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A framework to develop a cohesive structural and fire engineering design approach for buildings

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Preface

This report is prepared as part of an effort to develop a more cohesive design approach for structural and fire engineering practices. It is intended for structural engineers, fire engineers and building designers.

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Abstract

This report develops a framework for a more cohesive design approach to structural engineering and fire engineering for buildings than current ones. It overviews the current design approaches from a perspective of structural engineers and identifies differences and similarities of the two engineering practices. Based on this holistic understanding, it develops a method to incorporate fire conditions into structural design approaches. Various means to develop design fires and the responses of structural elements are also investigated.

Keywords

Fire safety; structure; seismic; holistic design; structural safety

Foreword

The catalyst for the research project that has culminated in the publication of this study report was my attendance at a fire conference in 2009. One of the presentations was about the World Trade Centre investigation, focusing on the WTC7 building. WTC7 is a rare example of an uncontrolled fire being solely responsible for the total collapse of the building. The fire involved just the contents of the building, with no significant external fire or structural contributory factors.

This immediately caused me to think about what I considered (and still do) to be shortcomings in the approach to designing buildings for fire in New Zealand. I tested my thinking with some industry colleagues to help crystallise my thoughts.

The philosophical basis for my perspective on designing buildings for fire is this:

- There is a gap between what is covered by structural engineering and what is covered by fire engineering to the extent that important aspects of design are largely glossed over. This isn't to say that a large number of buildings are unsafe per se, but some buildings have not been adequately designed to withstand the impact of a severe long-duration fire.
- Fire needs to be considered as a form of quasi-structural loading in the same way as any other type of loading in AS/NZS 1170 *Structural design actions*.
- Structural engineers should have primary responsibility for designing buildings to withstand the mechanical actions induced by fire. While the fire engineer would provide quantification of the thermal exposure of the building and its elements, the structural engineer should do the actual structural engineering.
- The current approach to designing buildings for fire is elemental. A fire resistance rating is determined and then concrete cover to reinforcing steel, char depth for timber or protection for steel are provided to achieve the required FRR.
- A new global (whole-of-building) approach to designing buildings for fire is required, in addition to the current elemental approach, very much in the same vein as seismic design. This new global approach would help ensure that secondary effects of fire loading, such as expansion/contraction, restraint and so on are given due consideration at the design stage. These secondary effects caused the WTC7 building to collapse.

A proposal for Building Research Levy funding was subsequently submitted and in due course a BRANZ research project commenced, with this document being the major output from the project.

I need to make some important clarifications about this document. This is not a comprehensive, step-by-step design guide that provides a detailed methodological approach to designing buildings for fire. Rather, this is a set of general guidelines that help the user identify key aspects to consider when designing a building to resist fire-induced loadings, with the bonus of some specific guidance on design fires for structural design.

I trust that you find this document both informative and useful and that it contributes to achieving the outcome of promoting the practice of structural design for fire.

Greg Baker, former BRANZ Fire and Structural Engineering Manager

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1. Introduction

1.1 General

Fire safety in a building is often achieved by a combination of active and passive fire protection systems (Buchanan, 1994). Active fire protection systems, such as fire alarms and smoke detection systems or sprinklers, control the fire or fire effects by some actions taken by a person or an automatic device. Unlike active fire protection systems, passive fire protection systems control the fire effects by:

- building sufficient fire resistance into the building structures to provide safe travel paths for the movement of occupants and firefighters and/or
- providing barriers to control the fire/smoke spread from its origin space either across the building or to the adjacent buildings and/or prevent structural collapse during fires (Buchanan, 2001).

Provision of fire resistance is just one part of the overall fire design strategy, but it is one important part of any building design for fire safety. Fire resistance is built into the structural system by:

- choice of building materials
- dimensions of building components
- compartmentation
- fire protection materials.

These control fire spread and fire effects by providing sufficient fire resistance to maintain compartmentation and/or prevent loss of structural stability within a prescribed time period. This is based on the building's occupancy and fire safety objectives under New Zealand Building Code (NZBC) clause C *Protection from fire*. Materials and construction assemblies that provide fire resistance, measured in terms of fire endurance time, are commonly referred to as fire resistance-rated construction or fire-resistive materials and construction (Corus Construction & Industrial, 2006).

The current practice in designing buildings for fire around the world is mainly limited to elemental approaches (Ellingwood et al., 2007; EN 1993-1-2:2005 *Eurocode 3: Design of steel structures – General rules – Structural fire design*; NZBC clauses C1–C6; Buchanan, 2001). Fire resistance rating (FRR) demands are determined based on the usage of the buildings, then all the structural elements necessary for load carrying or separation are required to have a fire resistance rating not less than the fire resistance rating demands. Fire resistance of an element is usually rated by standard ISO fire test on individual elements under ISO 834-1:1999 *Fire-resistance tests – elements of building construction – Part 1: General requirements*.

1.2 Building design for fires in New Zealand

The current New Zealand building regulation environment is based on a performance-based code, which states how a building is to perform under a wide range of conditions, such as wind, seismic, fire and so on. There is a multi-level format for structural designs to various design actions. At higher levels, the NZBC specifies the overall objectives that buildings, building elements and sitework shall withstand the combinations of loads that they are likely to experience, including gravity loads, earthquake and fire. At lower levels, there is a selection of alternative means (compliance methods) of achieving these goals, such as Acceptable Solutions, Verification Methods or Alternative Solutions.

NZBC clauses C1–C6 are intended to:

- safeguard people from an unacceptable risk of injury or illness caused by fire
- protect other property from damage caused by fire
- facilitate firefighting and rescue operations.

NZBC clauses specify functional requirements and performance criteria for:

- prevention of fire occurrence
- fire affecting areas beyond the fire source
- movement to place of safety
- access and safety for firefighting operations
- structural stability.

In designing buildings for protection from fire, buildings are classified in six risk groups, based on the use of the building or part of the building. Current compliance methods for protection from fire that are deemed to meet the NZBC-specified objectives are Acceptable Solutions C/AS1–C/AS6 and Verification Methods C/VM1 and C/Vm^2.

Acceptable Solutions prescribe the fire safety requirements (both active and passive fire protection systems) for:

- firecells, fire safety systems and fire resistance ratings
- means of escape
- control of internal fire and smoke spread
- control of external fire spread
- firefighting
- prevention of fire occurring.

The current Verification Methods for designing buildings for protection from fire are C/VM1 Verification Method for Solid Fuel Appliances and C/Vm^2 Verification Method: Framework for Fire Safety Design. C/Vm^2 is the relevant Verification Method for designing buildings for fires. C/Vm^2 specifies the rules and parameters for design fire modelling, which are the inputs for evaluating building performance in fires. However, C/Vm^2 has not provided guidance on the principles and methods for evaluating a building's structural performance after the design fires (inputs) are determined. At the most, C/Vm^2 can produce a time-equivalent formula to calculate the equivalent fire severity and specify building elements with a fire resistance rating not less than the calculated fire severity. This may be considered an elemental approach.

1.3 Need for a performance-based global approach to building fire design

It has long been realised that an elemental approach may not be sufficiently robust (Bailey and Moore, 1999; Bailey 2001, 2003; Wang, 2002; Shyam-Sunder, 2005; Zarghamee et al., 2005; Usmani, 2008; Toh and Bailey, 2007) because:

- it does not account for conditions of structural restraint

- it stipulates an unrealistic fire
- it does not distinguish differences in compartment ventilation or surface composition
- it does not account for realistic structural loads.

Consequently, this type of elemental approach does not provide information about the actual performance (i.e. load-carrying capacity) of a component in a complete building environment nor of the system as a whole or its connections. This is because the performance of building elements in a complete building during realistic fires may be very different from the theoretical prediction based on elemental approaches (Gann et al., 2008). Furthermore, the global structural stability during the entire fire burnout is not well articulated by the specified FRR for isolated elements (Wang, 2002; Gann et al., 2008; Usmani, 2006, 2008). Therefore, in some cases, this approach may over-predict the maximum possible fire endurance time of a structure without undergoing collapse.

For buildings such as high-rises or hospitals, the global structural stability is required for a longer period of time due to the longer evacuation time or the significance of the buildings. In this case, a performance-based fire resistance approach may be more appropriate, which considers the evolution of the building's global structural capacity as it undergoes more realistic fire exposures. It is to be appreciated that this approach may also be used with a standard fire exposure if that is deemed appropriate.

1.4 Scope of the study

The current study intends to present the fundamental principles of designing building for fires rather than the detailed methodology. Nevertheless, the example applications of the design fire modelling techniques and an elemental approach for assessing the structural adequacy of individual structural elements are illustrated in Appendix A.

The current study is limited:

- Design of the members required to fulfil the separating (compartmentation) functions is excluded in this document.
- This document only deals with one subset of building fire resistance design, namely passive fire protection measures only, and it consists of only part of the overall building fire safety strategy.
- Other structural design issues, such as structural design for seismic or other designs at ambient temperature conditions, are beyond the scope of this document.
- This document does not give mechanical properties of various building materials at elevated temperatures. Such information can be easily found elsewhere.

2. Performance-based building design process for fire

2.1 General

New Zealand has a performance-based Building Code in which the objectives, functional requirements and performances are specified. A selection of means (compliance methods) to achieve these are laid out in Acceptable Solutions and Verification Methods. However, other Alternative Solutions are also allowed as long as they comply with the Building Code. Except for the Acceptable Solutions, a compliance method typically consists of following elements:

- A set of explicitly stated performance requirements related to building category and design action (hazard) intensity, which are deemed to meet the NZBC-specified objectives.
- Evaluation methods (determination of actions, analysis method or testing criteria and so on) by which satisfaction of the performance requirements can be achieved.
- Extensive commentaries to explain the basis of the criteria and evaluation methods and to provide guidance in their application.

In a performance-based design environment, there are possibly unlimited compliance approaches that may achieve the established performance requirements. This is again true for structural design for fire conditions. It can be done in a very simple and crude way or in a complicated way considering more details. Whatever the level of complexity, it is essential for designers to know what is being achieved and what assumptions are being made.

In outlining the appropriate design process for conducting performance-based building design for fire, this study does not intend to mandate one process over other alternatives but does identify the process steps that should guide a performance-based design in a complete and systematic manner.

Fire safety in buildings is often achieved by a combination of active and passive fire protection systems. Although the issue of trade-offs between active and passive fire protection measures is not considered in this document, provision of active fire protection measures is still included in the overall process of the building design for fires because structural fire resistance calculation is only part of the larger structural fire safety strategy.

Figure 1 shows the steps for conducting the overall performance-based fire safety strategy, including active control measures and passive control measures. The first four steps are explained in the current section.

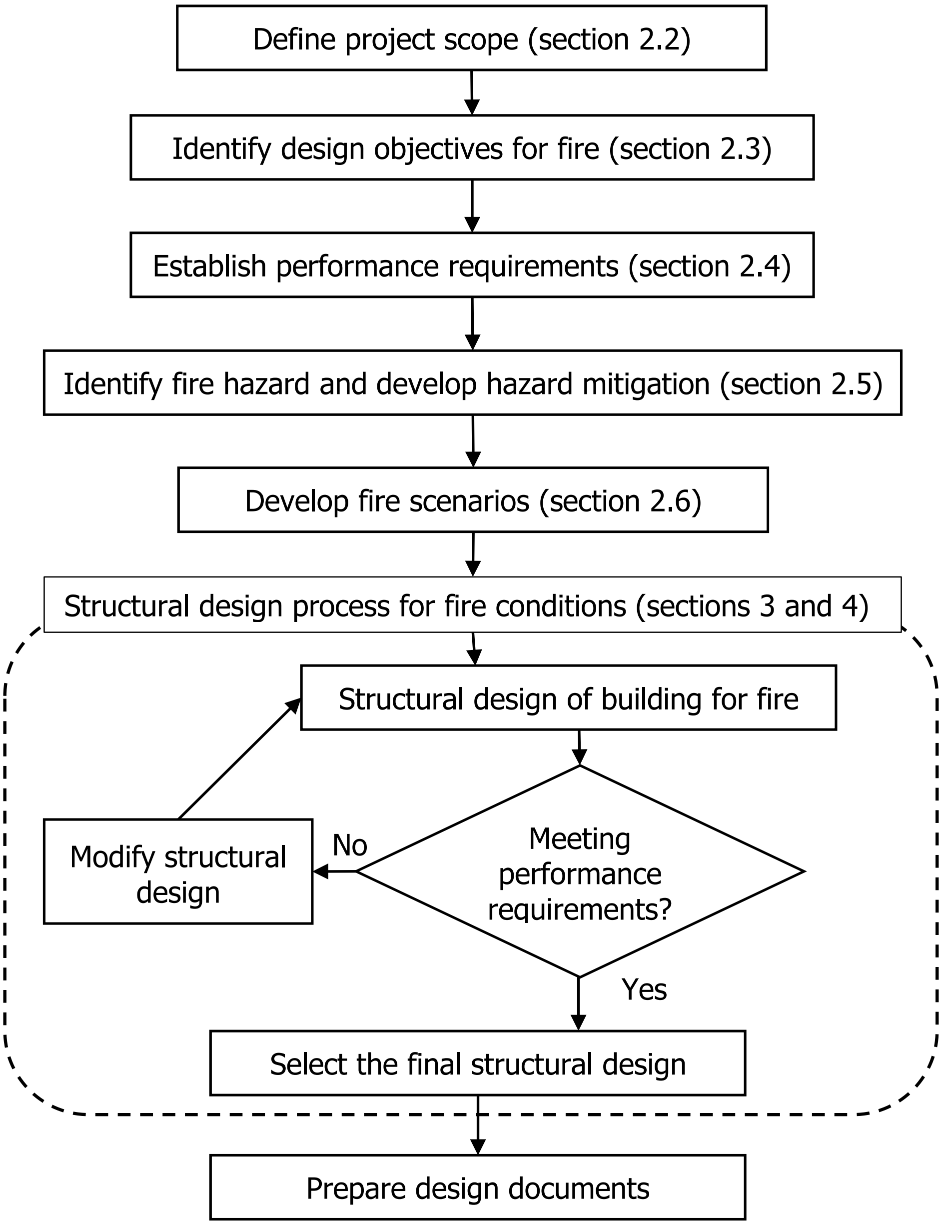


Figure 1. Overview of the performance-based fire safety design process.

2.2 Define project scope

The project scope has to be defined to identify the boundary of performance-based design or analysis under consideration. Boundaries of building design for fire safety could include but are not limited to the following:

- **Project extent:** Consideration in this category should be given to the issues defining the subject project such as whether or not the project under consideration is a new construction or renovation of an existing building and whether or not the project under consideration is a partial building, a complete building or a complex of many buildings.

- **Building and occupancy characteristics:** Building characteristics are essential information in conducting fire risk assessment and developing fire scenarios. Building characteristics include building geometry, ventilation conditions and relationships with the adjacent built environment, and a building's functional and operational characteristics. Occupancy characteristics refer to the proposed use of the building and characteristics of the occupants in order to establish the appropriate performance criteria.
- **Constraints:** There may be many constraints that can affect engineering design approach and philosophy, such as budget (money or time) restraints, restraints from design and construction team organisation, project schedule and so on.
- **Applicable regulations:** All aspects of building design have to be compliant with the applicable regulations so the engineer has to identify the appropriate codes and regulations in order to demonstrate the compliance of the design with the applicable regulations.

2.3 Identify design objectives for fire (specified by NZBC)

Under the current New Zealand regulation environment, the NZBC specifies the overall objectives of building construction work, and these objectives are minimum criteria for any building design. Individual compliance documents quantify the design performance requirements in compliance with NZBC requirements, based on the risk assessment method of the specific design actions. Extra design objectives may be incorporated into the design based on desires of various stakeholders of the buildings.

Relevant NZBC clauses for building fire safety are C1–C6 *Protection from fire*. Clause C6 is about structural stability and which is excerpted below.

> **Functional requirement**
>
> **C6.1** Structural systems in buildings must be constructed to maintain structural stability during fire so that there is:
>
> (a) a low probability of injury or illness to occupants,
> (b) a low probability of injury or illness to fire service personnel during rescue and firefighting operations, and
> (c) a low probability of direct or consequential damage to adjacent household units or other property.
>
> **Performance**
>
> **C6.2** Structural systems in buildings that are necessary for structural stability in fire must be designed and constructed so that they remain stable during fire and after fire when required to protect other property taking into account:
>
> (a) the fire severity,
> (b) any automatic fire sprinkler systems within the buildings,
> (c) any other active fire safety systems that affect the fire severity and its impact on structural stability, and
> (d) the likelihood and consequence of failure of any fire safety systems that affect the fire severity and its impact on structural stability.

> **C6.3** Structural systems in buildings that are necessary to provide firefighters with safe access to floors for the purpose of conducting firefighting and rescue operations must be designed and constructed so that they remain stable during and after fire.
>
> **C6.4** Collapse of building elements that have lesser fire resistance must not cause the consequential collapse of elements that are required to have a higher fire resistance.

In summary, NZBC clauses C6.1–C6.4 deal with three limit states (insulation/integrity/stability):

- Heat transmission leading to unacceptable rise in temperature on unexposed surfaces.
- Breach of barrier due to loss of integrity, which is excluded in this document.
- Loss of loadbearing capacity.

2.4 Establish performance requirements to satisfy design objectives

The objectives specified by NZBC describe how the building should behave, and individual compliance documents have to quantify the design performance requirements that are deemed to meet the Code compliance for different design actions.

For instance, one compliance path for designing buildings for seismic actions is to use NZS 1170.5:2004 *Structural design actions – Part 5: Earthquake actions – New Zealand* in combination with relevant design standards. NZS 1170.5:2004 specifies the structural building performance requirements related to the specific design actions of different risk levels.

Similar to earthquake actions, fire effects have many uncertainties, so no building system can be engineered and constructed to be absolutely risk free from the effects of fires. As it will never be possible to design to be totally risk free, an appropriate level of risk must be established. An informed assessment and risk management of the fire hazards becomes the basis to the success of performance-based fire engineering for measuring compliance with performance objectives.

Seismic design of buildings is an example. Design seismic actions are derived based on probabilistic risk assessment and are presented for two limit states – ultimate limit state (ULS) and serviceability limit state (SLS). Subsequently, the building performance requirements are established for two limit states, as specified in NZS 1170.5:2004.

Building design requirements for earthquake actions at ULS are:

- avoidance of collapse of the structural system
- avoidance of collapse or loss of support to parts
- avoidance of damage to non-structural systems necessary for emergency building evacuation that renders them inoperative.

Building design requirements for earthquake actions at SLS are:

- to avoid damage to the structure and non-structural components that would prevent the structure from being used as originally intended without repair after an SLS1 earthquake
- to avoid damage to either a structure deemed as a critical post-earthquake designation or all the elements required to maintain those operations after an SLS2 earthquake.

Here, SLS1 and SLS2 indicate different levels of serviceability limit states in accordance with NZS 1170.5:2004.

For building fire safety designs, there is research development towards the probability-based fire risk assessment, which is more compatible with the derivation of other actions used in structural design. Fire safety design for buildings is different from other structural designs, such as earthquakes, because the potential for fires to cause significant structural damage depends on the presence and timely activation of fire and smoke detection and suppression systems and quick response of the Fire Service. The conditional nature of the limit states in the fire condition means that general design strategies for the fire condition require a statement of performance objectives and a general approach to risk management, and the latter is beyond the scope of this study.

Establishment of building performance requirements when structurally designing buildings for fire conditions is required to take risk management and firefighting into account. In addition, the objectives specified in NZBC clauses C6.1, C6.2, C6.3 and C6.4 set up the minimum objectives, and stakeholders may want higher structural fire performance objectives. Hence, defining building performance requirements in fire conditions is multi-faceted.

For a specific fire scenario, the minimum performance requirements in designing buildings for fires from the structural design perspective, which are deemed to meet NZBC specified objectives, are:

- avoidance of collapse or damage to structural elements necessary for maintaining structural stability and integrity of the escape route for at least the time appropriate for the occupancy and the fire intensity
- structural members and assemblies exposed to fire shall have adequate fire resistance for at least the time appropriate to their functions, the occupancy groups, the fire hazard (fire load, fire intensity and the duration), the fire protection measures and their proximity to the adjacent building
- avoidance of progressive (disproportionate) structural collapse in accordance with NZBC clause C6.4.

Apparently, the primary performance objective underlying structural engineering for fire conditions specified in NZBC clauses for fire safety is prevention of premature structural failure and control of structural failure progress.

Unlike structural design for earthquakes where design strategies aim at the lateral force-resisting system, for fire, they would be aimed at enhancing the integrity of the structure subjected to gravity loads in a degraded or damaged condition.

2.5 Identify fire hazard and develop hazard mitigation

To identify fire hazards appropriately, it is necessary to consider various factors that can influence the development of a fire such as fuel load, type and arrangement, ventilation characteristics and the geometry of the compartment. Note that the effectiveness of firefighting is generally not included although it can affect the fire development.

However, these factors and their influences on fire development are not often easily obtained. It is partly because modern buildings include complex and non-conventional architectonical elements and designs. These can lead to fire environments diverging significantly from those used in the development of current codes and standards as well as many engineering calculations. Large enclosures, high ceilings, connected floors, and composite building elements and plastic-based fuels are not exceptional features of modern architecture. These distinctive characteristics influence the fire environment but have not been systematically and consistently reflected in selecting design fires. Certainly, further research needs to be conducted in this area.

2.6 Develop fire scenarios

To account for building-specific characteristics, the fire hazard analysis is likely to be scenario-based. A design fire scenario identifies a set of conditions – sources of ignition, nature and configuration of fuel, ventilation, patterns of growth and spread of smoke, availability of active fire detection and protection systems and so on. Of course, this approach cannot encompass all possible fire events, but by analysing multiple fire scenarios, the uncertainty is mitigated to the extent agreed among stakeholders.

From a structural engineering viewpoint, the critical factor is the hot gas temperature from fire, since the performance of structural elements can be decreased at an elevated temperature. Therefore, obtaining an adequate time-temperature curve is required to predict structural performance appropriately.

There are various ways to obtain the time-temperature curve such as:

- the standard time-temperature curve (ISO 834-1:1999)
- parametric fires (EN 1991-1-2:2002 *Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire*)
- travelling fires (Stern-Gottfried, 2011)
- temperature output from fire modelling tools.

More details for the time-temperature curve from design fires are included in Appendix A:.

3. Structural design process for fire conditions

3.1 General

Structural design of buildings for fires is carried out on the basis of the established building performance requirements for fire and selected design fire. This section provides general guidance on approaches to and practical aspects of implementing a fire resistant design for conventional applications after the design fire is determined.

Fire can affect a building's structural capacity in two ways:

- Prolonged exposure of structural components or subsystems to elevated temperatures degrades their engineering properties, thus resulting in the reduction of overall structural capacity.
- Exposure to elevated temperature may induce internal forces (due to restraint of thermal expansion) or axial deformations in structural members due to plastic and creep strains or buckling, which may require alternative load path/load redistribution, causing progressive structural failure and other adverse effects on the global structural stability of the building.

Designing structural fire resistance on a performance basis generally includes the following steps:

- Determine the design fire, which gives information on the thermal environment surrounding a building structure (or portion thereof) in the event of a fire (see section 2.5 and Appendix A:).
- Determine the thermal response of the structure/structural elements exposed to fire by conducting heat transfer analysis. This will produce temperature profiles across the member sections and along the lengths of components (see Appendix B:).
- Evaluate the structural response in fire conditions. This involves evaluation of the direct impacts of temperature dependent material properties, secondary effects due to thermal expansion and contraction as well as load redistributions across the entire building. Hence, material and geometry non-linearities often are important.

3.2 Heat transfer (thermal analysis)

Heat transfer analysis can range from simple one-dimensional (or lumped mass) equations to analysis with finite element software depending on the complexity of the geometry and heat flow. One-dimensional analysis is simple and can be performed by hand, while more complicated two and three-dimensional heat transfer analysis normally requires computer-based solutions and is used for complex problems, particularly when heat flows through different materials with dissimilar thermal properties.

Some finite element software can model the characteristics of materials of construction and insulation, including the effects of air gaps and the various modes of heat transfer (radiation, convection and conduction) in complicated geometries.

The output of heat transfer analysis usually needs to be converted to input files for structural analysis software. Designers decide the required analysis levels of precision and select a proper analysis tool.

The component temperature data may need to be reduced to be compatible with the structural response analysis program and the design objectives. For instance, the designer might need to approximate non-linear temperature profiles as linear if the analysis program cannot receive as input non-linear profiles between nodes along the length or across the member section.

The temperature histories from the heat transfer analysis should be reviewed before selecting a time interval for the structural analysis input of temperature data. For example, a 120-minute fire scenario may produce relatively rapid heating in the first 30–60 minute, followed by cooling at a slower rate.

The time interval for the analysis of the heating portion of the time-temperature relationship should capture the temperature rise through linear interpolation between data points (Zarghamee et al., 2005). The same time interval could be used for the more gradual temperature changes typically associated with cooling. The time interval could be increased if the time-temperature curve can be adequately simulated with large time increments.

3.3 General approach for evaluating building structural response in the fire scenarios

3.3.1 Structural analysis principles

The principles of structural analysis used in designing buildings for fire conditions are similar to the structural analysis for other loading conditions such as earthquakes. In earthquakes:

- material mechanical properties (stiffness, strength and so on) of predefined building elements could degrade as earthquakes progress with time
- different elements will experience different degrees of degradation at a time
- building elements could also experience failure mechanism changes from a flexure-dominated mechanism at small displacement level to shear-dominated failure mechanism at larger displacement level as for a reinforced concrete beam.

