design.

Each member of the family of fires produces far field temperatures and burning durations related to its size. Large fires have high far field temperatures but short durations, while small fires have low far field temperatures and long durations. The application of the travelling fire methodology has shown that medium sized fires (10% to 25% of the floor area) prove most challenging to a generic concrete

structure. These fire sizes have an optimum balance of elevated far field temperatures and total fire duration.

The sensitivity studies conducted in Chapters 4 and 5 give insight into the effect of varying the methodology's input parameters. It has been shown that the most sensitive parameters are related to the building design and use and not the theoretical or numerical formulation of the approach. This enables practical application of the methodology in design without the need for cumbersome sensitivity studies of every parameter.

Accurately calculating the behaviour of a structure exposed to fire is necessary for performance based design. However, quantifying the thermal environment created by a fire, the resultant heating of the building elements, and the subsequent structural response requires a broad set of skills from disparate engineering disciplines. Therefore it has been recognised that no one individual should do this alone and fire engineers should work with structural fire engineers to jointly solve this problem [1, 2].

This is the spirit in which the methodology presented in this thesis has been developed. Chapter 4 provides a good example of collaborative work between fire and structural fire engineers. However, further collaboration is needed to better understand the impact of travelling fires on structural performance. The travelling fires methodology provides a robust platform for this collaborative research.

## **6.2 Future Work**

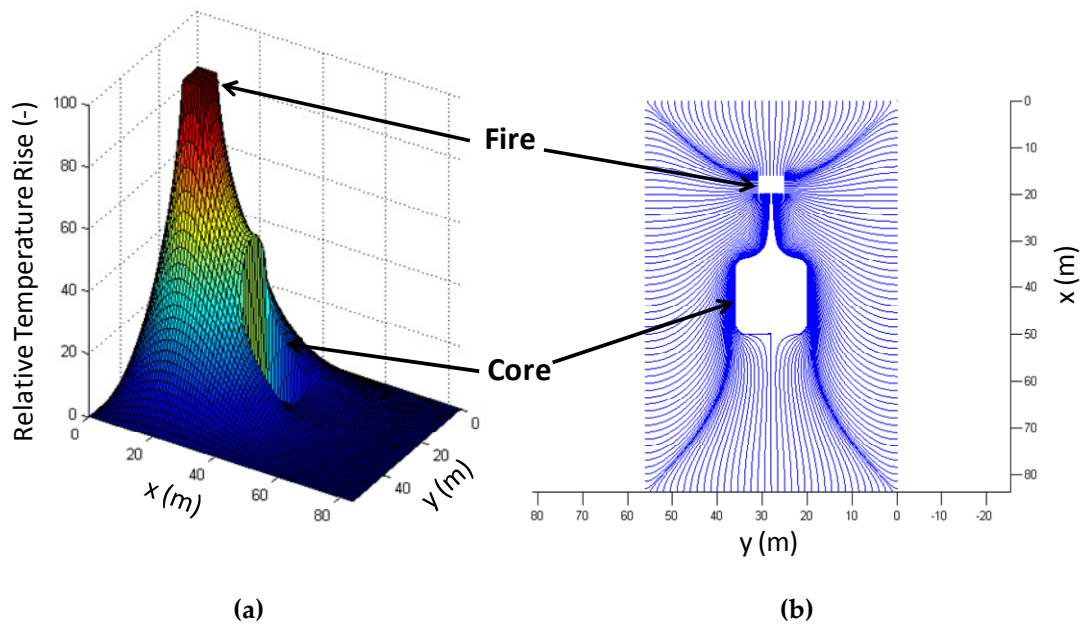
While the travelling fires methodology developed in this thesis provides a practical tool that can be used in design, as shown by the case studies and sensitivity analyses in Chapters 4 and 5, further research would serve to improve it and make it more robust.

### 6.2.1 Fire Environment

Often large scale tests in structural fire research aim to create uniform fire conditions by providing multiple ignition points throughout evenly distributed fuel packages. These tests also tend to be sparsely instrumented to collect data in the gas phase. Conducting large scale tests in real buildings could greatly serve to increase understanding of the dynamics of travelling fires. These tests should allow the fire to grow and travel on its own and be well instrumented so the far field temperatures and fire movement could be recorded.

Far field temperatures in this thesis have been calculated by means of an empirical ceiling jet correlation. It has been noted that the use of this correlation is a simple solution to a complex problem. The correlation provides temperatures as a function of distance from the fire, but can only be applied in limited geometries. Computational Fluid Dynamics (CFD) was used in the earliest version of the travelling fires methodology [3] and is more readily applicable to complex geometry; however it has drawbacks related to complexity and computational effort. The lack of large scale test data means it is difficult to validate both the simplistic correlations and CFD approaches taken. Work to develop better engineering tools to calculate the far field temperature would benefit the methodology. Such tools should consider the open nature of large, modern compartments and not automatically assume the fires are ventilation limited [4].