Hence, the structural analysis for earthquakes needs to capture the following characteristics:

- Updating material mechanical property degradations of the predefined building elements likely to be in non-linear response range at a time. The building elements likely to be in elastic range have unchanged stiffness properties.
- Adequately simulating failure mode changes of the building elements whenever these occur.
- Updating the resulting dynamic property of the building based on the updated material mechanical property.

Structural fire analysis will follow similar principles to those used in structural seismic analysis except that it is a static analysis process.

The structural analysis for fire conditions needs to capture the following characteristics:

- Updating material mechanical properties of the building elements directly exposed to fires at a time based on the time-dependent thermal analysis results. The building elements outside the area directly exposed to fire are likely to have unchanged stiffness properties.
- Adequately simulating failure mode changes to the building elements directly exposed to fire whenever these occur.
- Secondary effects due to the large deflections have to be effectively modelled. The necessary load redistribution resulting from material property degradation of the building elements exposed to fire will be automatically considered if a global structural analysis is carried out.

Compared to conventional structural analysis under gravity load, structural fire analysis is more complicated because of the effects of elevated temperature on material properties and the introduction of internal actions.

The outlined differences of structural fire analysis from structural analysis at ambient temperature are that, at the time of a fire:

- internal forces may be induced by thermal expansion/contraction
- strength of materials may be reduced by elevated temperatures
- cross-sectional area may be reduced by charring or spalling
- smaller load factors can be used because of the low likelihood of the combination of events
- deflections are less important than strength
- different failure mechanisms need to be considered.

3.3.2 Design requirement

As discussed previously, small deflections of the structural systems in fire conditions are deemed to be less important than the strength criterion.

Verification of design for strength requires that the applied loads are less than the load capacity of the structure for the entire duration of the fire design time. This requires satisfying the design equation given by:

$$\mathrm{U}^*_{\mathrm{fire}} \leq \Phi_f \mathrm{R}_{\mathrm{fire}} \qquad \text{Eq. (1)}$$

Where,

$\mathrm{U}^*_{\mathrm{fire}}$: the design action from the applied load at the time of the fire

$\mathrm{R}_{\mathrm{fire}}$: the nominal load capacity at the time of fire

Φ_f : the strength reduction factor for fire design to account for the uncertainties in estimating the material strength and section size. Usually Φ_f =1.0 is used for structural fire design because fires are rare events.

Hence, the structural fire design equation for all structures becomes:

$$\mathrm{U}^*_{\mathrm{fire}} \leq \mathrm{R}_{\mathrm{fire}} \qquad \text{Eq. (2)}$$

In determining nominal load capacity of structural members at the time of fire, material property degradation at elevated temperatures as well as the internal actions induced due to secondary effects resulting from the restraints by the surrounding structures or the change of member's load-carrying mechanisms (for instance, the mechanism has changed from flexural failure mechanism to buckling failure mode) need to be taken into account.

3.3.3 Load combinations when designing buildings for fire

The load combination under fire conditions specified by B1/VM1 is:

$$L_f = G + \psi_l Q + T \qquad \text{Eq. (3)}$$

Where,

L_f: load combination considering fire conditions

G: permanent action

Q: imposed action

T: thermal action arising from fire being internal forces or displacement

ψ_l: long-term factor, which is usually taken as 0.4 except for storage or similar occupancy.

Thermal actions, T, are the actions induced in structures due to the restraints from the surrounding structures or the change of a member's load-carrying mechanisms, for example, an original flexural member can no long take moment actions and has degraded into a tensile member.

3.3.4 Evaluation methods for predicting structural performance in fires

As stated in section 3.3, principles of structural analysis in fire conditions are similar to those in earthquakes. Adequate finite element structural analysis software programs need to simulate the material property degradation based on the outputs of thermal analysis. They also need to simulate the transition of failure modes of building elements and to capture the secondary effects of thermal expansion/contraction.

As for seismic analysis, methods of different complexities should be developed for different applications. Currently available approaches for evaluating a building's performance in fire conditions range from a simple member method to a partial analysis method to a complete structural analysis. Depending on the design situations and the scope of the projects, different structural analysis approaches for fire conditions may apply. Examples of these three methods are described in EN 1992-1-2:2004 *Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire* design, and these three methods will be briefly described here.

After the design fires are determined, a typical analysis process for conducting structural fire performance evaluation is shown in Figure 2.

Time-temperature curve

⇩

Derive material properties at time, t_i, based on temperature profile for structural member exposed to fire

⇩

Apply loads (load combination in fire condition is G+0.4Q)

⇩

Carry out structural static analysis for the complete building under G+0.4Q based on the derived material properties at the time, t_i, by adequately allowing for the effects of end restraints and mechanical property degradation due to elevated temperature. As a result, the actions (forces and deformations) generated in the structural members are found.

⇩

Estimate the load-carrying capacity of the members at time, t_i, by allowing for the temperature profile derived at time, t_i, and compare the capacity with the actions. If the action is less than the capacity, the member still can carry load, otherwise the member fails and the structural analysis model has to be modified to take account of the failed member(s). A failed member may introduce new loads (e.g. tensions) on the surrounding structure. Rerun the model with a modified member size until all the members have derived actions less than their capacities.

Figure 2. Flow chart of conducting structural fire performance evaluation.

Member method

The member method can be prescriptive or a standardised analytical approach.

Prescriptive method

Building codes specify fire resistance rating requirements (in terms of hours) for performance of building components under exposure to the standard fire. Fire ratings are usually determined by testing (e.g. following ASTM E119 *Standard test methods for fire tests of building construction and materials*). Catalogues of components with specified construction details and their approved fire rating are available for selecting an appropriate protection configuration that is consistent with the architecture and goals of the project. When this approach is used, designers need to consider connections between rated structural components.

For instance, if columns have a required 3-hour rating and floor beams and girders have a required 2-hour rating, the design may specify that the connections be protected according to the higher rating (i.e. match the column level of protection). By testing structural components in accordance with the standard fire curve, fire protection products are rated according to the common standard. Performance of components and the structural system during actual fires requires a comparison between actual conditions and conditions during standard fire tests. Codification of performance levels based on exposures to standard fires does not necessarily ensure that the structural system will, indeed, be able to sustain the effects of a real fire for the duration implied by the rating.

Standard analytical method

For designs where prequalified details based on standard fire testing are not available, analytical approaches for determining fire resistance of many common structural systems are available in reference documents such as ASCE/SEI/SFPE 29-99 1999 *Standard calculation methods for structural fire protection* and ACI 216.1-97/TMS 0216.1-97 1997 *Standard method for determining fire resistance of concrete and masonry construction assemblies*. These standard methods provide procedures for determining fire resistance to standard ASTM E119 fires in terms of hours, before reaching a defined endpoint criteria for common structural components and type of fire protection.

Partial structural analysis method

EN 1992-1-2:2004 has a partial structural analysis method. A time-temperature curve is necessary input data for heat transfer analysis to determine temperature profiles through component cross-sections. The structural performance is determined based on the potential thermal expansion and deformations. Their interaction with other parts of the structure can be approximated by time-independent supports and boundary conditions during fire exposure. In conducting a partial structural analysis, realistic simulation of the boundary condition of the chosen partial structural model is a crucial part of the process in realistically evaluating the structural performance in fires. However, EN 1992-1-2:2004 does not contain detailed information on modelling of the boundary condition. The partial analysis method in EN 1992-1-2:2004 also does not require an investigation into the secondary effects on the structural global performance, such as the availability of an alternative load path and other general global stability issues.

The partial structural analysis method is further developed in section 4.

Complete structural analysis method

Using this method, analysis is conducted on the complete building. The fire effects on the material property degradation are considered for building elements exposed directly to fire while other elements have unchanged material properties as at ambient temperature. The secondary effects due to large deformations are to be taken into account.

A time-temperature curve is necessary input data for heat transfer analysis to determine temperature profiles through component cross-sections. With some software packages such as Abaqus, component stresses and deformations can be evaluated simultaneously with the determination of temperatures. With others, temperature profiles are computed separately and then provided as input for structural analysis.

Typically, if a finite element analysis package offers an option to solve heat transfer and structural response in a single analysis, it uses solid elements. While some software packages have features that partially automate the analysis for certain mechanical systems, such as engines or radiators, such conveniences generally do not exist for beam and shell elements normally needed for structural analysis. The analyst must first conduct a heat transfer analysis with solid elements and then translate equivalent temperatures to the nodes of beam and shell elements for the structural analysis.

4. Rational global approach to building design for fires

4.1 General

As discussed in section 1, an elemental approach considers a structural element in isolation. It neither allows for building characteristics in stipulating the fire environment nor the interactions between the elements/assemblies directly affected by fire and the rest of the structure. Hence, an elemental approach is not rational in many ways. Fire effects are often not limited to the building elements directly exposed to fire – the indirect (or secondary) fire effects can cause progressive failure of the building structure or elements. They can even lead to the loss of building's global structural stability and total collapse of the entire building, especially with a severe and long-duration fire.

In a complete building, the structural elements directly affected by fire will undergo expansion during the growth phase of the fire and contraction during the decay and cooling phase of the fire. Expansion and contraction of the fire-exposed structural elements will induce interactions between the structural elements directly affected by fire and the adjacent structural systems, necessitating a load redistribution across the complete building. If alternative load paths are not available or inadequate, localised structural failure may spread from the fire-exposed structural elements to the rest of the building, leading to possible progressive structural failure or even total structural collapse.

In recent years, there have been developments in global structural fire analysis approaches, such as the partial and complete structural analysis methods as discussed in section 3. Both the partial and complete structural analysis methods in EN 1992-1-2:2004 do not give much detailed guidance for structural engineers and do not have in-depth quantification and clarification about the design philosophy. Hence, the challenges of theoretically predicting structural performance in fires by partial or complete structural analysis are huge, and wide application of these methods requires more clarification and tools.

The partial analysis method in EN 1992-1-2:2004 states:

> … that the part of the structure to be analysed should be specified on the basis of the potential thermal expansions and deformations such that their interactions with other parts of the structure can be approximated by time-independent support and boundary conditions during fire exposure …
>
> … the boundary conditions at supports and forces and moments at boundaries of part of the structure, applicable at time t = 0, are assumed to remain unchanged throughout the fire exposure.

The following questions arise:

- Is the partial analysis method as described above adequate for preventing the failure of elements exposed to fire causing a progression to the other parts of the building?

- Is the partial analysis method as described above adequate for capturing the maintenance of structural global stability due to indirect (secondary) fire effects?

The determination of adequate boundary conditions for the isolated partial structural model has become crucial in answering these questions. Inadequate determination of the boundary conditions could result in failure to identify the potential progressive failure/collapse and the required measures to maintain structural global stability.

In this section, a two-staged global approach is developed – a partial structure in fire analysis supplemented by global structural stability verification. Building performance in earthquakes and in fires is examined first to investigate the possibility of using the seismic design principles for building designs in fire conditions. Following the examination, a philosophy of designing buildings in fire conditions is established and a rational design procedure framework is developed.

4.2 Building performance in earthquakes versus building performance in fires

4.2.1 Fire actions versus earthquake actions

Compared with other design actions used in building structural designs, fire actions have more similarities to earthquake actions although differences still exist between these two types of actions.

In seismic engineering, the input actions to the buildings are time-dependent ground motions. In fire events, the input actions are time-dependent fire histories, and the time periods are always much longer than earthquake ground motions. The time-dependent nature of either earthquakes or fires means that structural performance for the entire period of the event needs to be studied. However, unlike seismic structural engineering where the building response is dynamic and the entire building is affected by the ground motion, a fire event is typically a localised phenomenon and is more likely to occur within one part of a building, although exposure to high temperature in one area can induce internal actions to the affected members and to the structural systems surrounding the affected area.

4.2.2 Building performance comparison

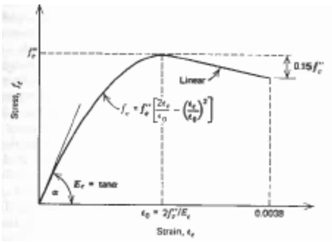
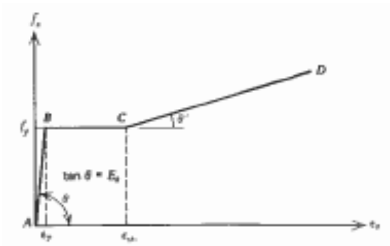
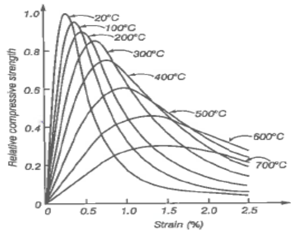
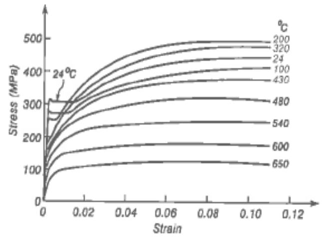
When a building is subjected to an earthquake, the building responds in a dynamic manner, that is, the actions induced in building elements due to the earthquake are dependent on the dynamic properties of the building. The building's dynamic properties may vary as the earthquake progresses because some structural members of the building are stressed beyond yield. This leads to degradation of a material's engineering properties and reduction in the overall structural stiffness of the entire building.

This phenomenon is typically described as a non-linear response. In analysing the structural performance in earthquakes, the assumption of small deflections is always assumed to be true.

Similarly, prolonged direct exposure to high temperature can lead to degradation of a material's engineering properties, resulting in reductions in strength, stiffness and the load-carrying capacity of the building elements.

The material property reduction evolves as the fire develops with time, which is one type of non-linearity in structural fire engineering. Due to the degradation of mechanical properties of building elements directly exposed to fires, load redistributions could occur, inducing secondary actions to the rest of the building structure. If the rest of the building is not adequately designed to resist the induced actions or if there are no alternative load paths for redistributing the load, disproportionate failure will occur, potentially progressing to total collapse of the building. In addition, the small deflection assumption is no longer true when analysing the building response. Secondary effects due to large deflection become critical issues and have to be taken into account when analysing the building response in fire conditions. Table 1 lists the characteristics of building performance in earthquakes and in fires.

Table 1. Similarities and differences of building performance in earthquakes and fires.

	Seismic response analysis of buildings	**Structural analysis for buildings in fires**
Material mechanical properties	concrete  steel  One single stress-strain curve for each material over the entire time period.	concrete  steel  An array of stress-strain curves for each material over the entire time period, varying with temperature.
Material properties	Elastic or non-linear response.	Non-linear response and material mechanical property degradation.
Geometry	Small deflection assumption is adequate.	Small deflection assumption is no longer adequate and the secondary effects need to be considered.

4.3 Review of seismic code development

Seismic engineering development for building designs has advanced from an elemental approach to a global approach.

Prior to the advent of modern seismic design concepts, the seismic design code in New Zealand for buildings was basically an elemental approach. Namely, a horizontal seismic loading was applied to the building, the distribution of the total horizontal seismic load up the height of a building was either uniform or an inverted triangular shape and the design method was a working stress method (Park and Paulay, 1975; Paulay and Priestley, 1992). Static structural analysis was conducted for the building under such horizontal seismic forces to derive the actions for each member, referred to as seismic demand. The engineer just had to provide sufficient capacity for each member without considering the interrelations between adjacent members. The combined specific stress (for instance, shear) under seismic action and gravity action for a member was not allowed to exceed the allowable stress. If this was the case, the member design was considered to be adequate irrespective of the condition of other members or other failure modes.

Therefore, the buildings designed using this elemental approach (as seen in pre-1970s reinforced concrete buildings) are often expected to develop an undesirable failure mechanism in a major earthquake. For example, this may include premature brittle failure modes for a member, such as can occur with a weak column-strong beam mechanism.

The modern seismic design standards for buildings have moved away from the elemental approach to a global approach. The key strategies used in modern building design for earthquakes are to predetermine the desirable failure mechanisms by considering the integrated nature of building elements in a complete building and to take measures to prevent premature brittle failure from occurring for a member. For example, columns in a reinforced concrete frame structure will be designed to resist the overstrength of the beams framing into the column to ensure that the beams fail first.

4.4 Development of a rational design procedure for buildings in fires

Based on the similarity of actions on buildings from earthquakes and fires, it is proposed that the procedure to design buildings for fire be similar to the current building design procedure for earthquakes (Ellingwood et al., 2007; Grosshandler, 2008; Usmani, 2008; Flint, 2005; Lamont, Lane, Flint and Usmani, 2006; Gann et al., 2008).

4.4.1 Objectives of the NZBC for structure and structural stability in fires

The nature of earthquake and fire events is the same in that both are rare events. This also means that society would be able to accept higher consequences of building damage under these events than gravity loads or live loads.

In line with the similarity of this acceptable risk concept, the objectives for structure and structural stability in fire conditions specified in the NZBC are also quite similar as shown in Table 2.

Table 2. The objectives specified in the NZBC.

	For structure	For structural stability in fires
Objectives	• Safeguard people from injury caused by structural failure. • Safeguard people from loss of amenity caused by structural behaviour. • Protect other property from physical damage caused by structural failure.	• Safeguard people from an unacceptable risk of injury or illness caused by fire. • Protect other property from damage caused by fire. • Facilitate firefighting and rescue operations.

4.4.2 Design verification procedures for structural design of buildings for fire

Compliance with the NZBC is typically achieved through Acceptable Solutions or Verification Methods, which refer to various standards. For earthquakes, NZS 1170.5:2004 is commonly referred to. It sets out procedures and criteria for earthquake actions to be used in the limit state design approach. The performance requirements for the ULS state in NZS 1170.5:2004 and structural stability in fire conditions in the NZBC also show a similarity, as shown in Table 3.

Table 3. Comparison of performance requirements of ULS and fire design approaches

	For ULS design from NZS 1170.5:2004	For structural stability in fires from the NZBC
Performance requirements	• Avoidance of collapse of structural systems. • Avoidance of collapse or loss of support to crucial parts. • Avoidance of damage to non-structural systems necessary for emergency evacuation.	• Structural systems in buildings that are necessary for structural stability in fire must be designed and constructed so that they remain stable during fire and after fire when required to protect other property. • Structural systems in buildings that are necessary to provide firefighters with safe access to floors for the purpose of conducting firefighting and rescue operations must be designed and constructed so that they remain stable during and after fire. • Collapse of building elements that have lesser fire resistance must not cause the consequential collapse of elements that are required to have a higher fire resistance.

To achieve the required building performance, the principles of a Verification Method for designing buildings for fires are established based on the format of structural design for earthquakes. These are outlined in Table 4.

Table 4. Verification methodologies

	Building design for earthquakes at ULS	Building design for fires
Aspects to be verified	• Deflection criteria. • Strength criteria. • Promoting ductile failure mode by ensuring adequate structural failure mechanisms and adequate failure mode of building elements – capacity design.	• Adequate strength of building elements for the entire fire development cycle. • Adequate structural system to maintain structural stability at local scale. • Adequate structural system to maintain structural stability at global scale to prevent disproportionate failure.

Verification of the deflection and strength criteria in building designs for earthquakes is basically conducted at an elemental level to ensure the action (demand) is less than the capacity. The core element of seismic building design is the appropriate capacity hierarchy among the elements and within a single element. The capacity design philosophy promotes a ductile failure mode and prevents brittle failure modes at both a global scale and local scale. This is outlined in Table 5.

A fire incident starts as a localised phenomenon affecting the structural members directly exposed to the fire. The loads imposed by these members, however, are transferred to other surrounding structural members as well. Therefore, it is again very important that the strength capacity hierarchy between building elements and the maintenance of structural global stability after the necessary load redistribution are adequately addressed. Similar to the design philosophy for earthquakes, it is postulated that building design for fires should promote stronger vertical supporting systems than horizontal floor systems in both a local and global sense. This is to maintain local and global structural stability during the entire fire development period, preventing possible disproportionate collapse of the structure. This concept is summarised in Table 5.

Table 5. Concept of capacity hierarchy in designing buildings for earthquakes and fires.

	Building design for earthquakes at ULS	Building design for fires
At global scale	For a frame structure, preferred failure mechanism is a strong column-weak beam mechanism rather than a soft-storey mechanism (see Figure 3).	For the entire building, the indirect fire effects, such as the load redistributions, resulting from the degradation of the material properties of the fire-affected elements must be checked to prevent global structural instability from occurring and prevent the spread of local failure in the fire enclosure from happening.
At local scale	For each individual structural element, ductile failure modes such as flexural failure are promoted. Meanwhile, premature brittle failure modes, such as shear or bond failure, are prohibited.	For a firecell – the partial structural model – vertical load-carrying elements must be designed to have more reserve capacity than horizontal elements such as floors and beams.

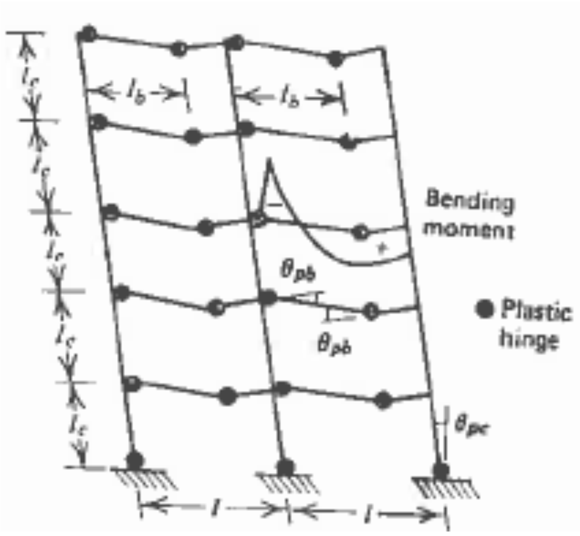


Figure 3. Preferred strong column-weak beam failure mechanisms in earthquakes for a frame structure.

Adequate strength of building elements throughout the entire fire development period is verified by ensuring the actions (demands) are less than the available capacity, taking into account the effect of elevated temperatures.

To this end, the actions are to be calculated by a two-stage static analysis – one being a partial structural analysis allowing for thermal effects and the other being a global analysis. In designing individual elements, a greater safety factor is considered for vertical load-resisting systems (e.g. columns) to result in a stronger vertical system than a horizontal system (e.g. beams).

4.4.3 Two-stage analysis procedure for structural fire design of buildings

The global approach developed here is a two-stage design approach that aims to achieve structural stability at a local and global scale. Throughout both design stages, the interactions between structural elements/assemblies directly exposed to fire and the rest of the structure within a complete building are considered.

The first stage is to conduct a structural fire analysis for the part of the structure directly exposed to fire, which requires thermal and structural analysis. The critical issue for the first stage is to adequately define the loads induced in the surrounding structure by the firecell. In analysing the partial structural model, the effects of elevated temperature on the mechanical properties of the affected building elements need to be allowed for. The resultant reaction forces at restraining boundary springs can then be obtained. The resultant reaction forces will be time-dependent actions. Structural engineers should use engineering judgement to determine the critical cases where the global structural stability may be compromised.

The second stage is to study the structural global stability by conducting a conventional structural analysis using the loads from the springs in the first stage of the analysis.

The two-stage analysis is rational and sufficient because the structural performance in fire at any moment includes two phenomena – fire effects on structural members directly exposed to fire and secondary effects to the entire structure caused by thermal expansion and change in behaviour/capacity of those members. Clearly, a global structural fire analysis for the entire building is unnecessary.

The designer needs to ask:

- Is the load redistribution capability available?
- Is the alternative load path adequate to maintain structural stability in the global sense after the fire-affected area has undergone thermal expansion and stiffness/strength degradation?

5. Conclusion

The fire engineering discipline has a relatively short history compared to the structural engineering discipline. Despite significant developments over the past 60 years or so, the knowledge of fire science/engineering has not been comprehensively merged into other associated disciplines such as structural engineering. This gap has been substantiated by several fire incidents that led to structural collapses such as the WTC7 building, New York, USA, the Architectural Department building in Delft University, the Netherlands, and the Windsor Tower, Spain. These incidents raised issues in both fire engineering and structural engineering communities, as the buildings could not withstand the fire conditions despite the compliant structural and fire design. The structural components may have been designed underestimating the fire effects, or the fire may have been estimated to be below the actual severity. In either case, it has often been pointed out that a more cohesive design approach is necessary between structural engineering and fire engineering.

The current study is part of an effort to decrease the gap between the two engineering disciplines. It overviews the current fire engineering and structural engineering approaches in building design and identifies differences and similarities. It also indicates that fire effects should be accounted for not only in individual structural elements but also in the structural system as a whole.

As this study aimed to establish a framework to develop a more cohesive design approach, the performance of structural elements and their influence on the whole structure in fire conditions have not been investigated. Further research needs to be conducted focusing on these.

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Appendix A: Design fires for structural design in fires

A.1 Introduction

Design fires for structural design are intended to include parameters for the fire load energy density (FLED), fire growth rate and heat of combustion, allowing a post-flashover structural design fire to be defined. Structural design fires are based on the complete burnout of the firecell with no intervention.

An important factor is to consider that higher temperatures than the standard fire curve (ISO 834-1:1999) may be reached, possibly up to 1200°C (Whiting, 2003; (Nyman, 2002), and that the total heat release (THR) over the duration of the fire matches the FLED of the compartment. The maximum temperature is more of a function of the ventilation allowing exchange of hot gases with fresh air and the rate at which fuel is available and to a lesser extent the total fuel available. Having specified a time-temperature profile for the expected conditions (worst-case scenario), the thermal response of the individual elements of the structure can be ascertained and then its ongoing ability to support the overall load imposed on and within the structure.

A.2 Background

Previous BRANZ research (Narayanan, 1995) considered the fire loads in a range of buildings (New Zealand conditions) and how this may be applied to design fires and structural fire design. A simplified design process is presented in Figure 4. Fire severity is determined on the basis of the Swedish curves (Petterson, 1984), and determination of the structural adequacy is outside the scope of the study.

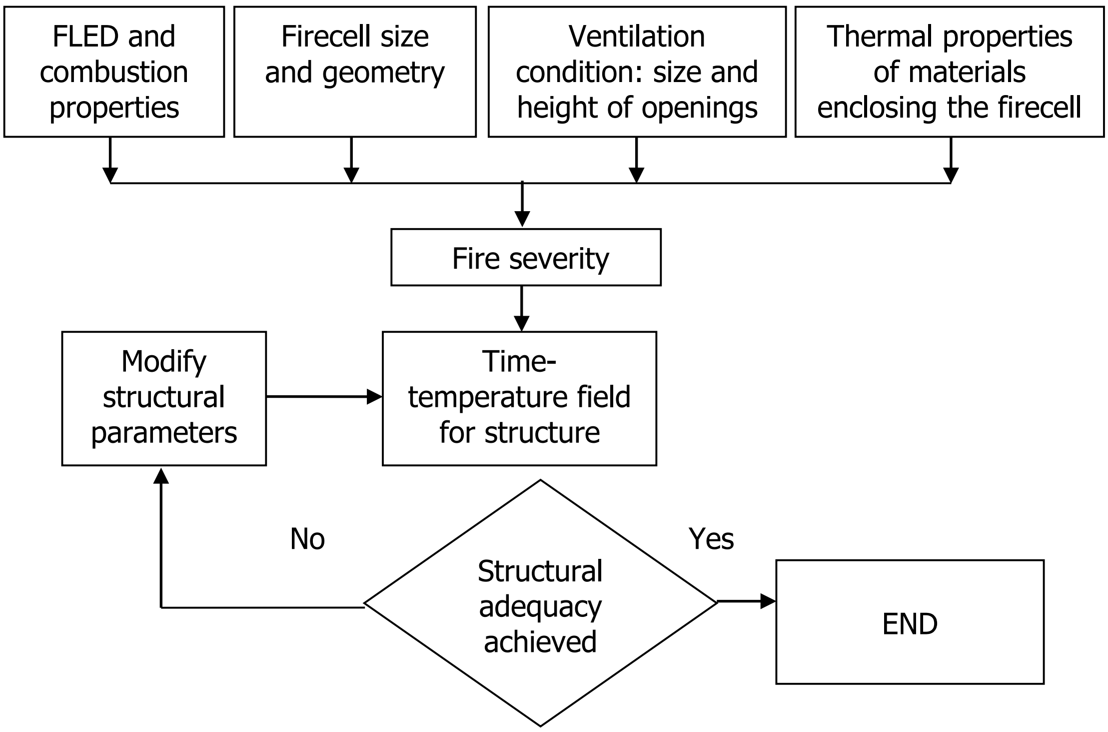


Figure 4. Simplified flow chart – steps in fire engineering design (Narayanan, 1995).

This guide considers fire scenarios that may be outside the envelope of current documented design fire practice where fires may challenge a structure in different ways.

The concept of equivalent fire severity does not necessarily take account of higher temperatures that may be more harmful to the structural performance, albeit for a shorter duration.

Extreme fire exposures to a structure may come about by fires burning in a localised area where flames may impinge directly onto a part of the structure and entrained air may generate near stoichometric conditions resulting in more effective burning of the fuel.

For design purposes, a creditable worst-case fire exposure scenario needs to be identified against which the fire performance of the structural design is assessed and modified as required.

A.3 Design fires for structural design in fire

Selecting suitable design fires along with methods to assess thermal impact on structural elements is the key to evaluating a fire design. The following design fires are of increasing complexity but are likely to yield more realistic results.

A.3.1 ISO fires

Traditionally, fire design has been based on the standard fire resistance test (ISO 834-1:1999, AS 1530.4 -*2005 Methods for fire tests on building materials, components and structures –-Fire-resistance test of elements of construction*) to demonstrate structural adequacy/integrity/insulation of various building elements (Figure 5).

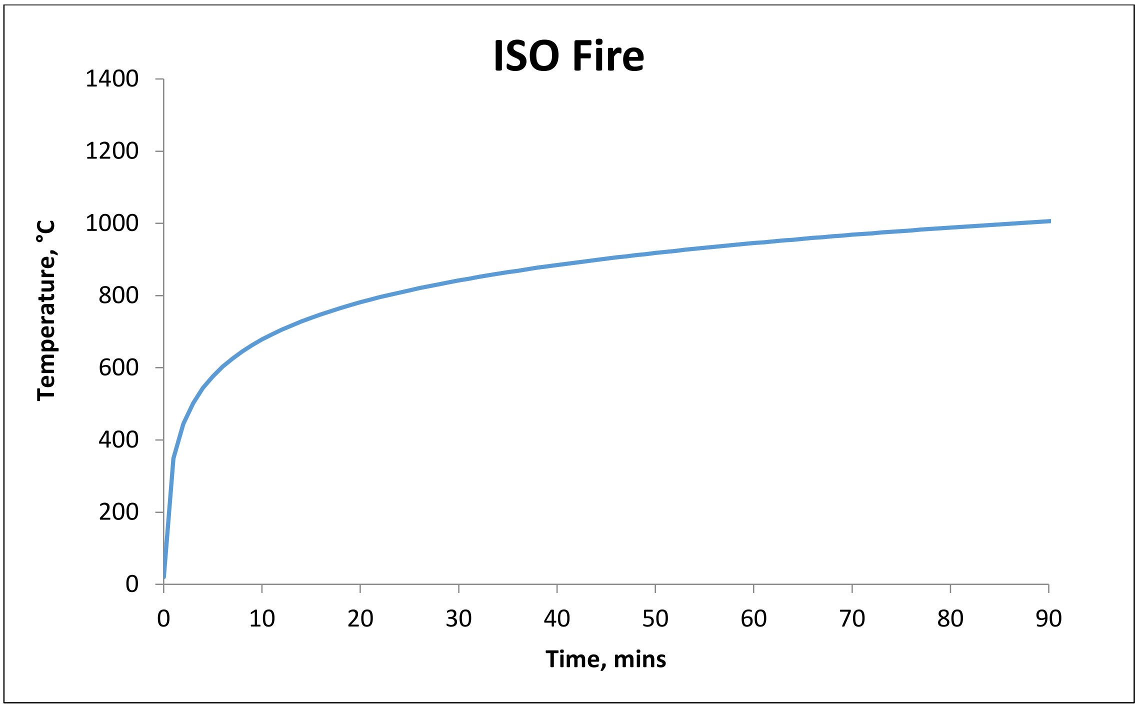


Figure 5. ISO fire.

This is only a rating and, apart from specifying the fire resistance rating (FRR), does not bear any relationship to the actual results that would be expected. In practice, the FRR is based on occupancy type and depends on the likely fire load (FLED).

A structural building element may achieve an FRR by test for the required period as stipulated by the occupancy risk group and the FLED, but in reality, an actual fire may be markedly different such that the structural adequacy achieved is significantly less than the rating.

A.3.2 Parametric fires

Improvements to specifying the likely fire exposure temperatures may be achieved using parametric fires (EN 1991-1-2:2002) as shown in Figure 6. These are based on the fire load, ventilation, compartment dimensions and thermal properties of the internal surfaces, in this case, standard density concrete.

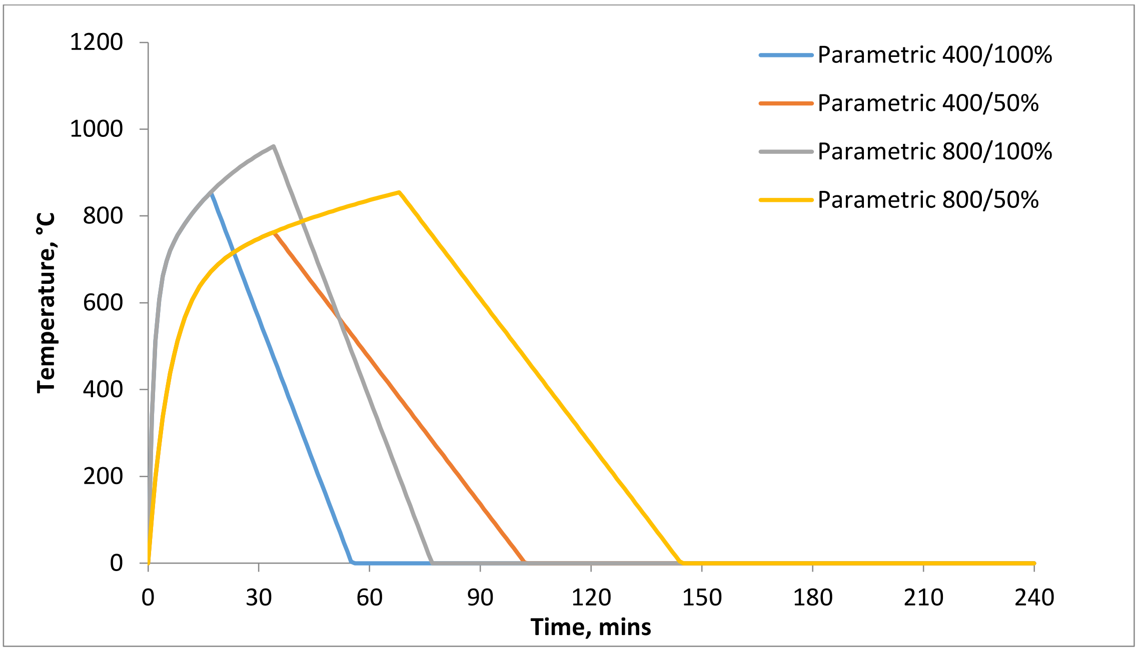


Figure 6. Parametric fires in a concrete enclosure.

The legend refers to fire loads of 400 or 800 MJ/m^2 and opening factors of 0.072 $m^{-1/2}$ (100%) and 0.036 $m^{-1/2}$ (50%).

The relative merits of using parametric fires are compared as follows.

Advantages

Variation of the parameters deliver a grouping of time-temperature curves such as shown in Figure 6 indicating maximum temperatures and durations.

- Fire load increases > longer duration and higher temperatures.
- Ventilation increases > fire grows faster but fuel is consumed quicker so duration is reduced, net result is higher temperature.

Disadvantages

- Assumed that the fire is ventilation controlled but may not be.

- Assumes that the total ventilation is available, i.e. all windows are broken, which is unlikely.
- The firecell size and geometry can be expected to have an influence, and floor area is limited to 500 m^2.

For example, in a fire in a large firecell such as an open-plan office (with a low ceiling), the contents are unlikely to burn all at once or in sufficient quantity to achieve flashover conditions. A more likely scenario is that a fire establishes itself relatively independently of the ventilation available, consuming instead some of the existing air within the compartment to get established, and then the resulting window breakage or not will dictate the course of the fire event.

A.3.3 Realistic fire scenarios

Acknowledging the shortfalls of the ISO and parametric fire exposure, real fires come into consideration, and the challenge is to select one that is appropriate to the building/compartment being considered.

In Figure 7, real fires taken from experimental trials with fully developed fires in small compartments are shown to be comparable with fires generated in computational fluid dynamics (CFD) models such as Fire Dynamics Simulator (FDS) (McGratten et al., 2009) in larger compartments. However, if ventilation is limited, the fire may stagnate and slowly move through a compartment at a relatively lower temperature. The significant factor is that temperatures well in excess of the ISO curve for periods of time of up to 30 minutes and as high as 1200–1300°C may be achieved for shorter times, and for structural consideration, this is more likely a worst-case scenario.

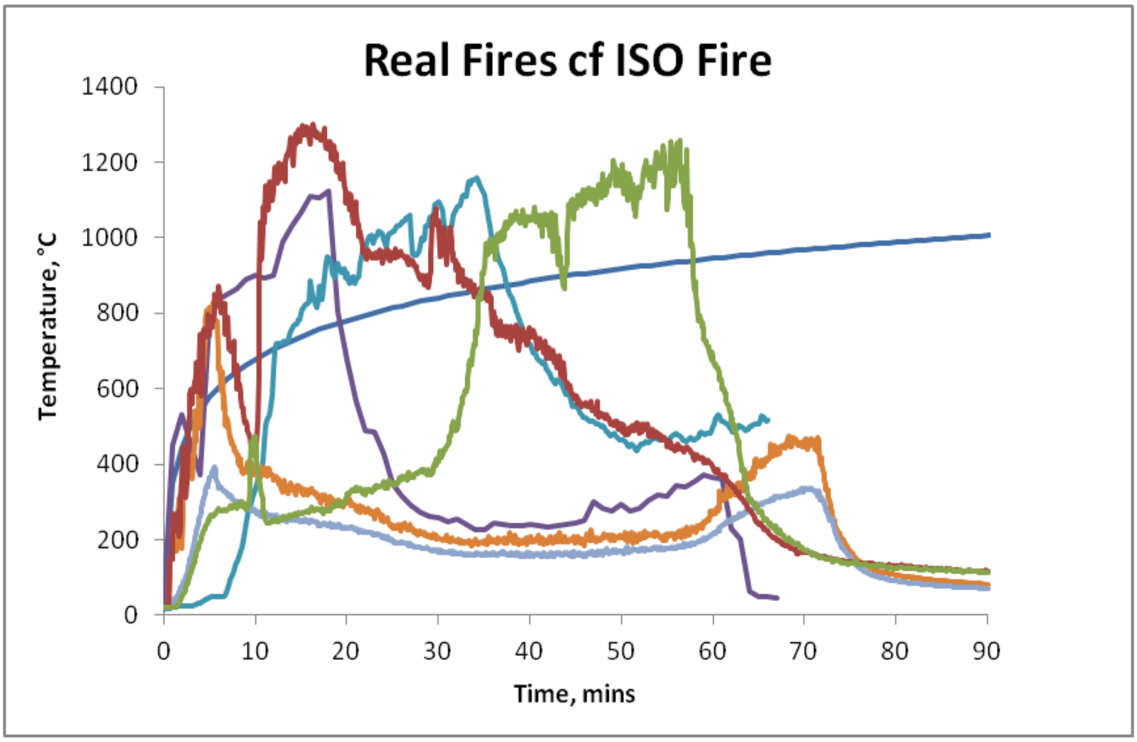


Figure 7. Real fires compared with the ISO fire.

A.3.4 Travelling fires (Stern-Gottfried, 2011)

The concept of travelling fire is a more likely scenario worth considering, especially for larger compartments with relatively low ceilings. The firecell contents may not all become involved at once, and the portion of contents that do burn are limited by the available air already in the firecell and the rate at which the ventilation increases with breaking windows introducing more air. It is then likely that the fire will move to unburned fuel, and it may move in more than one direction towards fresher air. It will be the conditions at the interface that dictate the rate at which the building contents actually burn.

The concept of travelling fires in large enclosures is introduced (Stern-Gottfried, 2011; (Stern-Gottfried, Rein and Torero, 2009) and acknowledges the possibility that full room fire involvement does not necessarily occur. Rather, a fire will move progressively through a compartment consuming fuel as it goes, and the temperature and heat flux from the upper layer away from the fire may not be sufficient to initiate ignition of other contents.

Acknowledging that no documented guidance for structural engineers to assess the impact of travelling fire behaviour is available, a method is proposed whereby the time-temperature exposure is an envelope conservatively enclosing the likely range exposure parameters for a given fire load (FLED) and floor area.

The proposed time-temperature exposure basically comprises of a relatively long duration exposure outside the vicinity of the fire (far field) and at some stage a stepwise increase in temperature (near field) as the fire passes through and then returns to the original temperature.

The travelling fire method (TFM) assumes the temperature in 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. A typical exposure is shown in Figure 8. The far field temperature depends on the percentage of the floor area that the fire occupies as it travels through the compartment.

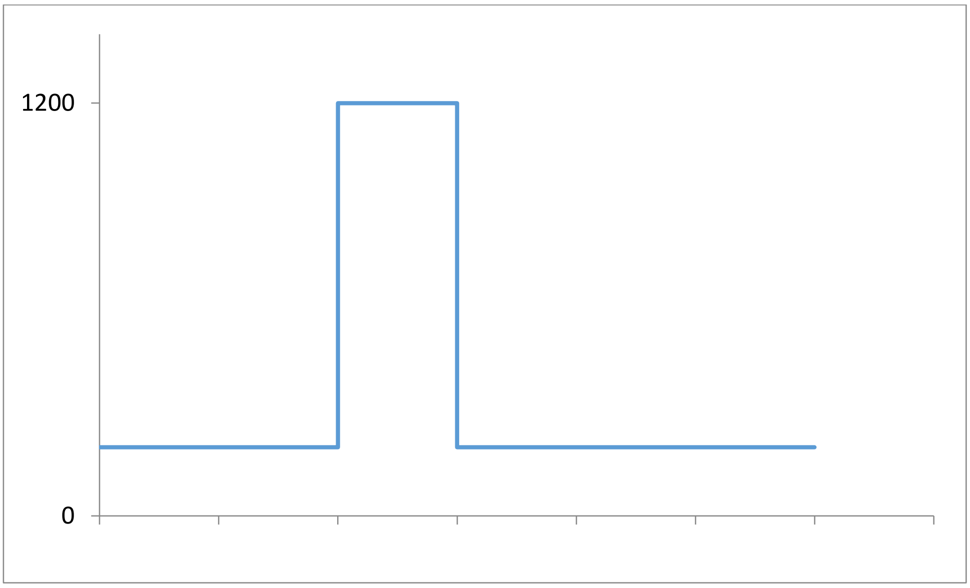


Figure 8. Temperature-time curve that a structural element may be exposed to.

EN 1991-1-2:2002 states that the parametric curves are only valid for compartments with floor areas up to 500 m^2 and heights up to 4 m. The enclosure must also have no openings through the ceiling, and the compartment linings are restricted to having a thermal inertia between 1000 and 2200 J/m^{2}s$^{1/2}$.K and for compartments with floor areas up to 500 m^2. The range of thermal inertia covers concrete except lightweight concrete where the density is ~ <1000 kg/m^3 and gypsum plasterboard on the basis that the water of hydration effectively contributes to an increased specific heat putting it into that range of thermal inertia, but highly conductive such as glass facades or insulating materials fall outside the range. Increasing the volume to surface area ratio with greater floor area means the boundary material properties have less influence.

The process for applying a travelling fire scenario as illustrated in Figure 9 and begins with defining a time-temperature that is applicable to the enclosure.

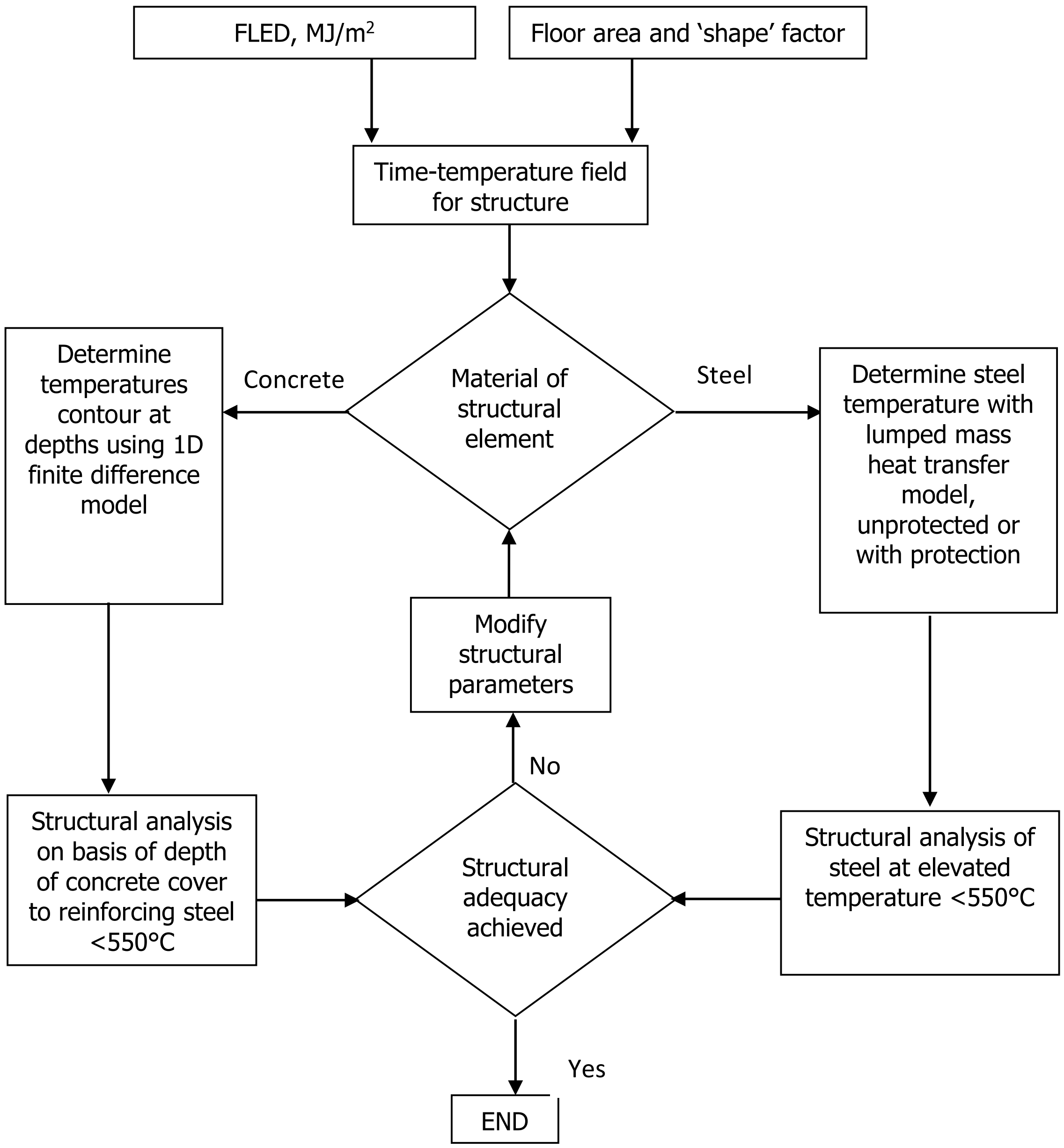


Figure 9. Flow chart for travelling fire design.

First, the floor area needs to be greater than 500 m^2, otherwise a parametric (EN 1991-1-2:2002) exposure is applicable on the assumption that 100% of the compartment is involved.

For floor areas greater than 500 m^2, a travelling fire methodology is applicable and the following input data and qualifications are required:

- FLED in MJ/m^2
- Floor area
  - Greater than 500 m^2
  - Shape factors
    - Open-plan
    - Ceiling height range 3–5 m

The travelling fire scenario lends itself to large aspect ratio enclosures with large floor areas and relatively low ceilings that inhibit total room involvement.

The process for selecting travelling fire parameters in Table 6 uses the assumption that the near field temperature is 1200°C and its duration is proportional to the FLED and independent of the percentage (%) of floor area involved.

The far field temperature and duration depends on the percentage of the floor area involved (in the near field) fire, while the duration is dependent on the FLED and the floor area.

Table 6. Travelling fire parameters.

FLED, MJ/m^2	**400**	**600**	**800**	**1200**
Near field temperature, °C	1200	1200	1200	1200
Near field duration, mins	14	21	28	42
Floor, m^2	**<500**	**>500**	**2000**	
% floor area	100	25	10	
Far field temperature, °C	1200	800	545	

Combined fire duration depends on FLED and % of floor area:

FLED, MJ/m^2 **% area**	**400**	**600**	**800**	**1200**
10%	140	210	280	420
25%	56	84	112	168
100%	14	21	28	42

Example 1: Determine the time-temperature exposure in an enclosure of 1000 m^2 with an FLED of 400 MJ/m^2:

- Near field temperature of 1200°C with a duration 14 minutes that is independent of the percentage floor involvement.
- Far field temperature with 25% floor area involved will have a duration of 42 minutes (56 minutes - 14 minutes) of 800°C around the near field exposure.

Figure 10 shows two alternative versions of fire exposure depending on when the fire (near field of 14 minute duration) arrives at 14 minutes (left) and 42 minutes (right).

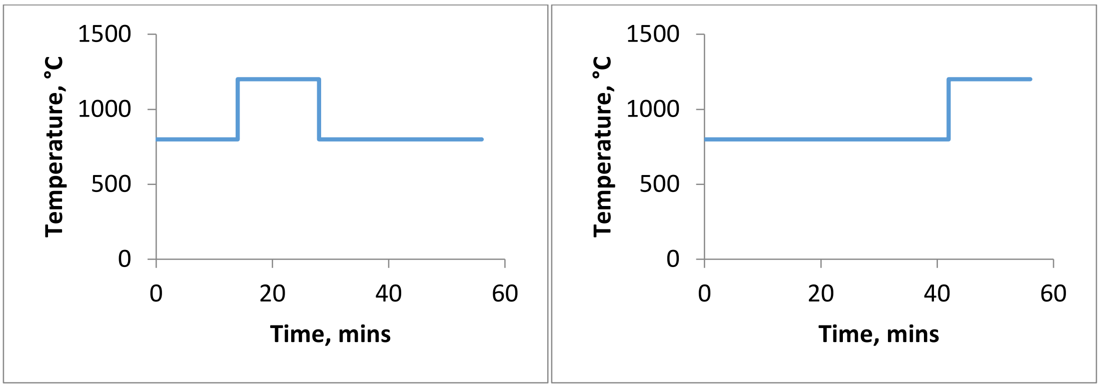


Figure 10. Travelling fire exposure 400 MJ/m² and 25% floor involvement in travelling fire.

Both are acceptable scenarios although the delayed arrival of the near field temperature would be expected to be a worst-case scenario on the basis that the structure has had a longer period of preheating at 800°C and will achieve a higher maximum temperature. For that reason, selection of the scenario with the latest arrival of the near field is likely to deliver a conservative result.

As another example, if it is considered that the floor area of the near field is reduced to 10%, the near field exposure remains the same at 1200°C for 14 minutes, but the far field exposure reduces in temperature to 545°C and the duration of the entire fire event increases to 140 minutes as shown in Figure 11.