One potential compromise between the correlation and CFD could be to solve the Laplace Equation ( $\nabla^2\phi = 0$ ). The solution to this transport equation could be viewed as an analogue of smoke movement. This approach, which is illustrated in Figure 5.11 for a generic structure similar to those used in Chapters 4 and 5, could explore more complex geometry than the ceiling jet correlation, but in much less calculation time than CFD. A Dirichlet boundary condition could represent flow out of an open window and a Neumann boundary condition flow next to a solid wall.



**Figure 6.1:** Illustration of a solution of the Laplace Equation in a generic concrete structure showing (a) 3D contours and; (b) streamlines in plan view.

The travelling fires methodology has, to date, only focused on horizontally travelling fires. However, as noted in Chapter 3, accidental fires do travel vertically as well. The sole study on the structural impact of vertically travelling fires [5], only considers uniform fires on each floor. The methodology presented in this thesis could be applied to vertically travelling fires as well. The time delay of spread from one floor to the next would need to be parametrically varied. Care would need to be taken to combine a range of such time delays with various fire sizes to ensure the most onerous design case is identified.

### 6.2.2 Fire – Structure Interface

The traditional methods used in defining the thermal environment for structural fire engineering specify gas phase temperature-time curves. The subsequent heat transfer calculations utilise these curves to calculate the structural temperature evolution. It was actively decided to produce gas phase temperature-time curves as the output of the travelling fires methodology to conform to the existing heat transfer methods used by the structural fire community. However, it is noted that

the actual heating of a structure results from the net heat flux to it, rather than solely the exposure temperature. Calculation of the heat flux from a fire to a structural element is a complex process and requires knowledge of smoke conditions such as velocity, soot content, and layer depth. Methods have been developed to examine this [6, 7], but this concept should be explored in relation to travelling fires.

### **6.2.3 Structural Response**

The travelling fires methodology provides a powerful tool for structural fire research. In collaboration with fire engineers, this methodology enables structural engineers to examine more realistic structural response to fire than the traditional methods. Construction types other than the concrete frame examined in Chapters 4 and 5 should be examined with this approach. Composite steel–concrete construction is of particular interest due to its prevalence in the built environment.

A central concept of travelling fires is the non-uniform temperature field. The impact of this on the full frame behaviour of structures should be explored. Additionally, some structures may be more vulnerable to severe near field conditions than others, such as post-tensioned concrete slabs. The traditional methods do not generally consider local near field conditions, so this should be researched.

Small travelling fires also result in total fire durations much longer than those calculated by the traditional methods. Thus a structure could be exposed to far field temperatures of several hundred degrees Celsius for many hours. This could have a significant impact on creep in steel structures and spalling in concrete frames.

Most importantly, however, the travelling fires methodology provides a framework that allows fire engineers and structural fire engineers to jointly determine a building's true response in fire, thereby enabling architectural innovation and structural optimisation.

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# Appendix



# A

## Heat Transfer Calculations

This appendix provides the details of the simplified heat transfer calculations used to quantify the rebar and steel temperatures used in Chapters 2 and 5 of this thesis.

### A.1 Concrete Temperature

To determine the in-depth temperature of the concrete, a one-dimensional finite-difference approach to the heat conduction equation was taken in explicit form, as given by Incropera et al. [1]. It is assumed that the rebar of the concrete is the same temperature as the adjacent concrete.

The formulation from Incropera et al. only includes surface convection, so a radiative term was added for the surface nodes. This gives Eq. (A.1) for calculating the exposed surface node temperature, and Eq. (A.2) for the interior nodes, and Eq. (A.3) for the backside surface node. Chapter 2 did not use Eq. (A.3), but rather

assumed a sufficient depth of slab above the concrete beam such that the boundary condition did not influence the results.

$$T_0^{t+1} = \frac{2\Delta t}{\rho_c c_c \Delta Z} \left[ h_0 (T_g - T_0^t) + \sigma \varepsilon (T_g^4 - T_0^{t4}) + \frac{k_c}{\Delta Z} (T_1^t - T_0^t) \right] + T_0^t \quad (\text{A.1})$$

$$T_i^{t+1} = Fo (T_{i+1}^t + T_{i-1}^t) + (1 - 2Fo) T_i^t \quad (\text{A.2})$$

$$T_n^{t+1} = \frac{2\Delta t}{\rho_c c_c \Delta Z} \left[ h_n (T_\infty - T_n^t) + \sigma \varepsilon (T_\infty^4 - T_n^{t4}) + \frac{k_c}{\Delta Z} (T_{n-1}^t - T_n^t) \right] + T_n^t \quad (\text{A.3})$$

where  $T_i^t$  is the concrete temperature at time  $t$ , and location  $i$  (K) – a subscript of 0 indicates the exposed surface and a subscript of  $n$  the backside surface.