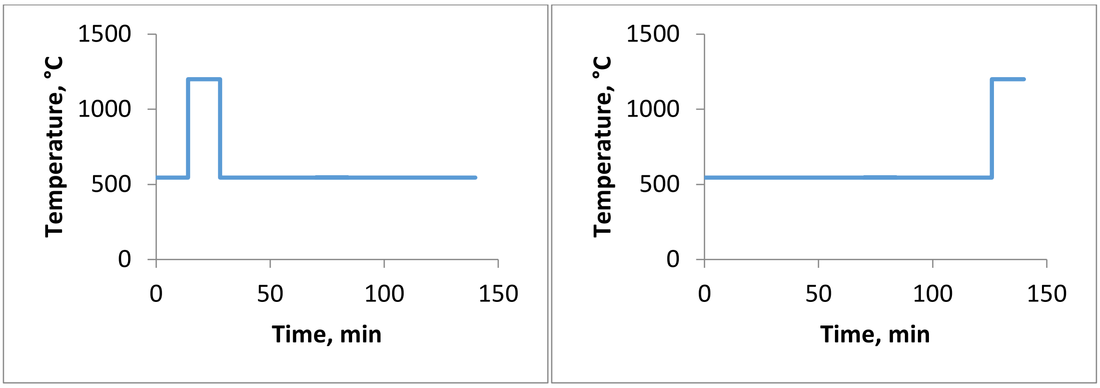


Figure 11. Travelling fire exposure 400 MJ/m² and 10% floor involvement in travelling fire.

Appendix B: Temperature response of structural elements

The thermal response of the structural elements to the (idealised) fire exposure above may be assessed using several different methods depending on the material and the simplicity or complexity required.

Concrete temperature (Incropera, DeWitt, Bergman and Levine, 2007)

Concrete temperatures at progressive depths from the exposed surface may be determined using a one-dimensional finite difference (1D FD) approach using radiant heat flux and convection on the surface and heat conduction with the material. The results of this calculation give the concrete temperature at progressive depths, and it is assumed that reinforcing steel at that depth will be at the same temperature. Structural adequacy may be achieved by ensuring that there is sufficient concrete cover to maintain the reinforcing steel below 550°C.

The one-dimensional finite difference method is applicable to slabs or essentially flat surfaces that are for the most part considered semi-infinite to the extent that heat loss from the non-fire side need not be included, although it can be if required if boundary conditions are calculated on the ambient side.

In situations where the exposure on a corner is considered, the principle of 'arris of rounding' (as for timber on corners) where the depth of contour is also the radius of the contour at the corner can be used.

Alternatively and for more complicated shapes in two or three dimensions such as beams and columns, proprietary finite element software such as SAFIR may be used to determine the temperature contours.

$$T_0^{t+1} = \frac{2\Delta t}{\rho c \Delta z}\left[h_0\left(T_g - T_0^t\right) + \sigma\varepsilon\left(T_g^4 - T_0^{t^4}\right) + \frac{k_c}{\Delta z}\left(T_1^t - T_0^t\right)\right] + T_0^t \qquad \text{Eq.(4)}$$

$$T_i^{t+1} = F_o\left(T_{i+1}^t + T_{i-1}^t\right) + (1 - 2F_o)T_i^t \qquad \text{Eq.(5)}$$

$$T_n^{t+1} = \frac{2\Delta t}{\rho_c c_c \Delta z}\left[h_n\left(T_\infty - T_n^t\right) + \sigma\epsilon\left(T_\infty^4 - T_n^{t^4}\right) + \frac{k_c}{\Delta z}\left(T_{n-1}^t - T_n^t\right)\right] + T_n^t \qquad \text{Eq.(6)}$$

Where,

T_i^t : concrete temperature at time t, and location i(k) – a subscript of '0' indicates the exposed surface and 'n' the backside surface

T_g : gas temperature (K)

T_∞ : ambient temperature (293K)

ρ_c : density of concrete (2300 kg/m^3)

c_c : specific heat of concrete (1000 J/kg-K)

h : convective heat transfer coefficient (25 W/m^2-K for exposed surface and 4 W/m^2-K for the ambient surface)

σ : the stefan-Boltzmann constant $(5.67x10^{-8}W/m^2K^4)$

ε : radiative and reradiative emissivity of the material and gas combined

$\mathrm{k_c}$: thermal conductivity of concrete (1.3 W/m-K)

$\Delta\mathrm{z}$: element length

$\mathrm{F_o}$: the Fourier number as below

$$F_o = \frac{k_c \Delta t}{\rho_c c_c \Delta z^2} \qquad \text{Eq.(7)}$$

Steel temperature (Buchanan, 2001)

For (unprotected) steel members, a lumped mass heat transfer (LMHT) calculation may be used to determine the steel temperature. This method assumes that the cross-section temperature is uniform on the basis that the conductivity is sufficiently high compared with the boundary conditions. The essential input parameters are the heated perimeter per area (H_p/A) ratio that is obtainable from tables of steel section properties for 3 and 4-sided exposure and the convection and radiant flux to the surface.

$$\Delta T_s = \frac{H_p}{A}\frac{1}{\rho c}\left[h_c(T_g - T_s) + \sigma\epsilon(T_g^4 - T_s^4)\right]\Delta t \qquad \text{Eq.(8)}$$

Where,

T_s : steel temperature (K)

T_g : gas temperature (K)

H_p : heated perimeter of the beam

A : cross-section of the beam

ρ : density of steel (7850 kg/m^3)

c_s : temperature-dependent specific heat of steel (J/kg-K)

h_c : convective heat transfer coefficient (25 W/m^2)

σ : the stefan-Boltzmann constant $(5.67x10^{-8}W/m^2K^4)$

ε : radiative and reradiative emissivity of the material and gas combined

Δt : time step

For protected steel, the properties of the applied insulation are included, and the H_p/A ratio may be different if the protection is boxed around the member as opposed to directly applied to all surfaces.

$$\Delta T_s = \frac{H_p}{A}\frac{k_i}{d_i \rho_c c_s}\frac{\rho_s c_s}{\left[\rho_s c_s + (H_p/A) d_i \rho_i c_i / A\right]}(T_g - T_s)\Delta t \qquad \text{Eq.(9)}$$

Where,

k_i : thermal conductivity of the insulation (0.12 W/m K)

d_i : thickness of the insulation(m)

ρ_i : density of insulation (550 kg/m^3)

c_i : specific heat of the insulation (1200J/kg-k)

The convection and radiation terms that were included in Eq.(8) become insignificant by comparison with the resistance to heat transfer of the applied insulation. Structural adequacy may be considered to be achieved if the temperature of the steel is maintained below 550°C for the duration of the fire scenario.

A series of fires ranging from parametric to travelling fires with open vents and travelling fires with progressive window breakage are examined to determine the temperature response of structural elements. The following sequence of examples considers a hypothetical open-plan office scenario to demonstrate how variations in the FLED and ventilation influence the fire temperatures and thermal response of structural elements. A single floor is considered measuring 36 x 24 x 4 m (h) with windows 1 x 2 m (h) around the perimeter as shown in Figure 12.

B.1 Parametric fires

The temperature response of structural elements is considered for a series of parametric fires.

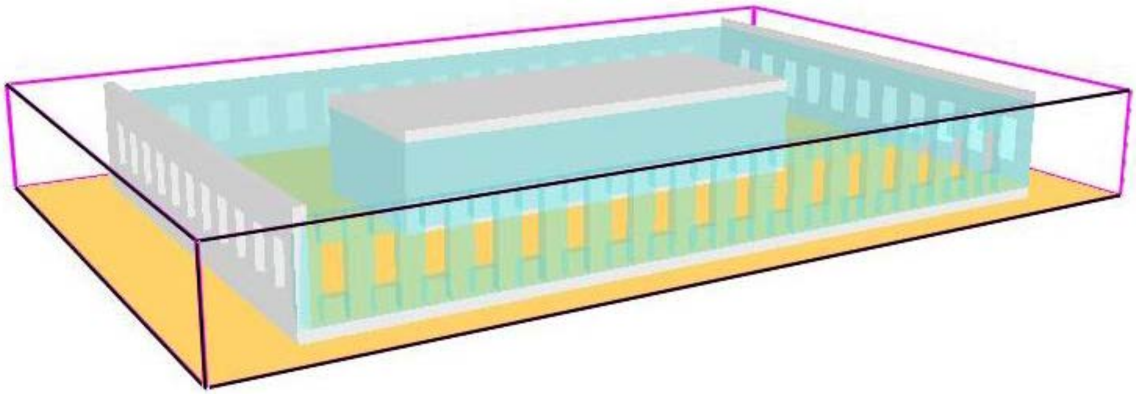


Figure 12. Single floor of multi-storey office building with central service core, 100% ventilation from FDS simulation.

Ventilation is considered in two scenarios, vents 1 x 2 m (h) 56 in total around the perimeter (100%) and 28 (50% ventilation condition) as summarised in Table 7.

Table 7. Scenarios considered

FLED/ventilation	Parametric fire		FDS – moving fire	
	100% (56)	50% (28)	100% (56)	50% (28)
Ventilation factor, m$^{3/2}$	158.4	79.2	158.4	79.2
Opening factor, m$^{-1/2}$	0.072	0.036	0.072	0.036
400 MJ/ m^2	*	*	*	*
800 MJ/ m^2	*	*	*	*

The parametric fire exposure curves (EN 1991-1-2:2002) generated for the FLED are shown in Figure 13 for the condition where the nominated ventilation is the condition from the beginning. The duration of fire exposure is determined by a combination of the fuel (FLED) available to burn and the ventilation condition, which governs the rate at which the fuel is consumed through the control of combustion air. The temperature reached in the firecell is determined by the rate of combustion, the net exchange of hot and cold gases through the vent and the thermal inertia of the boundary materials, which are all concrete.

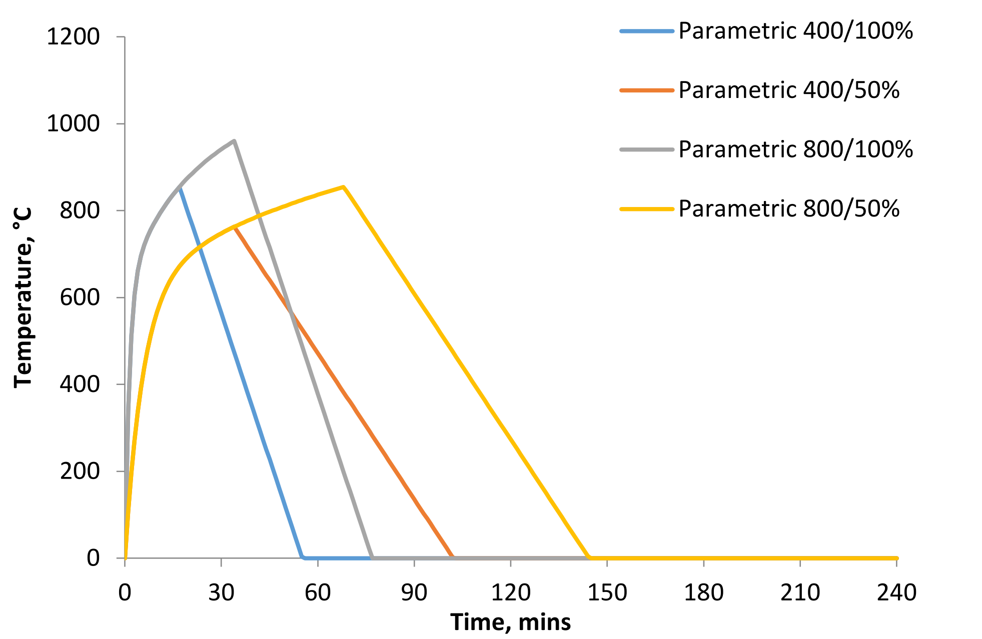


Figure 13. Parametric fire temperatures at FLED/ventilation%.

Higher temperatures are achieved with greater ventilation where it is assumed that all fuel is immediately available for combustion. This may not be the case if, for instance, initial layers need to burn off to expose lower layers of the same fuel that would then be a fuel controlled fire.

Even with the most extreme limit of open ventilation, the maximum temperature reached is in the order of 1200°C within the firecell even if it is only of very short duration. In practical terms, this is unlikely to be a realistic scenario, except perhaps with a terrorist attack where finely dispersed jet fuel is suddenly introduced to the firecell and ignited.

For practical considerations of fire design, some educated judgement is required to select a likely time-temperature exposure.

B.1.1 Temperature response of concrete to parametric fires

A range of simple modelling trials using a simple spreadsheet based on a one-dimensional finite difference model was used to compare the depths of temperature penetration into a standard/generic mix 2200 kg/m^3 concrete. Alternatively, a proprietary finite element package such as SAFIR or TASEF may be used to determine the temperature contours by entering the desired time-temperature curve.

Temperatures at 5 mm depth intervals in the concrete for the four parametric exposure conditions are shown in Figure 14 to Figure 17.

Of importance is the depth of any reinforcing steel and that there is sufficient cover (of concrete) to ensure that its temperature does not exceed specified code values.

The duration of the fire exposure has as much or maybe more influence on the temperature reached at greater depths (of say 25 mm) compared with a higher fire temperature for a shorter duration. In Figure 15 and Figure 17, for the 50% ventilation condition, the maximum temperature at 25 mm depth is greater than the 100% ventilation case in Figure 14 and Figure 16 where the fire temperature is higher but of shorter duration.

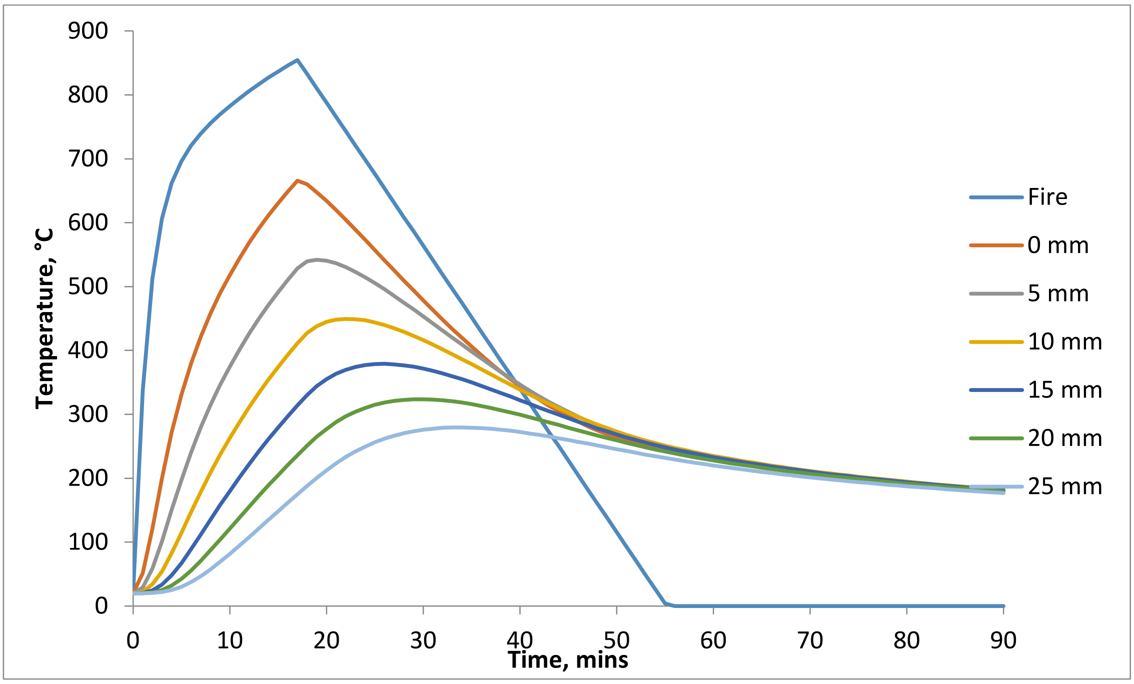


Figure 14. Temperatures in concrete at depths in mm for FLED 400 MJ/m^2 and 100% openings.

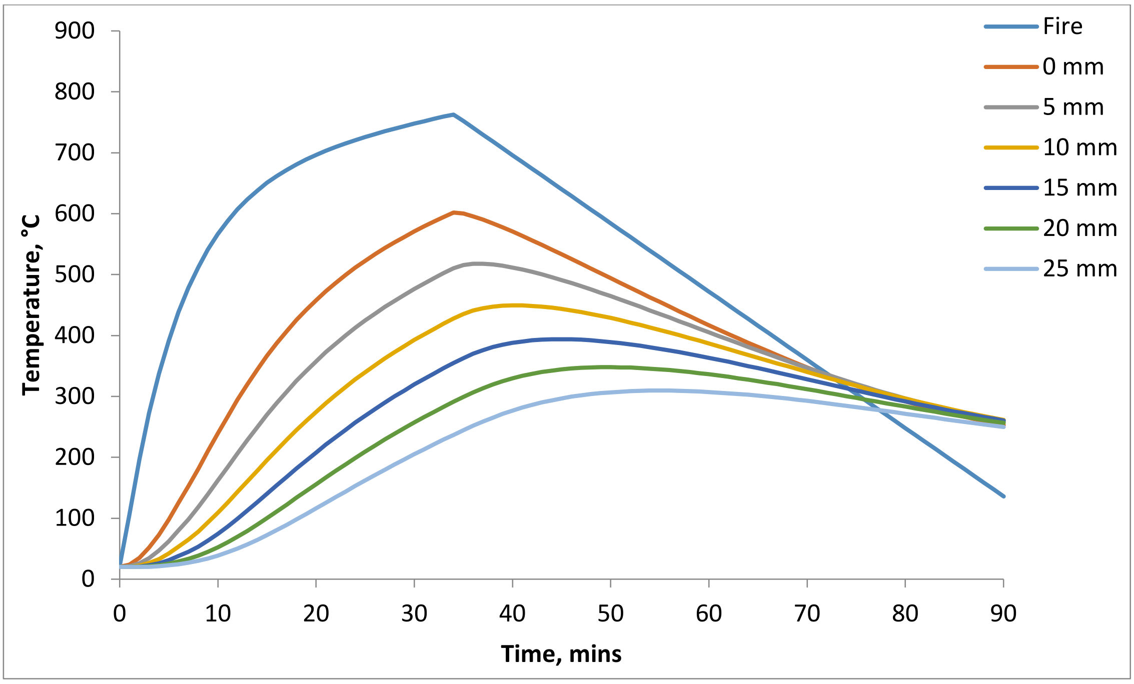


Figure 15. Temperatures in concrete at depths in mm for FLED 400 MJ/m² and 50% openings.

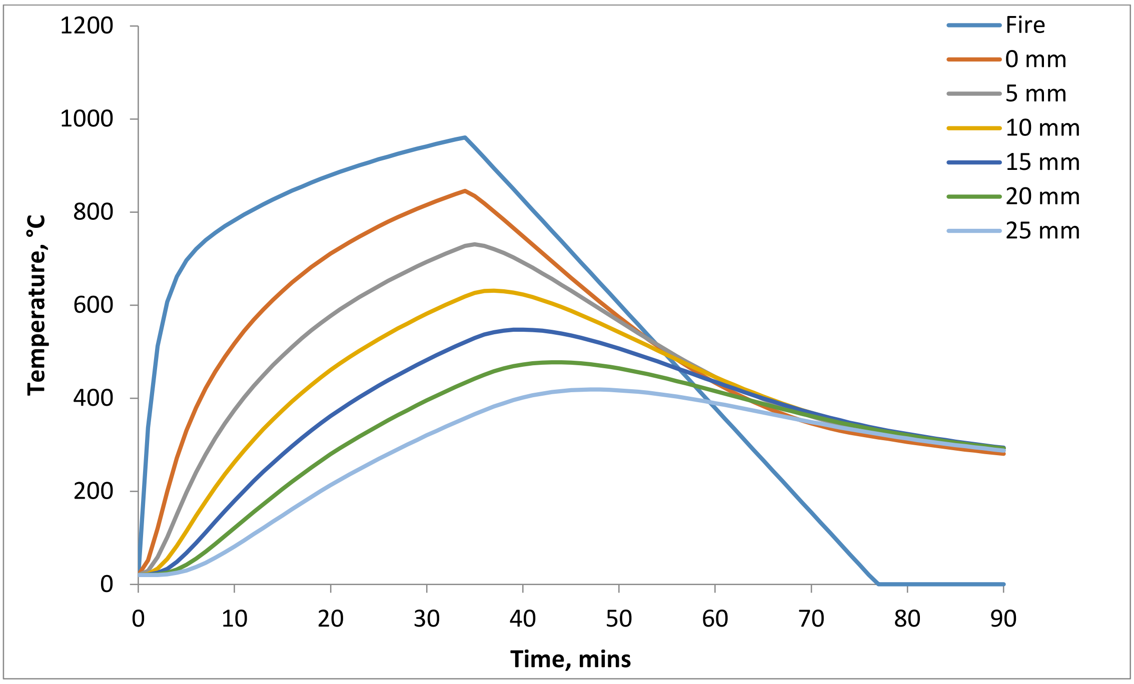


Figure 16. Temperatures in concrete at depths in mm for FLED 800 MJ/m² and 100% openings.

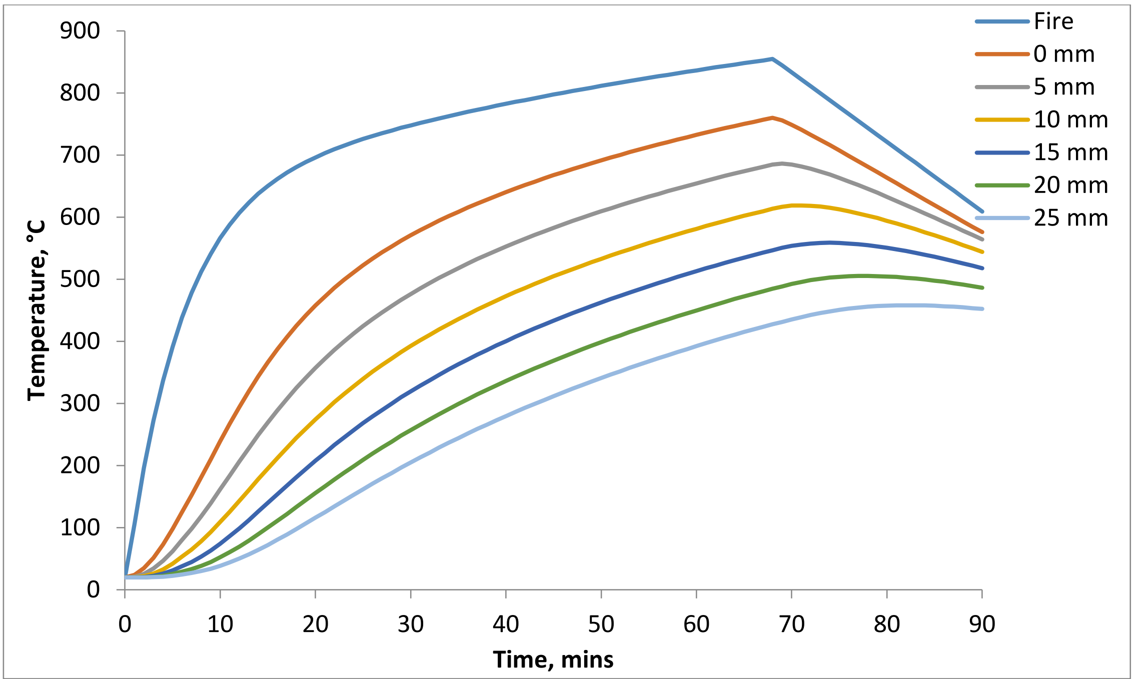


Figure 17. Temperatures in concrete at depths in mm for FLED 800 MJ/m² and 50% openings.

The maximum concrete temperatures as calculated by a 1D finite difference method are shown in Table 8. Temperatures exceeding the permitted 550°C are in bold and indicate 20 mm of cover would be required for an FLED of 800 MJ/m².

Table 8. Maximum concrete temperatures.

Fire scenario FLED/ ventilation	Max. fire temperature °C	Concrete temperature °C @ depth mm					
		0 mm	5 mm	10 mm	15 mm	20 mm	25 mm
Parametric 400/100%	855	**666**	543	450	379	324	280
Parametric 400/50%	760	**602**	518	449	394	348	310
Parametric 800/100%	956	**846**	**731**	**632**	548	478	419
Parametric 800/50%	855	**760**	**687**	**619**	**559**	505	458

B.1.2 Temperature response of steel to parametric fires

Similar to concrete, the temperature of steel exposed to fire rises in response to the fire environment and in accordance with the thermal inertia $\sqrt{k\rho c}$. However, due to the significantly greater thermal conductivity k compared with concrete, a simplifying assumption that the entire steel cross-section is at the same temperature is required for a simplified lumped mass model that can be used for initial evaluations of the steel temperature. The temperature of the steel is also dependent on the heated perimeter to area ratio (H_p/A) or surface area to volume ratio, which is directly proportional to the rate that heat is transferred to the steel. Any applied fire protection is intended to slow that process.

Some examples of the temperature response of a selection of steel members unprotected and protected with 13 mm of sprayed mineral fibre to the four parametric fires were examined. Three steel sections are considered with H_p/A ratios of 30, 60 and 130 broadly covering a heavy to lightweight range that can be more precisely related to the section in Table 9.

Table 9. Nominal steel sections and H_p/A for 3-sided exposure.

Universal beams	H_p/A	Universal beams	H_p/A	Universal beams	H_p/A
686 x 254, 125 kg/m	130	914 x 419, 388 kg/m	60	-	30
610 x 229, 113 kg/m	130				
533 x 210, 101 kg/m	130				
Columns 3-sided		**Columns 3-sided**		**Columns 3-sided**	
203 x 203, 60 kg/m	130	254 x 254, 167 kg/m	60	356 x 406, 551 kg/m	30

Considering that 550°C is the critical temperature for hot rolled steel above which the loadbearing properties decrease below the 70% level, the objective is to maintain steel temperatures below that level. The maximum temperatures reached are summarised in Table 10.

Table 10. Maximum steel temperatures for unprotected and protected sections.

Fire scenario FLED/ ventilation	Max. fire temperature °C	Steel H_p/A=30	Steel H_p/A=60	Steel H_p/A=130
		unprot/prot	unprot/prot	unprot/prot
Parametric 400/100%	855	395/83	**600**/133	**783**/217
Parametric 400/50%	763	460/117	**628**/186	**738**/291
Parametric 800/100%	961	**700**/73	**893**/205	**943**/332
Parametric 800/50%	855	**741**/89	**833**/265	**848**/421

Individual temperatures of the steel sections are illustrated in Figure 18 to Figure 29.

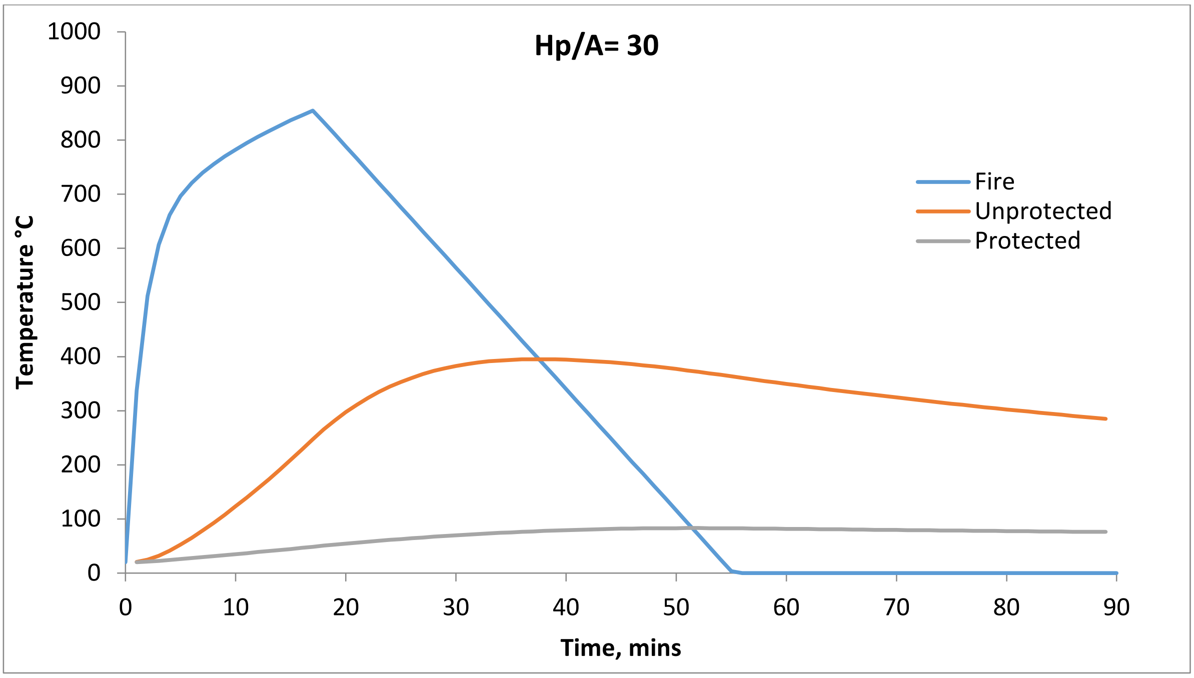


Figure 18. Temperatures in steel member (H_p/A 30) for FLED 400 MJ/m² and 100% openings.

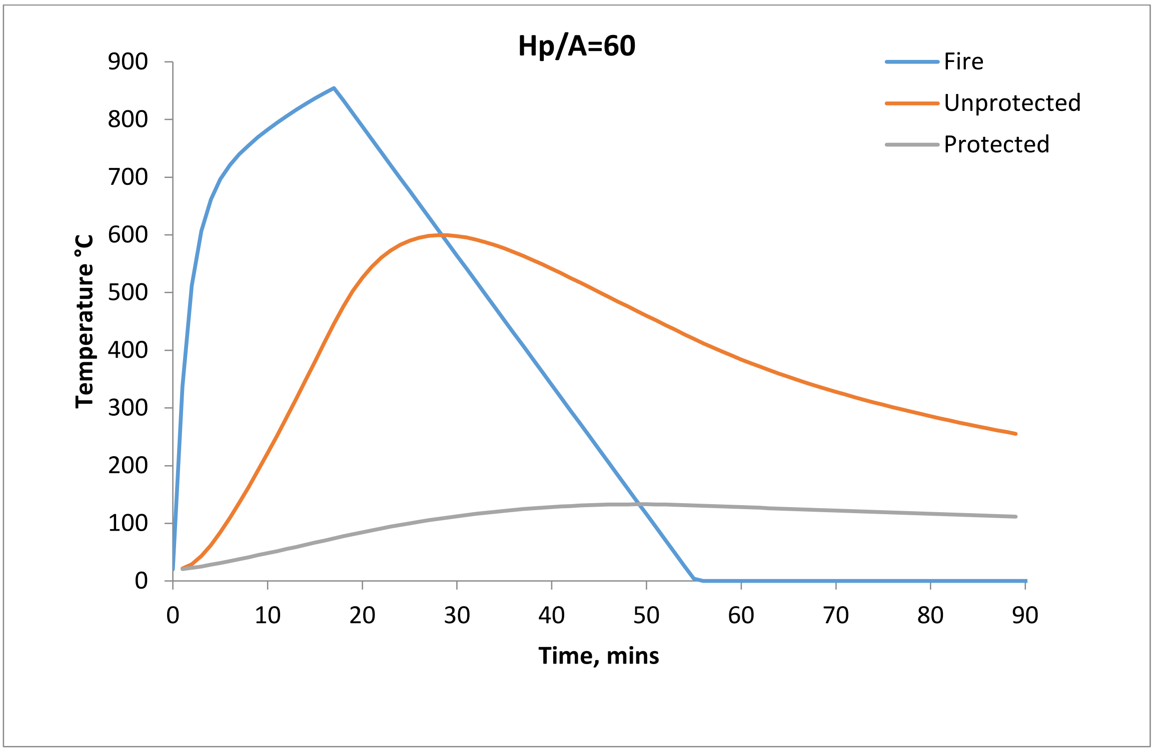


Figure 19. Temperatures in steel member (H_p/A 60) for FLED 400 MJ/m² and 100% openings.

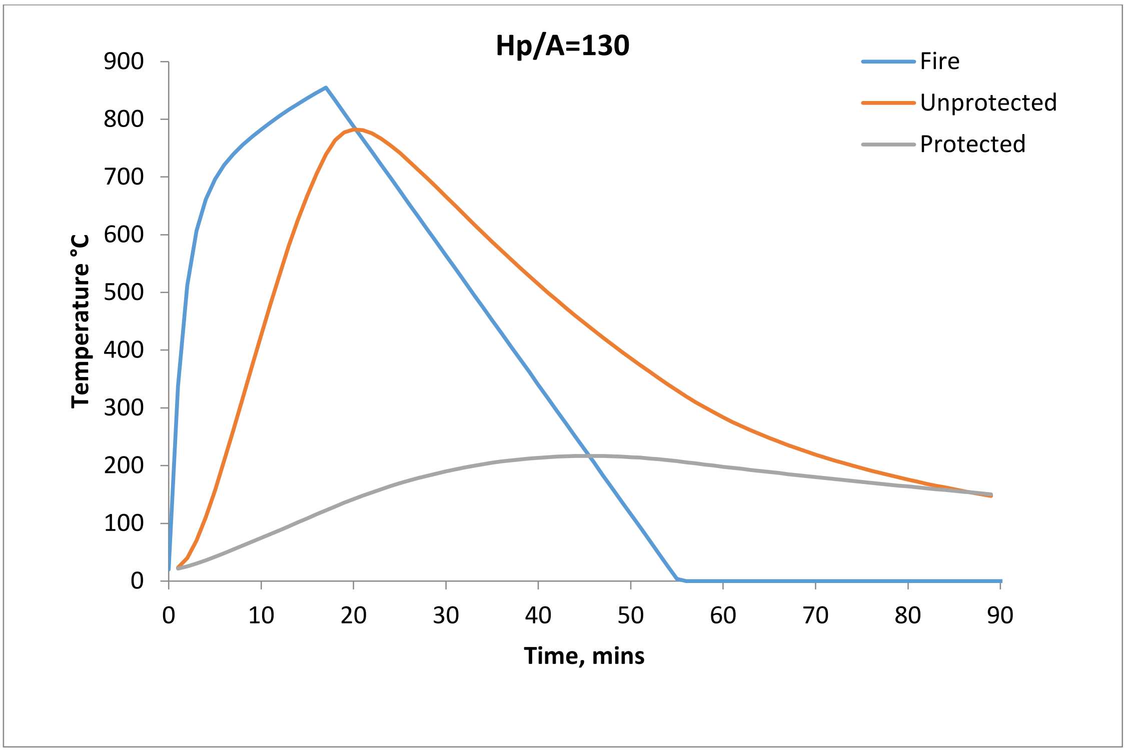


Figure 20. Temperatures in steel member (H_p/A 130) for FLED 400 MJ/m² and 100% openings.

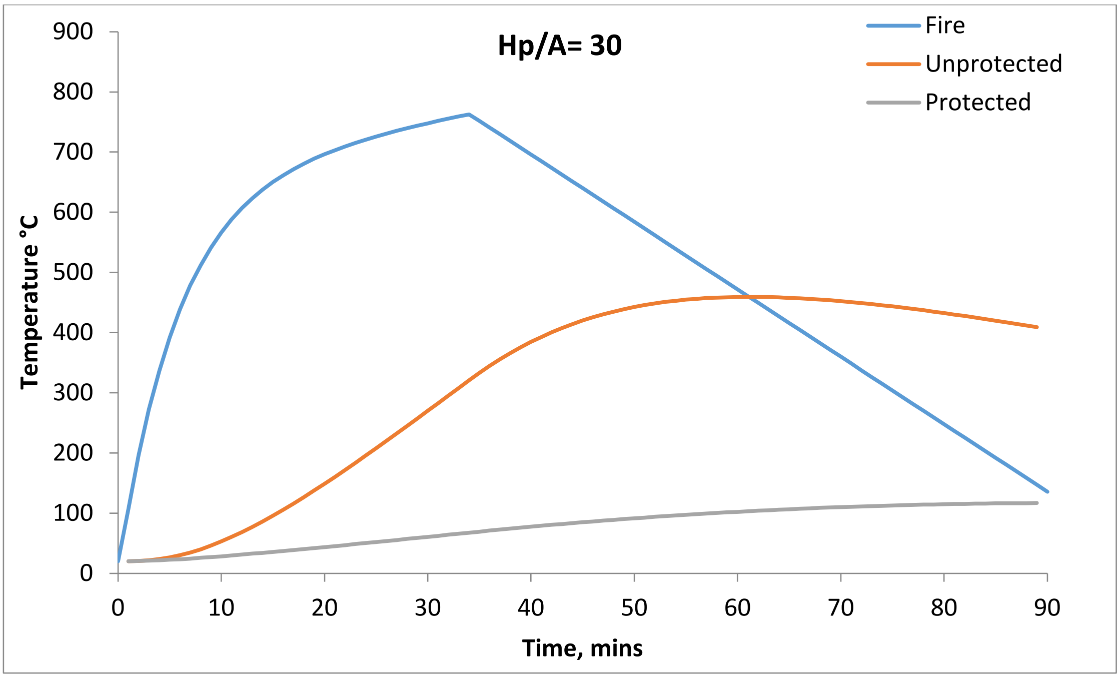


Figure 21. Temperatures in steel member (H_p/A 30) for FLED 400 MJ/m² and 50% openings.

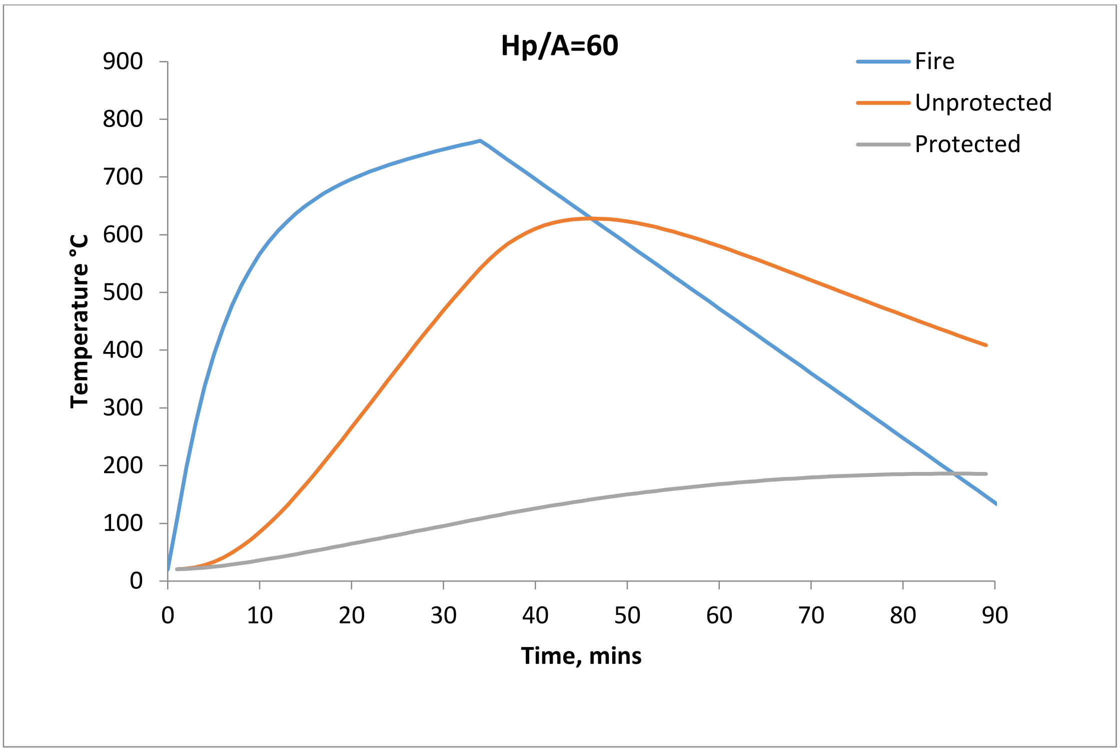


Figure 22. Temperatures in steel member (H_p/A 60) for FLED 400 MJ/m^2 and 50% openings.

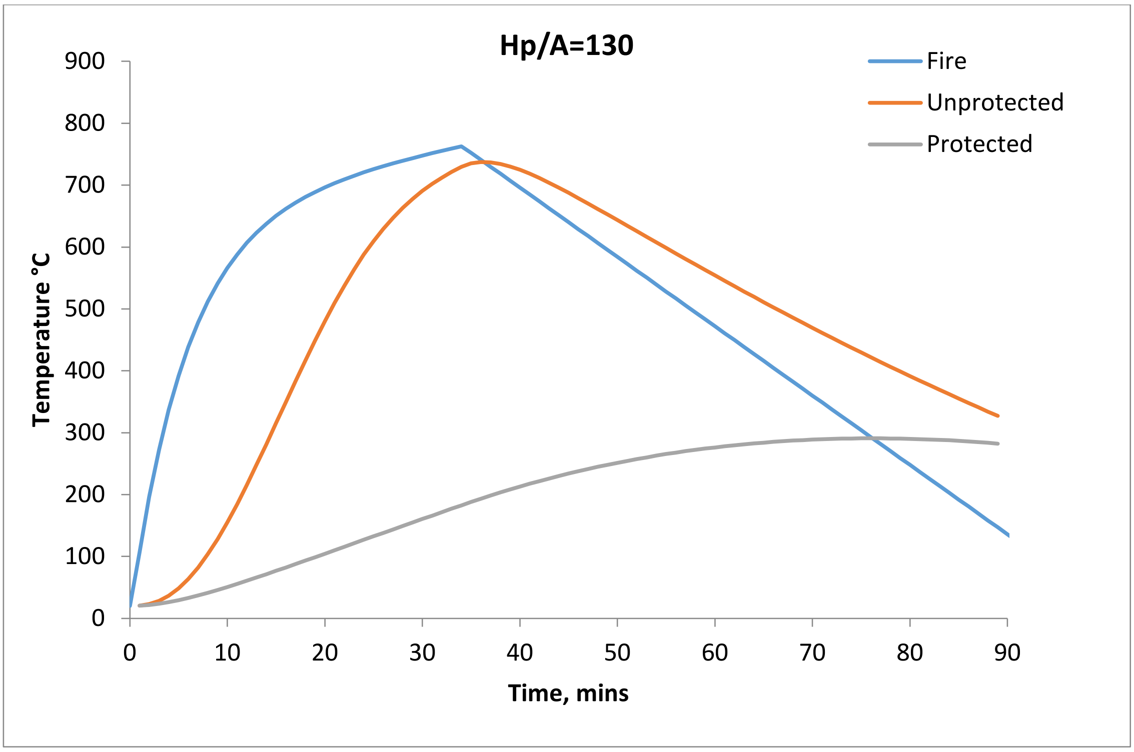


Figure 23. Temperatures in steel member (H_p/A 130) for FLED 400 MJ/m^2 and 50% openings.

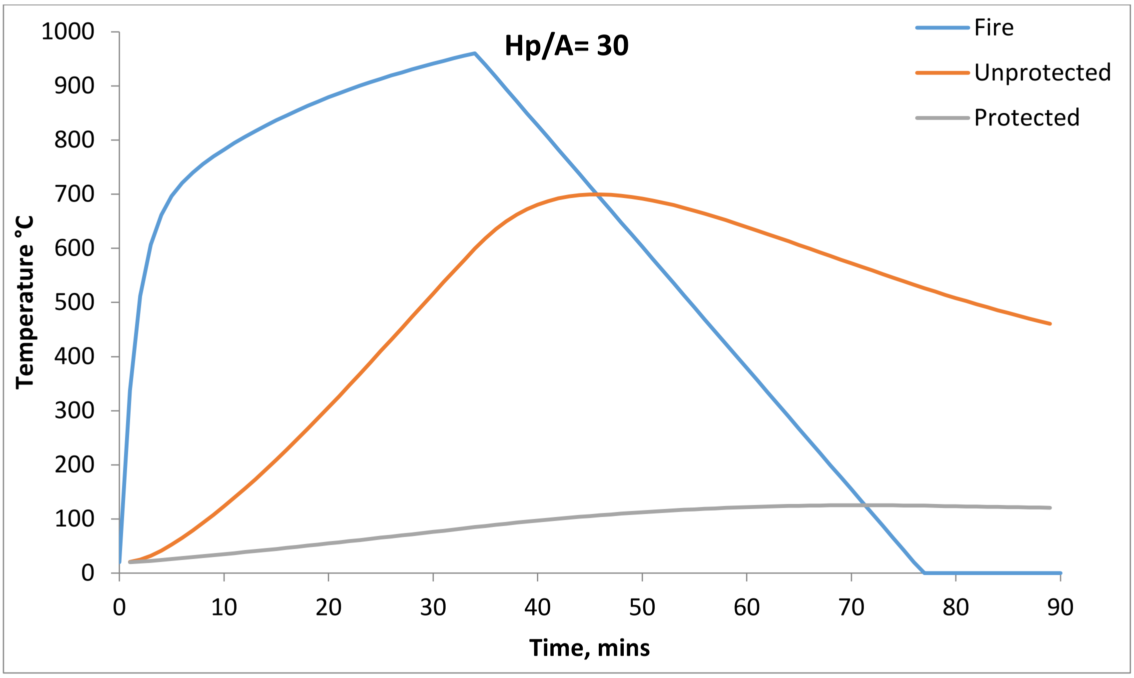


Figure 24. Temperatures in steel member (H_p/A 30) for FLED 800 MJ/m^2 and 100% openings.

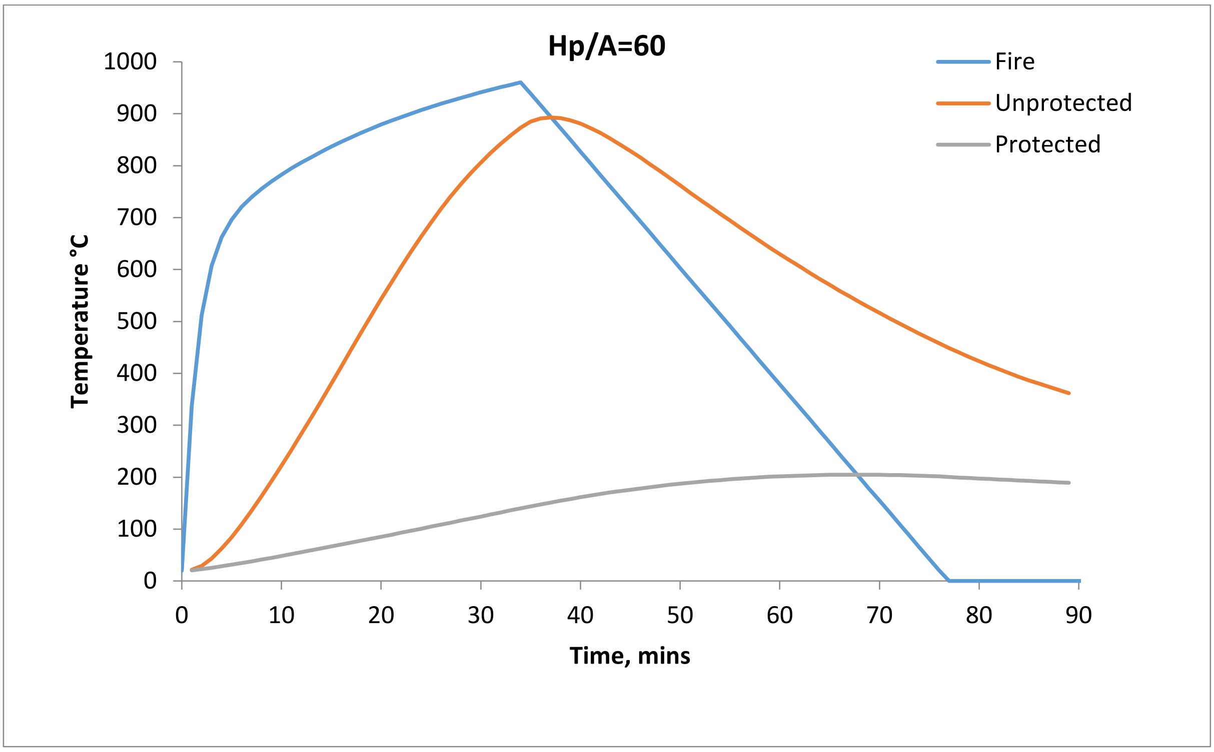


Figure 25. Temperatures in steel member (H_p/A 60) for FLED 800 MJ/m^2 and 100% openings.

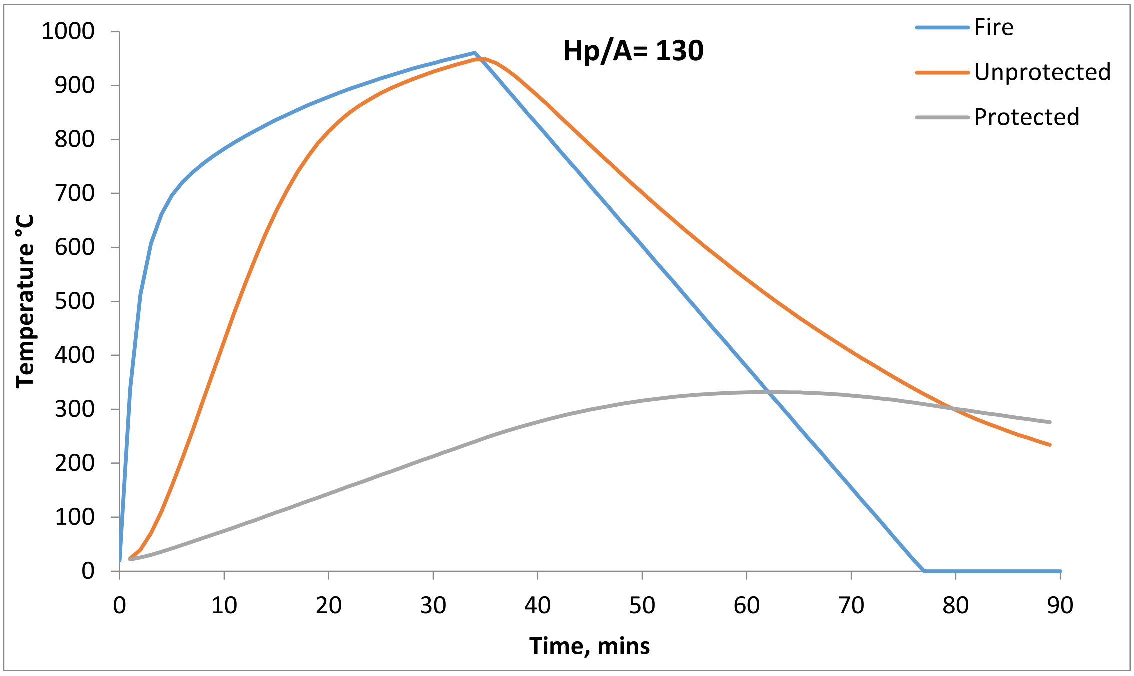


Figure 26. Temperatures in steel member (H_p/A 130) for FLED 800 MJ/m^2 and 100% openings.