$T_g$  is the gas temperature (K)

$T_\infty$  is the ambient temperature (293K)

$\rho_c$  is the density of concrete (2300kg/m<sup>3</sup>)

$c_c$  is the specific heat of concrete (1000J/kgK)

$h$  is the convective heat transfer coefficient (25W/m<sup>2</sup>K for exposed surface in Chapter 2, 35W/m<sup>2</sup>K for the exposed surface and 4W/m<sup>2</sup>K for the backside surface in Chapter 5 [2])

$\sigma$  is the Stefan-Boltzmann constant (5.67x10<sup>-8</sup>W/m<sup>2</sup>K<sup>4</sup>)

$\varepsilon$  is the radiative and reradiative emissivity of the material and gas combined (assumed to be unity in Chapter 2, varied in Chapter 5)

$k_c$  is the thermal conductivity of concrete (1.3W/mK)

$\Delta t$  is the time step (0.5s in Chapter 2, 10s in Chapter 5)

$\Delta z$  is the element length (0.001m in Chapter 2, 0.01m in Chapter 5)

$Fo$  is the Fourier number (-), given in Eq. (A.4)

$$Fo = \frac{k_c \Delta t}{\rho_c c_c \Delta Z^2} \quad (\text{A.4})$$

The time step and element length were selected to meet the stability criteria highlighted by Incropera et al. The concrete material properties were taken from Buchanan [3] for calcareous concrete.

## A.2 Unprotected Steel Beam Temperature

The unprotected steel beam temperatures were calculated by a lumped mass heat transfer method, as given by Buchanan [3], and shown below.

$$\Delta T_s = \frac{H_p}{A} \frac{1}{\rho_s c_s} [h_c (T_g - T_s) + \sigma \varepsilon (T_g^4 - T_s^4)] \Delta t \quad (\text{A.5})$$

where  $T_s$  is the steel temperature (K)

$T_g$  is the gas temperature (K)

$H_p$  is the heated perimeter of the beam (1.284m)

$A$  is the cross section of the beam (0.00856m<sup>2</sup>)

$\rho_s$  is the density of steel (7850kg/m<sup>3</sup>)

$c_s$  is the temperature dependent specific heat of steel (J/kgK)

$h_c$  is the convective heat transfer coefficient (25W/m<sup>2</sup>K in Chapter 2, 35W/m<sup>2</sup>K in Chapter 5)

$\sigma$  is the Stefan-Boltzmann constant (5.67×10<sup>-8</sup>W/m<sup>2</sup>K<sup>4</sup>)

$\varepsilon$  is the radiative and reradiative emissivity of the material and gas combined (assumed to be unity in Chapter 2 and 0.7 in Chapter 5)

$\Delta t$  is the time step (1s in Chapter 2, 10s in Chapter 5)

All constants and steel material properties (except the emissivity) are taken from Buchanan, including the temperature dependent specific heat.

## A.3 Protected Steel Beam Temperature

The protected beam temperature calculation was also taken from Buchanan [3] and is given below.

$$\Delta T_s = \frac{H_p}{A} \frac{k_i}{d_i \rho_s c_s} \frac{\rho_s c_s}{[\rho_s c_s + (H_p/A) d_i \rho_i c_i/A]} (T_g - T_s) \Delta t \quad (\text{A.6})$$

where  $k_i$  is the thermal conductivity of the insulation (0.12W/mK)

$d_i$  is the thickness of the insulation (m)

$\rho_i$  is the density of the insulation (550kg/m<sup>3</sup>)

$c_i$  is the specific heat of the insulation (1200J/kgK)

The material properties of the insulation were based on high density perlite, as given by Buchanan. The thickness of the insulation was solved for using Eq. (A.6), applying the standard temperature-time curve and limiting the steel temperature to below 550°C for 60 and 120 minutes. This method should ensure a similar level of performance for any insulating material used to achieve these fire ratings.

## References

- 1 Incropera, F., DeWitt, D., Bergman, T., and Lavine, A., *Fundamentals of Heat and Mass Transfer*, John Wiley & Sons, 2007.
- 2 Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire, European standard EN 1991-1-2, 2002. CEN, Brussels.
- 3 Buchanan, A., *Structural Design for Fire Safety*. John Wiley & Sons, 2002.