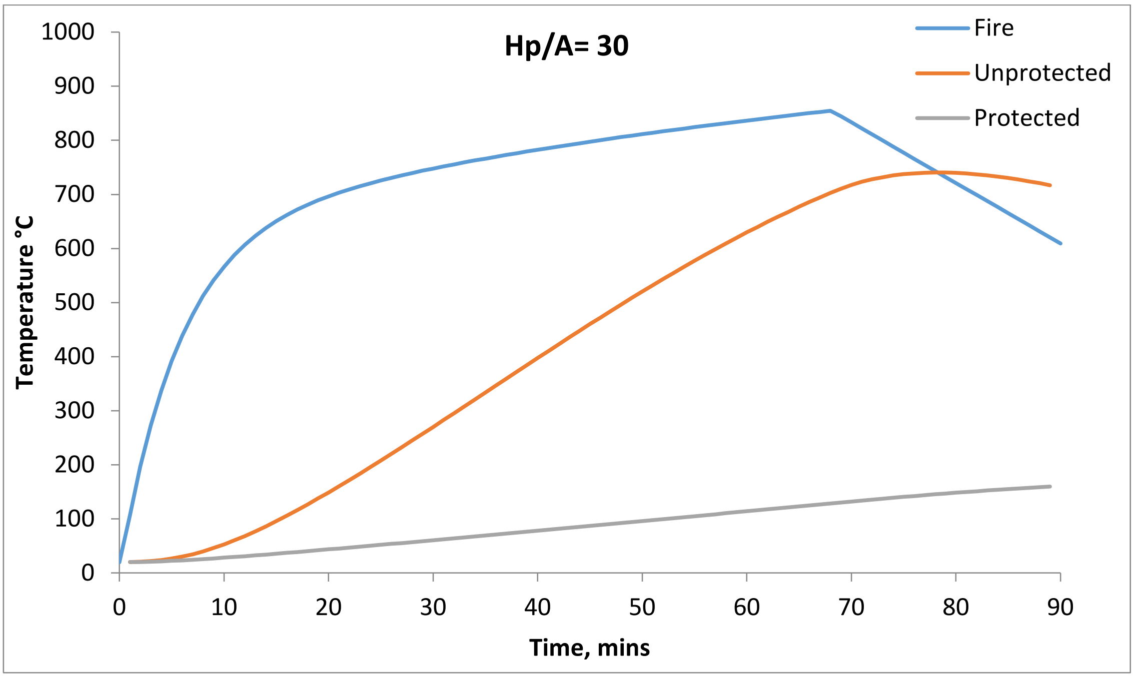


Figure 27. Temperatures in steel member (H_p/A 30) for FLED 800 MJ/m^2 and 50% openings.

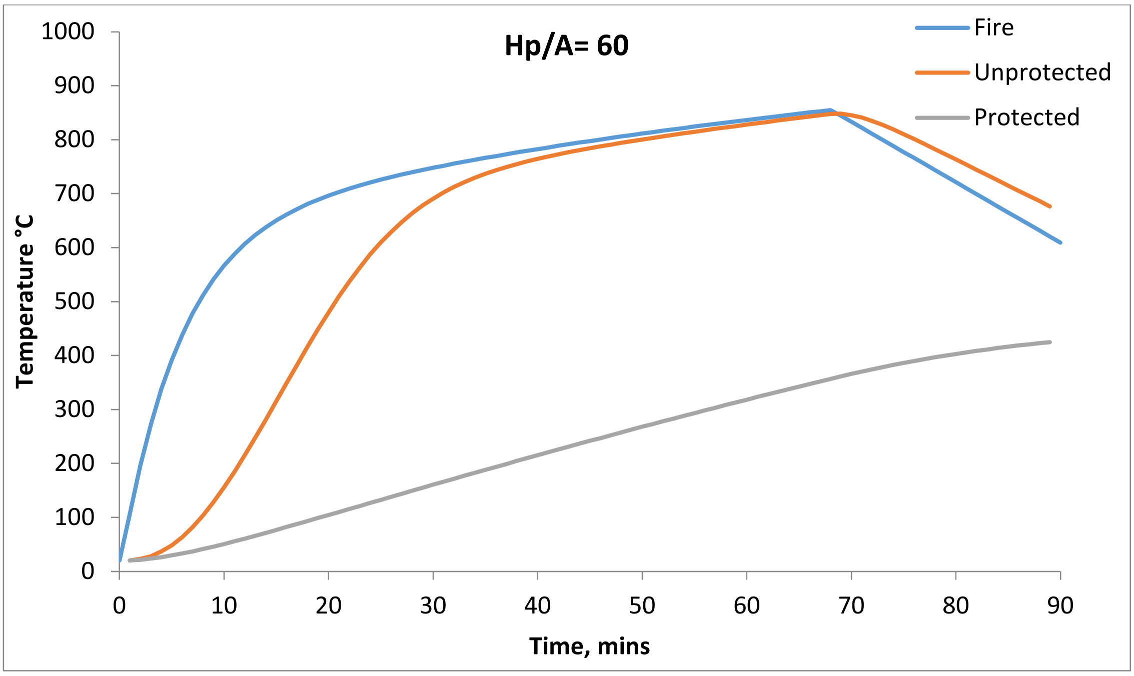


Figure 28. Temperatures in steel member (H_p/A 60) for FLED 800 MJ/m² and 50% openings.

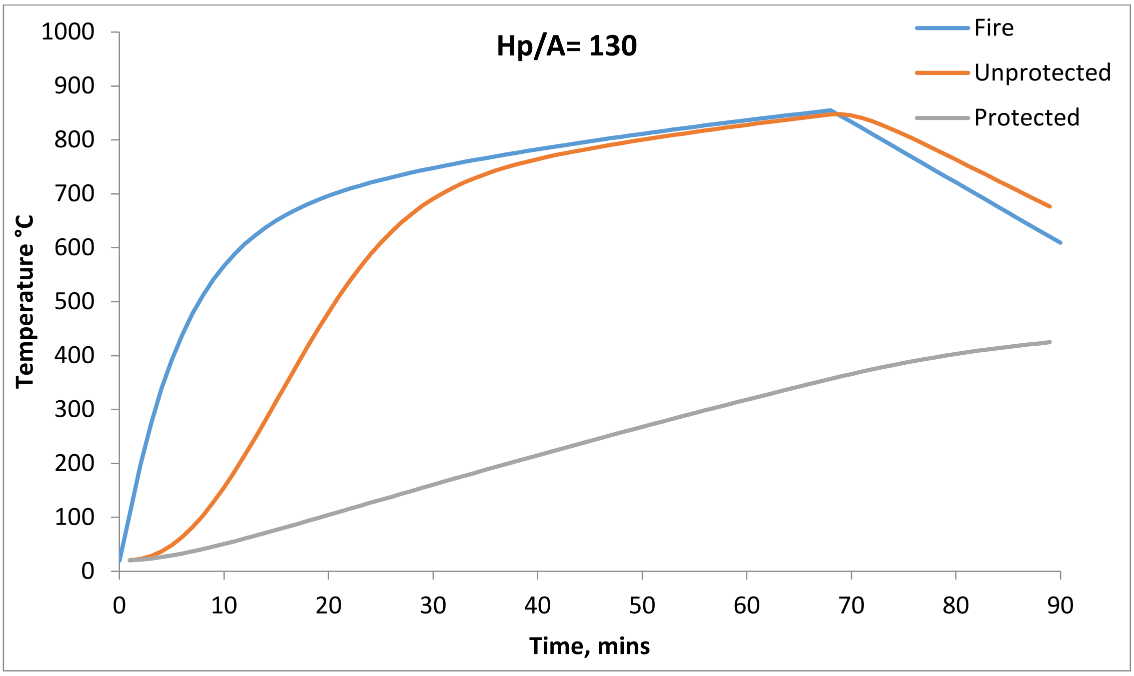


Figure 29. Temperatures in steel member (H_p/A 130) for FLED 800 MJ/m² and 50% openings.

The parametric fire results are summarised in Table 11 where temperatures exceeding 550°C are in bold.

Table 11. Summary maximum temperatures.

Scenario FLED/ ventilation	Fire temperature	Concrete @ 25 mm	Steel $H_p/A=30$ upper layer	Steel $H_p/A=60$ upper layer	Steel $H_p/A=130$ upper layer
Parametric 400/100%	855	280	395/83	**600**/133	**783**/217
Parametric 400/50%	763	310	460/117	**628**/186	**738**/291
Parametric 800/100%	961	419	**700**/73	**893**/205	**943**/332
Parametric 800/50%	855	458	**741**/89	**833**/265	**848**/421

B.1.3 Parametric fire summary

By using parametric fire scenarios based on the compartment fire load, geometry, bounding materials and ventilation conditions, the resulting time-temperature exposures can be used to assess the temperature response of structural elements.

B.2 Travelling fires (FDS modelling)

In the previous section, parametric fires based on the FLED and ventilation for an open-plan office were shown to raise the temperature of concrete to a level where a minimum of 20 mm cover to reinforcing was required.

In the case of the steel sections, generally some protection would be required except in the case of the lower FLED of 400 MJ/m^2 and the larger $H_p/A=30$. The addition of protection was shown to be capable of keeping steel temperatures below 550°C for the three section sizes.

The question arises as to whether the parametric fire assumption of whole firecell compartment involvement is valid. A more likely scenario is the concept of a moving (travelling) fire that in turn subjects regions of a structure to severe fire conditions such as in the direct fire plume and then that plume moves in the direction of fresh fuel allowing that part of the structure to cool.

The concept of a moving fire is compared with the previous parametric analysis using a methodology based on an energy balance such that the FLED in MJ/m^2 for the compartment is consumed in both the parametric fire scenario and the FDS simulation (McGratten et al., 2009) of the fire as it moves through the compartment.

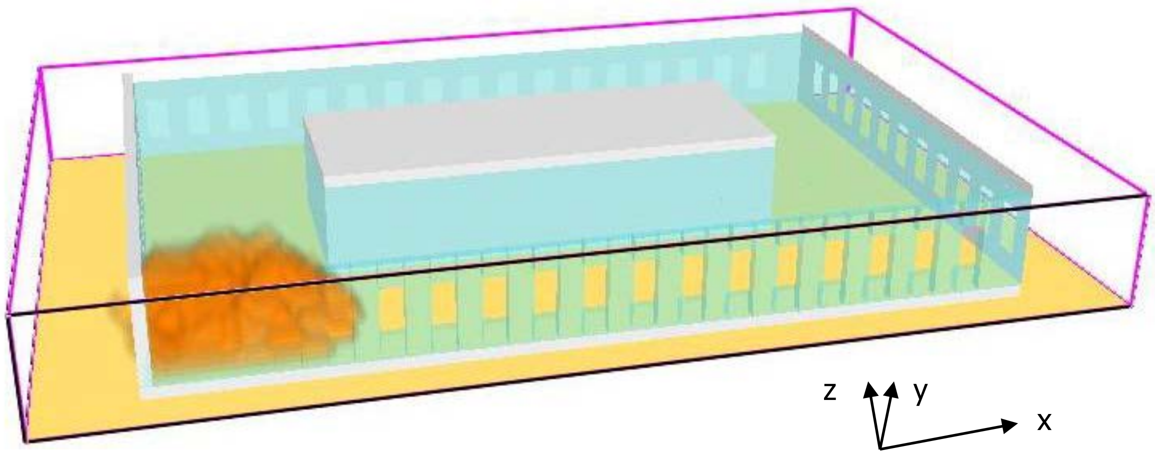


Figure 30. Moving fire on office floor at 300 seconds.

FDS modelling of a moving fire scenario is shown in Figure 30 at 300 seconds. The fire starts in the near left corner and moves according to radial spread until it reaches an obstruction such as a wall. The fire moves on two fronts in opposite directions around the perimeter. For an initial scenario, all vents are considered open at the start of the fire event rather than progressively breaking as the fire moves, which is considered later.

The x, y, z coordinates of the building floor are (0, 0, 0) on the floor at the left-hand near corner (internal). The volume outside the building is a control space to illustrate flame and smoke from vents.

Given the essential consideration is that the energy released by the moving fire is reconciled with the FLED in the compartment, some experimentation with the rate of fire growth and movement is required to achieve an energy balance. A family of moving fire HRR curves as generated by FDS for the four parametric equivalent fires are shown in Figure 31 for the case with all vents open initially.

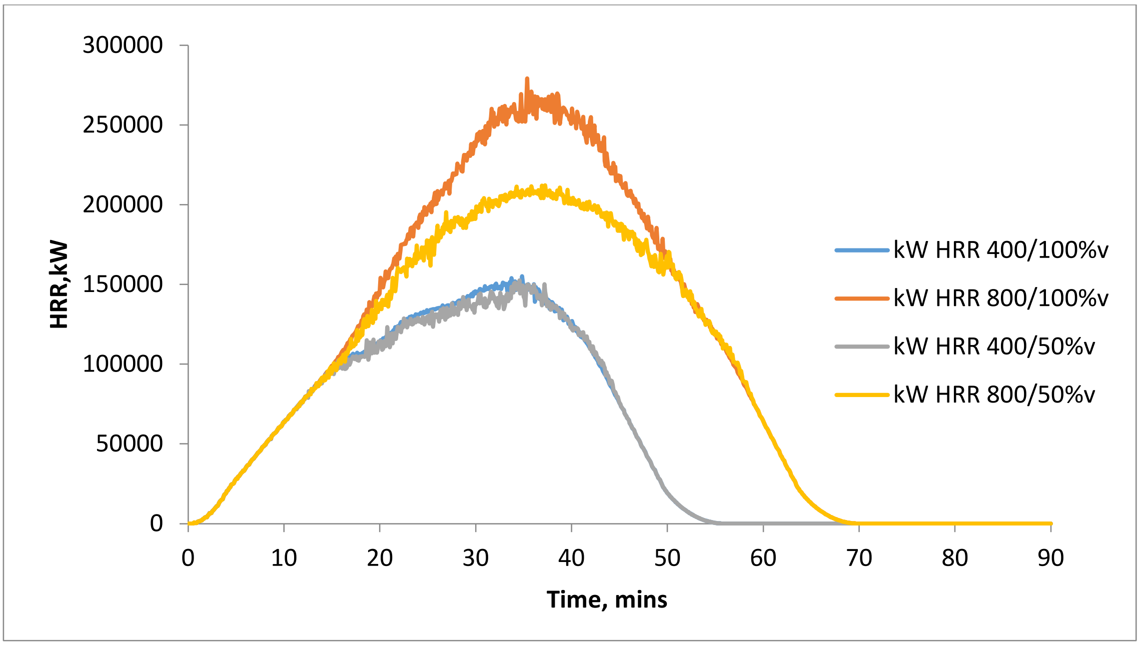


Figure 31. HRR for the whole firecell.

The HRR as determined by FDS for the entire firecell is integrated over the duration of the fire to reconcile the energy as shown in Figure 32. Reasonable agreement is achieved for the 400 and 800 MJ/m² FLEDs with the possible exception of 800 MJ/m² and 50% ventilation where the energy released is lower and can possibly be attributed to the reduced ventilation, meaning that some of the fuel is burning outside of the compartment as a vent fire.

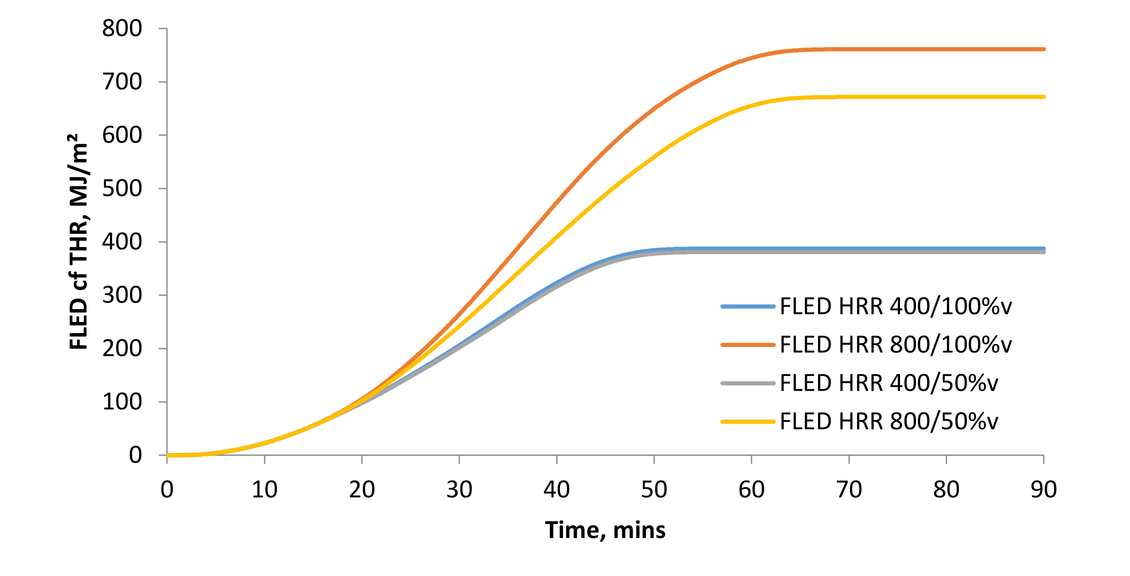


Figure 32. FLED compared with THR.

B.2.1 Temperatures throughout compartment with moving fires

The inclusion of moving fire scenarios to the FDS modelling demonstrates that the temperature excursion of any part of the exposed structure follows a pattern whereby, at some time period within the fire exposure, the structure will be exposed to a temperature spike, but for the remainder of the time, the temperature may be less than 600°C. This concept can be referred to as the near and far field, where the near field is simply the immediate floor area of the fire and the far field is the remainder of the floor.

The following analysis considers air temperatures in the office (open-plan) compartment at intervals as the fire moves onto fresh fuel.

For simplicity, the analysis of the FDS output converts temperatures at multiple locations into a zone model type format where the multiple temperatures within the compartments are integrated into upper and lower temperatures for comparison on a zone model type configuration. The only difference is that this refers to a single point (x, y) location with freedom in the vertical direction instead of considering the compartment as a whole.

Fire exposure with 400 MJ/m² 100% ventilation

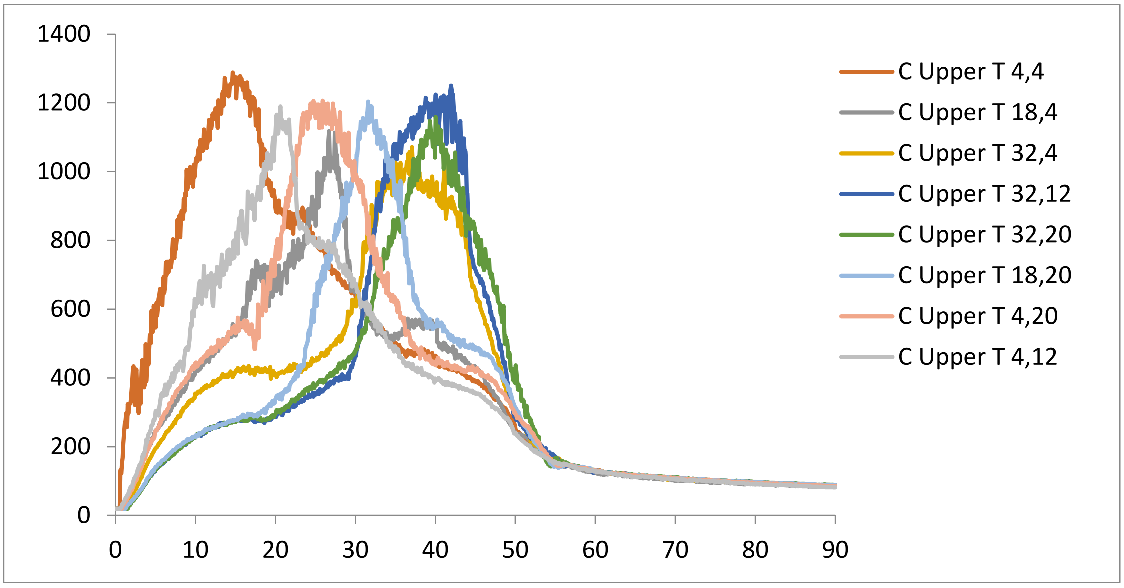


Figure 33. Upper layer gas temperatures 400 MJ/m² 100% ventilation.

The upper layer temperatures at the corners and mid points along the floor are shown in Figure 33, and the corresponding locations in the lower layer are shown in Figure 34. The upper layer/lower layer interface (Figure 36) is at approximately 1000 mm of the 4000 mm height at and around the immediate perimeter of the fire as it travels past. Otherwise, the interface is closer to the ceiling. The lower layer gas temperature does not exceed 200°C but rapidly approaches nearly 1200°C as the fire approaches and falls as it leaves (Figure 34).

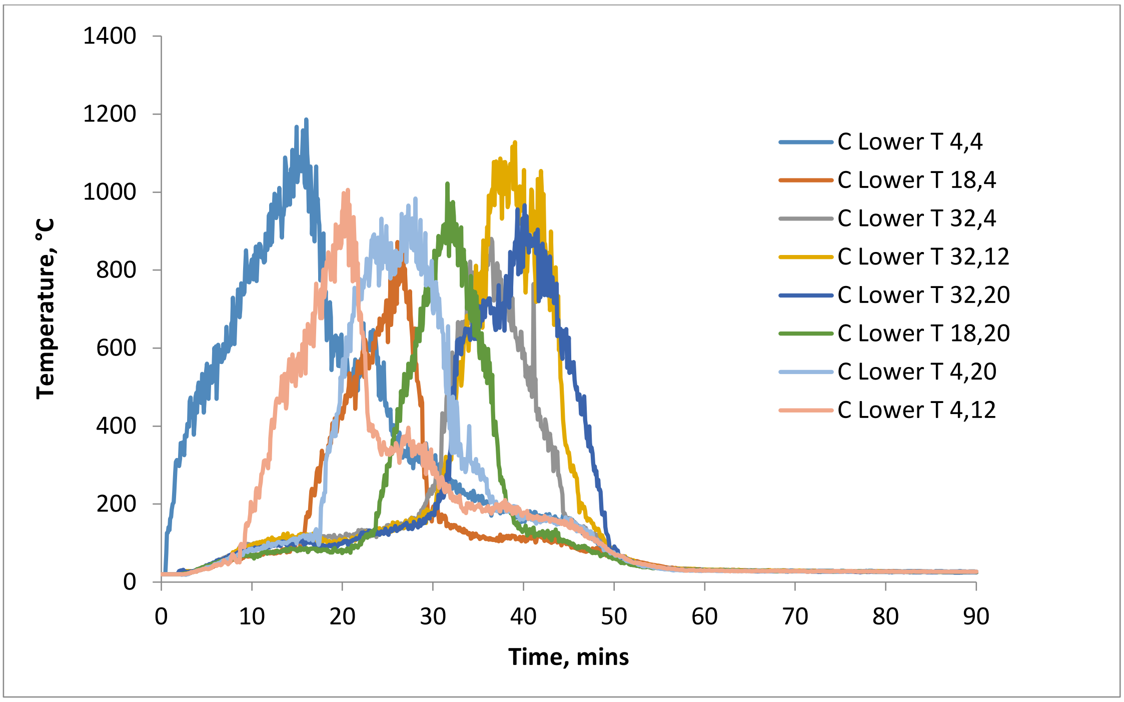


Figure 34. Lower layer gas temperatures 400 MJ/m² 100% ventilation.

A further simplification for the comparison requires the upper and lower temperatures to be averaged around the perimeter of the fire compartment (Figure 35).

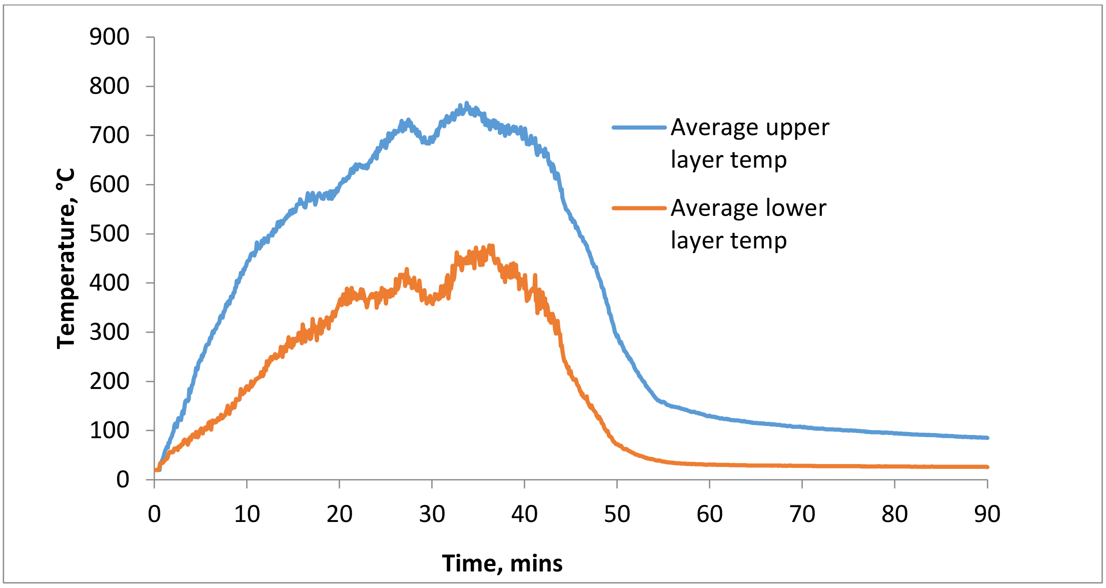


Figure 35. Average upper and lower layer temperatures 400 MJ/m² 100% ventilation.

The upper and lower layer averages are then compared with the relevant parametric fire in Figure 36. For the upper layer temperature, there is reasonable agreement with both maximum temperature and duration such that the parametric exposure could be considered conservative.

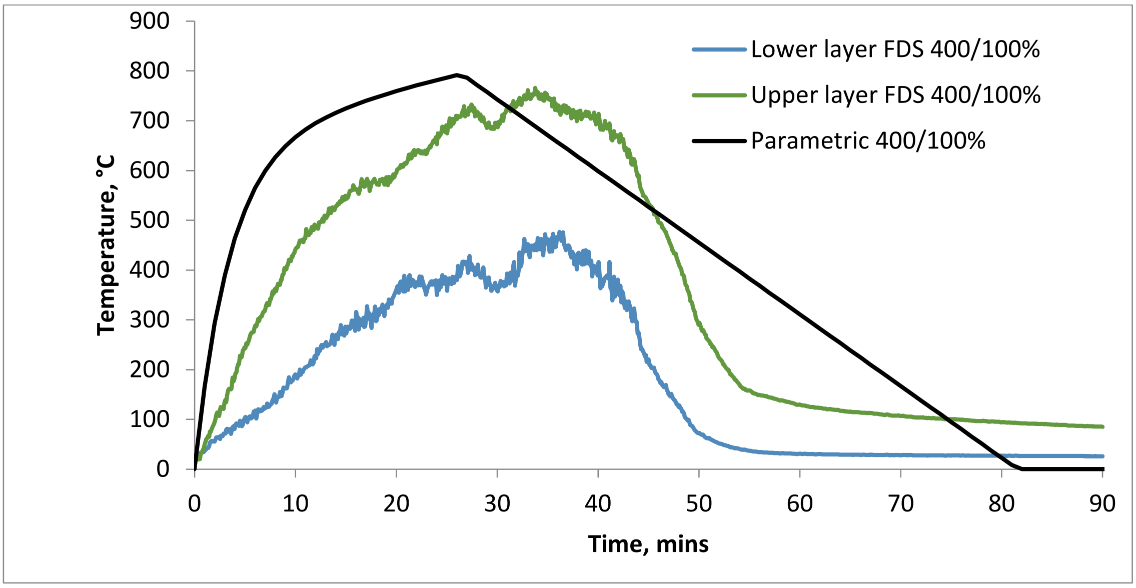


Figure 36. FDS trial compared with parametric fire for FLED of 400 MJ/m² and 100% ventilation.

However, if the upper layer temperatures for a range of locations around the perimeter of the enclosure are compared with the relevant parametric fire in Figure 37, the exposure on the basis of maximum temperature and for the most part duration is well in excess of the parametric fire and it is clearly non-conservative for a travelling fire exposure as the seat of the fire moves around the compartment.

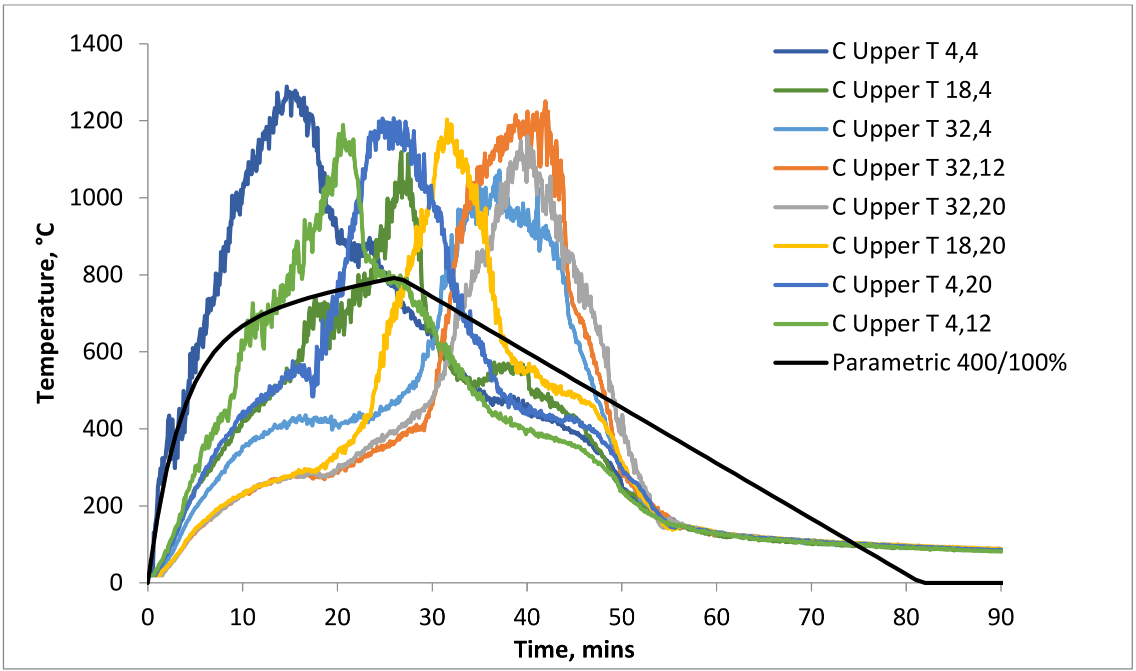


Figure 37. FDS trial of spot temperatures compared with parametric fire for FLED of 400 MJ/m² and 100% ventilation.

For the remaining three parametric fire scenarios, only the comparison of the spot fire temperatures in the upper layer and the parametric fire are presented.

Fire exposure with 400 MJ/m² 50% ventilation

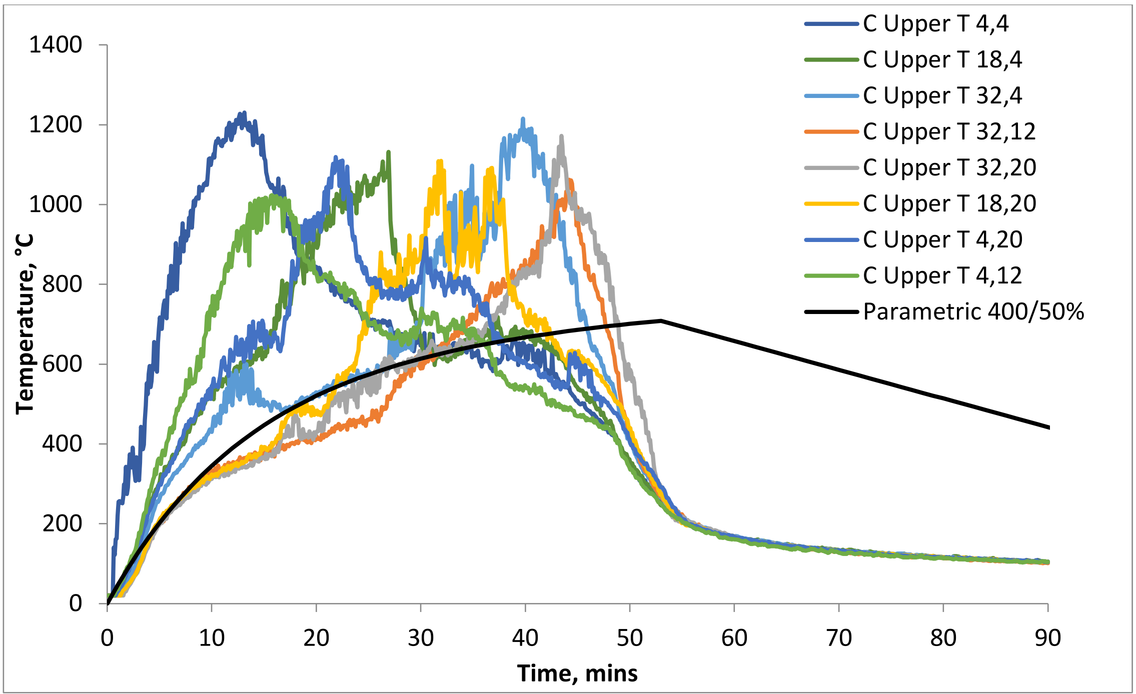


Figure 38. FDS trial of spot temperatures compared with parametric fire for FLED of 400 MJ/m² and 50% ventilation.

Fire exposure with 800 MJ/m² 100% ventilation

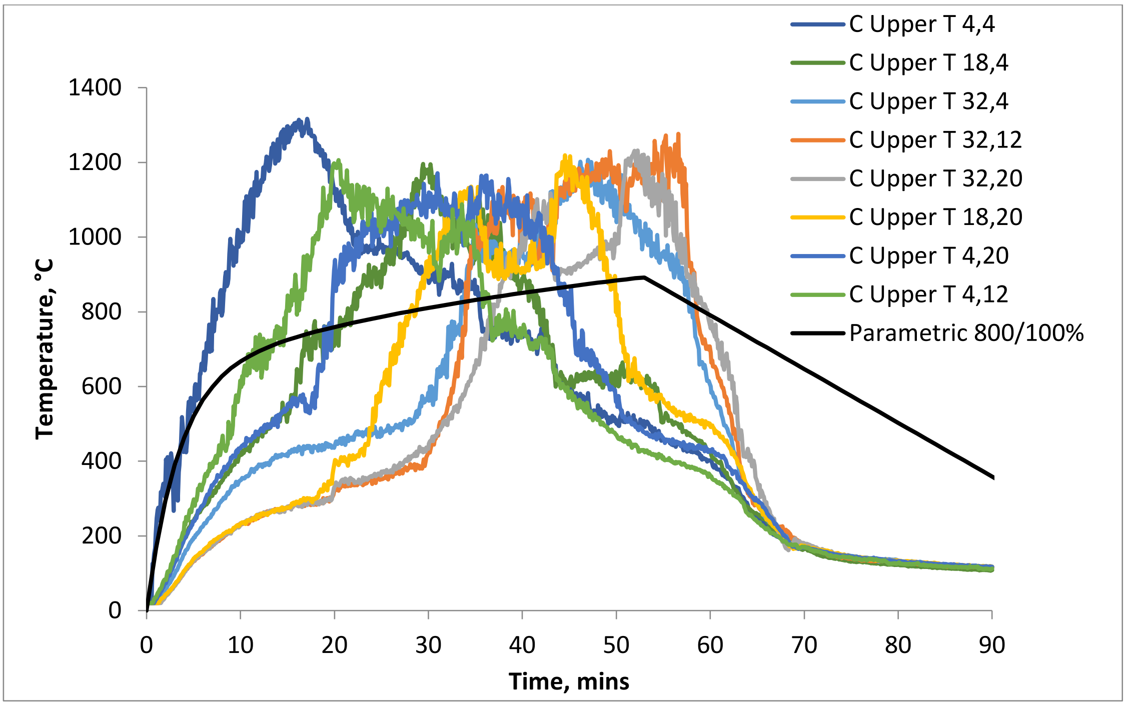


Figure 39. FDS trial of spot temperatures compared with parametric fire for FLED of 800 MJ/m² and 100% ventilation.

Fire exposure with 800 MJ/m² 50% ventilation

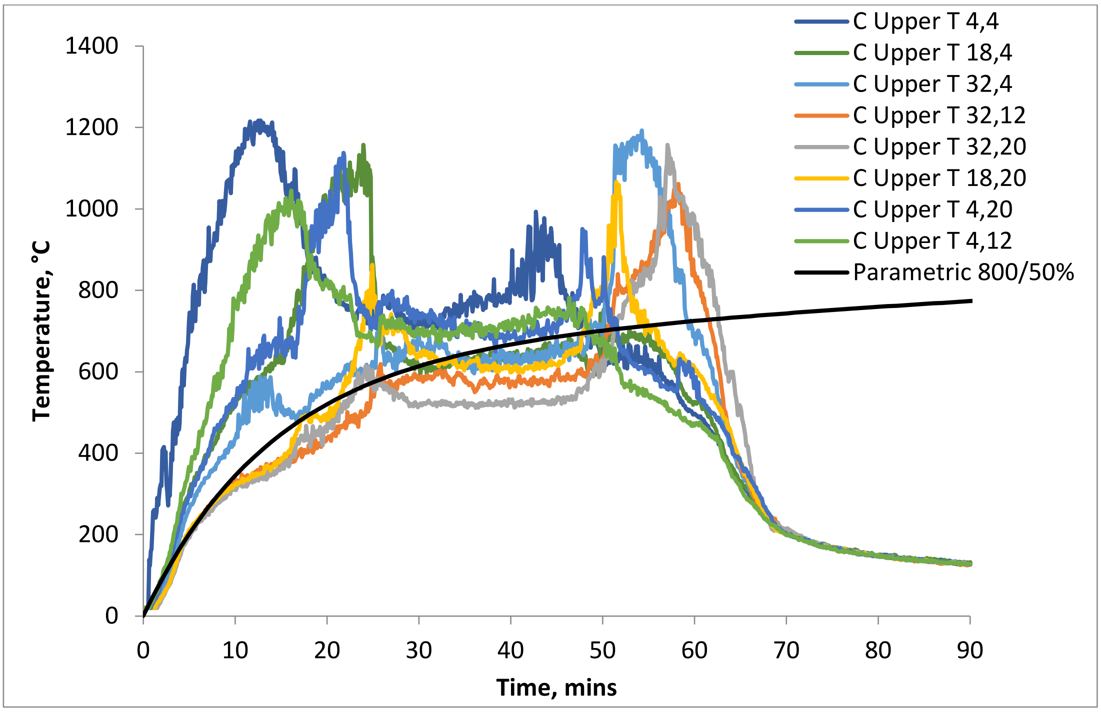


Figure 40. FDS trial of spot temperatures compared with parametric fire for FLED of 800 MJ/m² and 50% ventilation.

For the four parametric fire scenarios, the key comparison is with the spot temperatures that the structural elements may be subjected to as shown in Figure 37 to Figure 40, and in all cases, it is clear that a parametric exposure represents a non-conservative result.

This is primarily in terms of the maximum temperatures in the case that the fire duration of the parametric fire may significantly exceed the FDS modelled exposure, but it is more likely that the structural elements will be challenged more severely by the greater temperature and represent a worst-case scenario for the impact on the structure.

B.2.2 FDS modelling of four travelling fire scenarios in an office building

A travelling fire analysis gives more realistic results in large areas with low ceilings such as an open-plan office space. The rationale for this assumption is that, in the early stages of fire development, sufficient air (oxygen) is available for fire growth, even with no external ventilation to the enclosure. As the fire progresses, heat builds up and temperatures increase, which may initiate some breakage of windows to sustain fire development as it moves. Without sufficient ventilation, the fire progression may be affected where the pyrolysed fuel does not burn within the enclosure and instead may burn as a vent fire or simply escape as unburnt gases. Once some degree of equilibrium is established between the consumption of oxygen and replenishment through the vents, the progression of the fire will consume fuel and move accordingly.

In the extreme event with no ventilation from windows breaking, a fire may continue to burn to a limited degree with air circulating from locations remote from the fire. Eventually, there is insufficient air (oxygen) for further combustion.

To summarise:

- The fire does not occupy the entire enclosure at any one time.
- The seat of fire moves as fuel is consumed and fresh fuel is available.
- The concept of near field and far field is established.
- The near field is the seat of the fire and plume above and immediate surroundings that are subjected to high temperatures.
- The far field is the remainder that has either been exposed to the seat of the fire or may yet be exposed.
- A spike in temperature (gas and structure) is experienced as the fire moves by.
- The total heat release in the fire event can be reconciled with the FLED to perform an energy balance.
- The rate of fire movement will be a function of initially available air in the enclosure and then the ventilation once that is partly consumed.
- Some experimentation with the fire travel speed is recommended to determine a worst-case scenario in terms of structural fire exposure.

Worked example of travelling fire

To demonstrate the concept of a travelling fire, a single floor in a multi-storey building set up as open-plan office space is the baseline example with parameters in Table 12. The type of space where a travelling fire scenario is applicable is one with a high aspect ratio where the length and/or breadth (and thus floor area) is in the region of orders of magnitude greater than the height such that full cell (fire) involvement is inhibited in favour of progressive localised burning.

Table 12. Building – internal dimensions.

	Length	Breadth	Height
Building – internal dimensions, m	36	24	4
Vent dimensions, m	-	1	2
Number of vents	1	28	56
Ventilation factor, $m^{3/2}$	2.828	79.20	158.4
Opening factor Fv, $m^{-1/2}$		0.036	0.072
Relative ventilation in examples		50%	100%

The layout of the 36 x 24 m floor is shown in Figure 41 as set up in FDS.

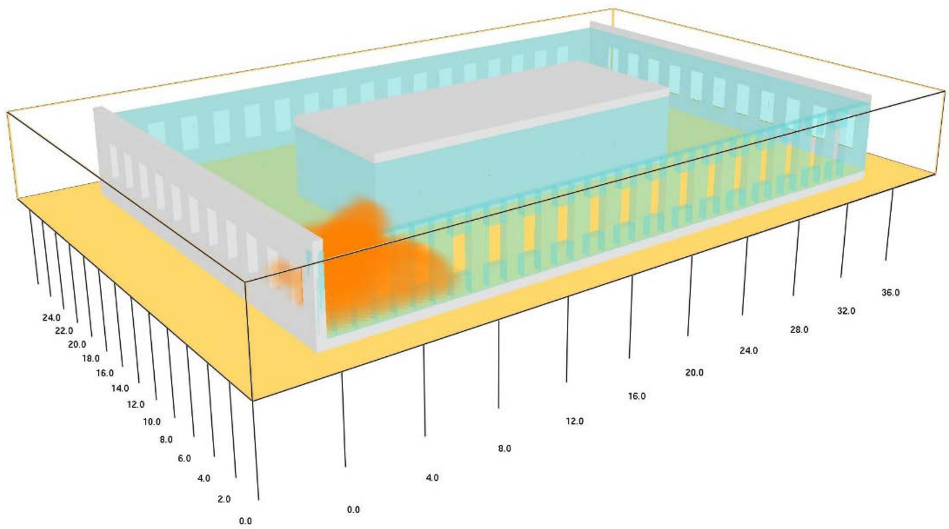


Figure 41. FDS simulation of travelling fire for 400 MJ/m^2 and 100% ventilation at 5 minutes.

The progression of the travelling fire throughout the compartment on two fronts is illustrated in Figure 42.

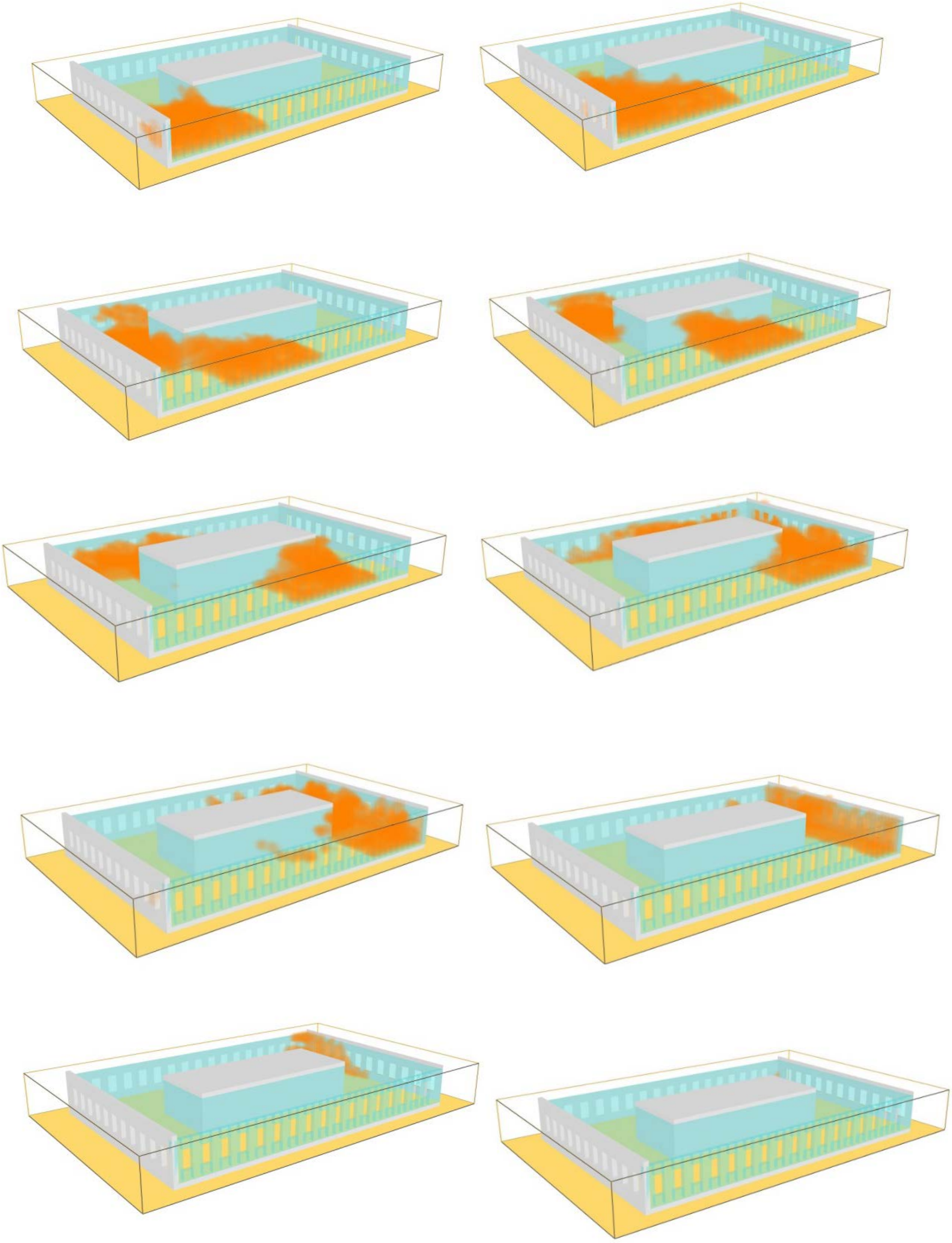


Figure 42. Progression of FDS modelling of travelling fire from 10 minutes to 55 minutes @ 5 minute intervals.

The travelling fire principles are employed for the four fire scenarios with FLEDs of 400 and 800 MJ/m^2 and 100 and 50% ventilation. The maximum temperatures of the concrete and steel structural elements (unprotected and protected with 13 mm of sprayed mineral fibre) are summarised in Table 13. More extensive data is shown in Figure 43 to Figure 60.

Table 13. Summary maximum temperatures.

Scenario	Fire, °C (max)	Concrete @ 25 mm FDS	Concrete @ 25 mm 1D FD	Steel H_p/A=30 upper layer	Steel H_p/A=30 lower layer
Parametric 400/100%	855		280	395/83	
Parametric 400/50%	763		310	460/117	
Parametric 800/100%	961		419	**700**/73	
Parametric 800/50%	855		458	**741**/89	
FDS 400/100%	1273	397 FDS	422	**655**/99	352/68
FDS 400/50%	1205	395 FDS	390	**607**/105	422/86
FDS 800/100%	1297	504 FDS	495	**831**/129	501/83
FDS 800/50%	1204	444 FDS	457	**709**/130	**571**/93

In general, the temperatures are higher for the travelling fire scenarios, where the same energy is released, than the earlier parametric scenarios, indicating that a parametric approach may be delivering non-conservative results.

FLED 400 MJ/m^2 and 100% ventilation

The two highest temperature peaks for the upper and lower layers occur at the original fire location and at the point where the two fire fronts meet again at the nearly opposite end point of the open-plan floor. These two peaks for the upper and lower temperature are combined in Figure 43.

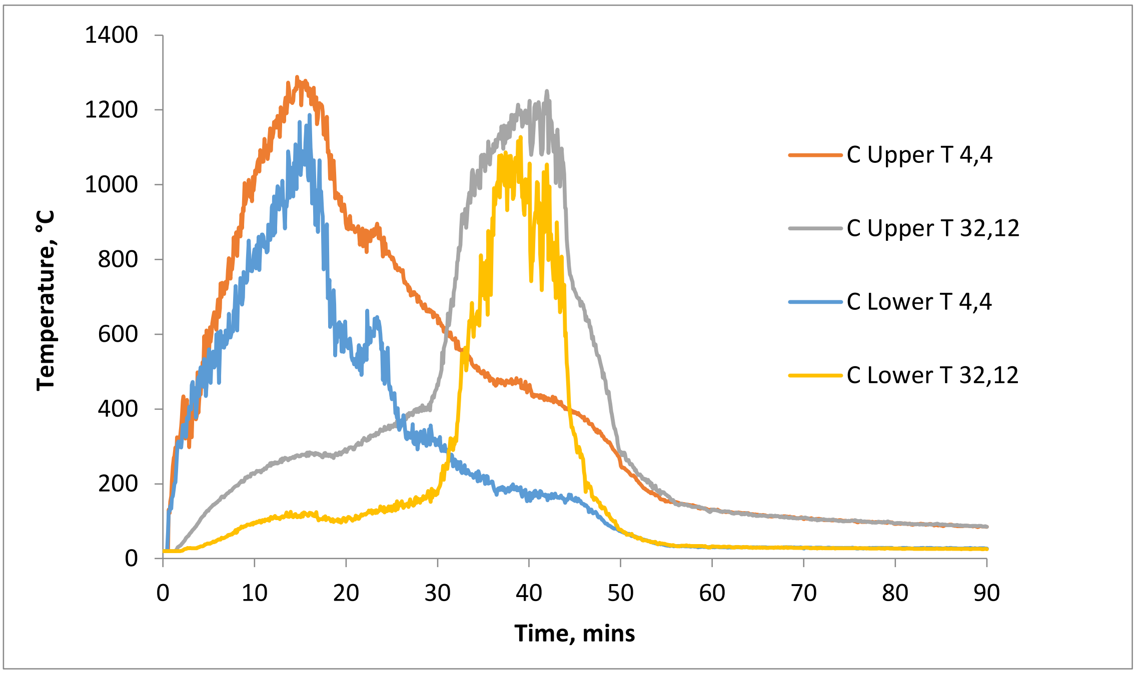


Figure 43. Fire temperature in upper and lower levels in worst regions 400 MJ/m^2 and 100%.

In Figure 43, the second peak(s) are when the two fire fronts converge on one location at the opposite end of the space.

For consideration of a worst-case heating scenario, the highest peak with the greatest area under the curve is a likely candidate for closer scrutiny, and in this case, it coincides with the origin of the fire (first peak). For a concrete ceiling, Figure 44 shows the concrete ceiling temperatures at 25 mm depth at 4 m intervals around the centreline of the perimeter as calculated by FDS for the moving fire. This indicates that the highest temperature (397°C) and greatest exposure is in the vicinity of the area where the fire originates, followed by a series of secondary peaks as the fire travels around the perimeter culminating in secondary higher peaks where the two fire fronts converge at the near opposite end of the space.

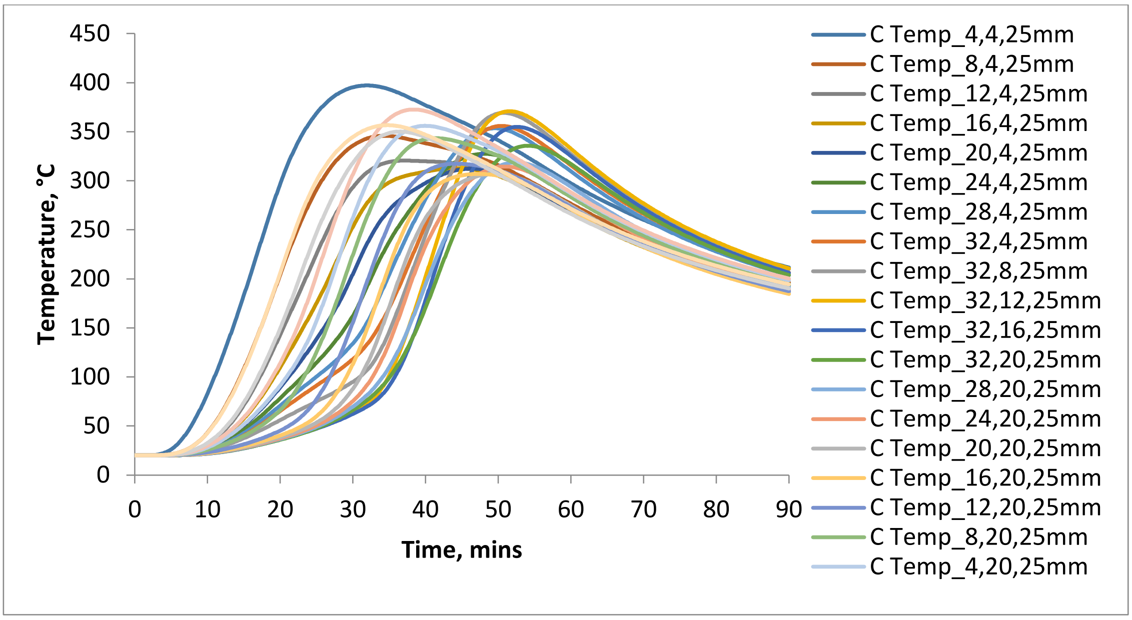


Figure 44. Concrete temperatures with FLED 400 MJ/m² and 100% ventilation at 25 mm depth in moving fire according to FDS.

Closer analysis of the concrete temperatures in Figure 45 compares the two fire peaks from Figure 43 and calculates concrete temperatures using a 1D finite difference method (Incropera, DeWitt, Bergman and Levine, 2007). Peak temperatures at the 25 mm depth for the fire origin and remote location are 422°C and 338°C for the fire origin and remote location respectively. This agrees favourably with the FDS calculation in Figure 44, where maximum temperatures of 397°C and 369°C were recorded.

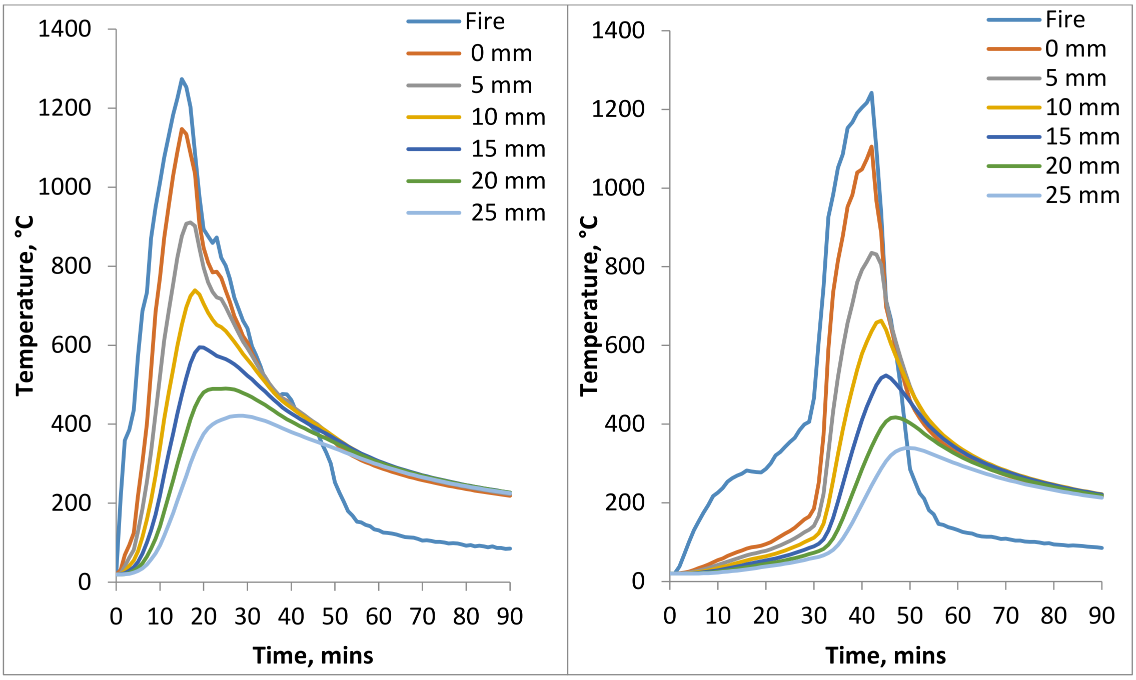


Figure 45. Concrete ceiling temperatures 400 MJ/m² at 100% ventilation.

Considering the temperatures of the steel member (column 356 x 406 X 551 kg/m and 4-sided exposure with H_p/A=30 without protection and with 13 mm sprayed mineral fibre protection) in the region of fire origin and remote, Figure 46 shows the same trend as the concrete where the unprotected steel in the location of the fire origin reaches a higher temperature of 655°C compared with 598°C.

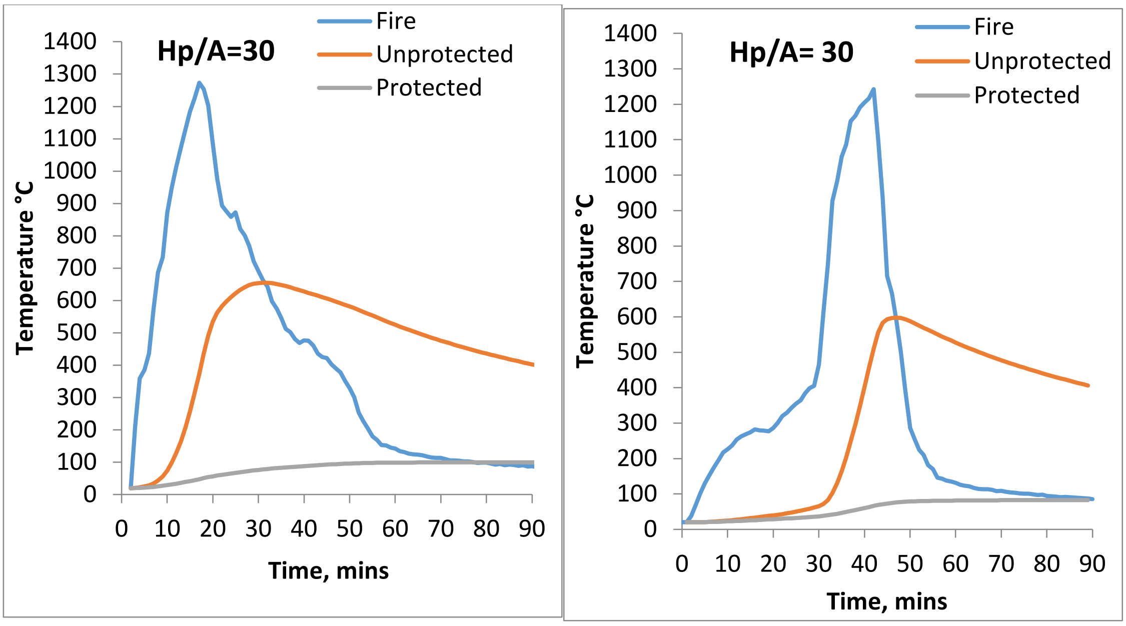


Figure 46. Steel temperatures in upper layer (H_p/A=30) 400 MJ/m² at 100% ventilation.

For the lower layer, where the fire temperature exposure appears to be only marginally less, the temperature response of the steel members is markedly lower as shown in Figure 47. This would be applicable to the lower portion of columns not subject to high temperatures in compartments' upper levels. If the column is continuous for the entire vertical space, protection for the upper level will be applicable, so the upper level temperature should be applied for the design.

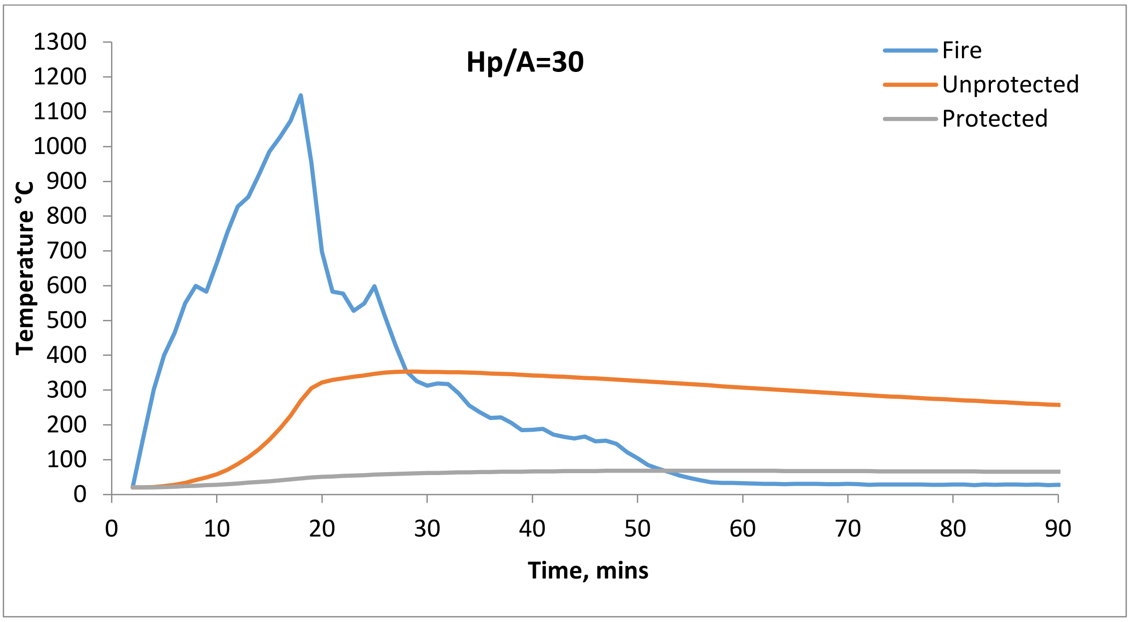


Figure 47. Steel temperatures in lower layer (H_p/A=30) 400 MJ/m² at 100% ventilation.

FLED 400 MJ/m² 50% ventilation

The above analysis is repeated with reduced ventilation and a slightly different time-temperature exposure in Figure 48.

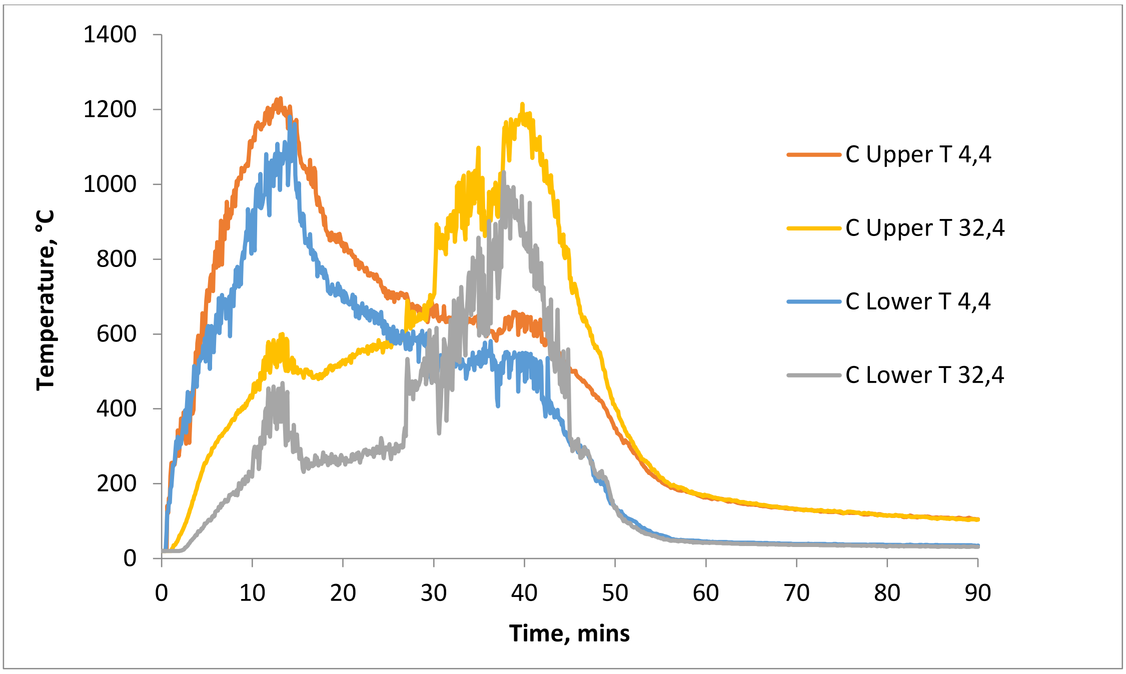


Figure 48. Upper and lower levels in worst regions 400 MJ/m² and 50% (2 maximums).

The response of the concrete in Figure 49 is slightly different with a seemingly stronger peak temperature at the remote (from fire origin) end of the enclosure. This can be attributed to the reduced ventilation causing a greater build-up of heat remote from the fire origin such that, when the fire does arrive, the concrete has undergone some considerable preheating. Comparing Figure 43 and Figure 48, the latter shows a greater temperature in advance of the fire arrival, but the maximum temperature (in the concrete) of 395°C is marginally less than the 397°C for the 400 MJ/m² 100% ventilation case.

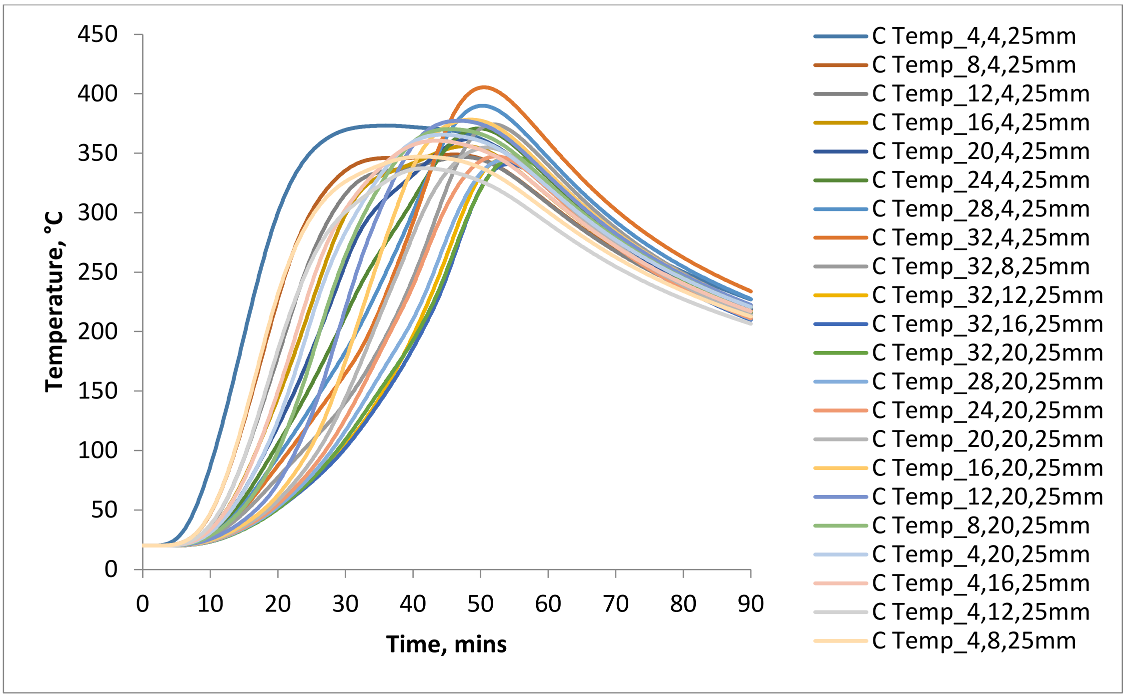


Figure 49. Concrete temperatures with FLED 400 MJ/m^2 and 50% ventilation at 25 mm depth in moving fire according to FDS.

The two different fire exposure regimes (fire origin and remote location) are compared in Figure 50 (1D FD), and the maximum temperatures reached at the 25 mm depth are 344°C and 370°C respectively, which concurs with the relative differences in Figure 49.

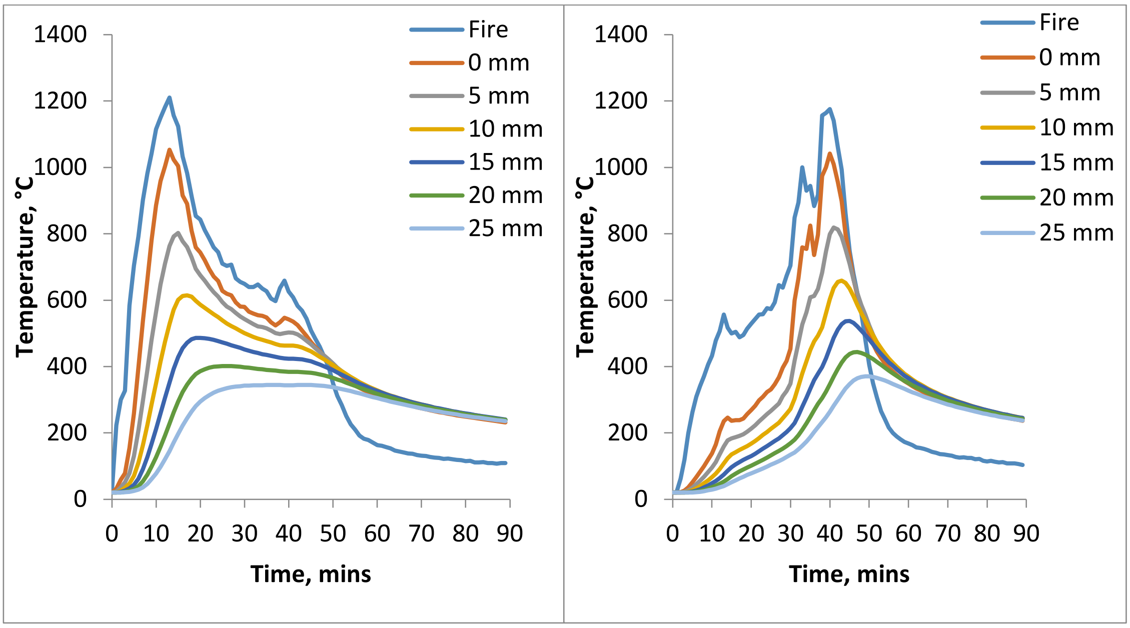
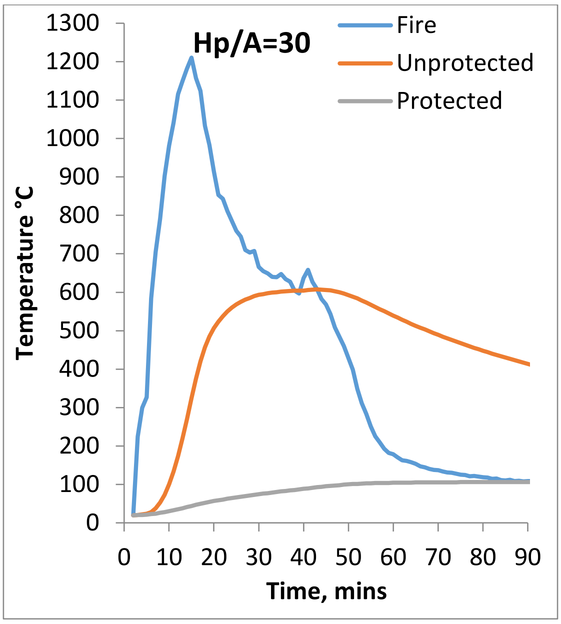


Figure 50. Concrete ceiling temperatures above fire origin and at a remote end of compartment 400 MJ/m^2 at 50% ventilation.

A similar comparison of steel temperatures for the two fire peaks in Figure 51 shows the same trend to the concrete where the unprotected steel in the location of the fire origin reaches a lower temperature of 607°C compared with a higher temperature of 621°C in the region where the fire fronts converge.



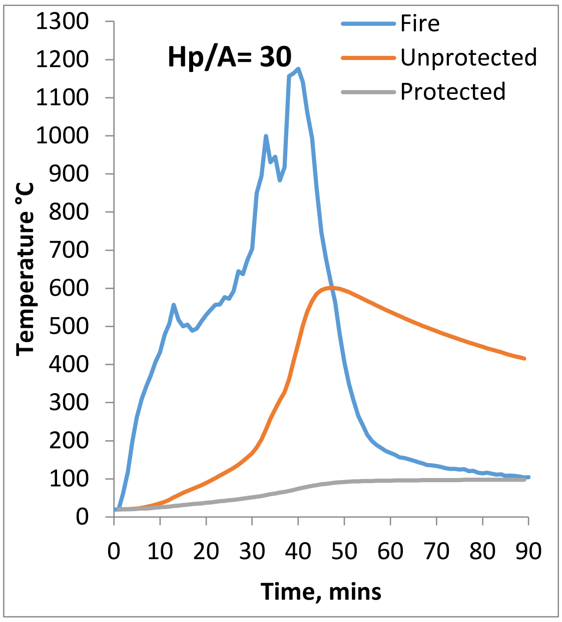


Figure 51. Steel temperatures in upper layer (H_p/A=30) 400 MJ/m² at 50% ventilation.

Figure 52 shows the temperature in the lower layer at the fire origin. Maximum steel temperatures for an H_p/A=30 column in the upper and lower zones at the fire origin were 607°C and 422°C respectively.

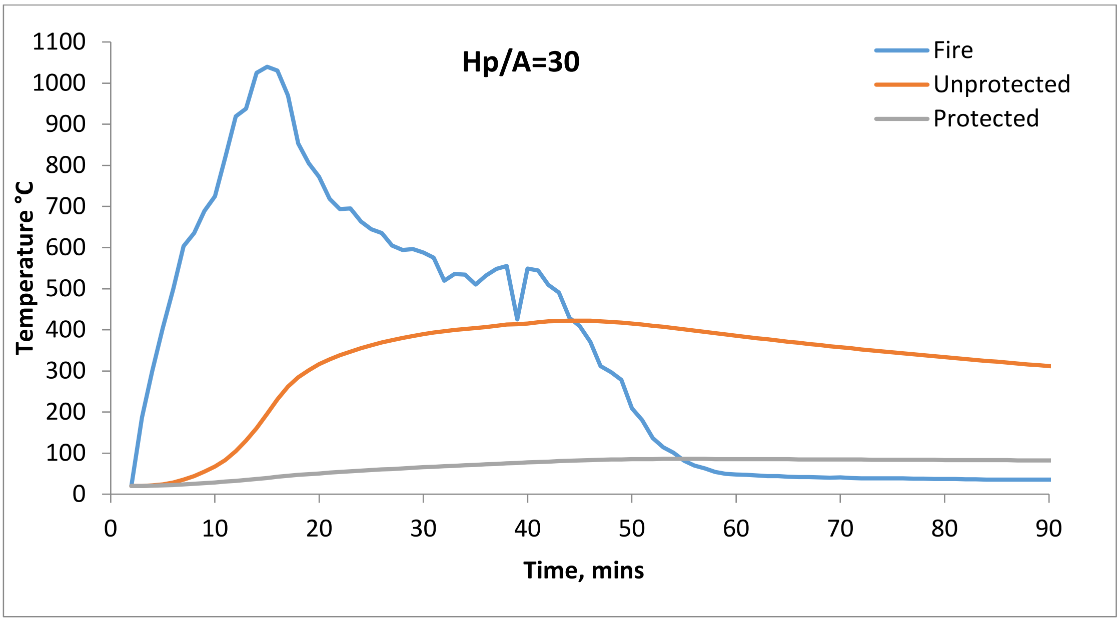


Figure 52. Steel temperatures in lower layer (H_p/A=30) 400 MJ/m² at 50% ventilation.

FLED 800 MJ/m² 100% ventilation

The two highest temperature peaks for the upper and lower layers occur at the original fire location and at the point where the two fire fronts meet again at the nearly opposite end point of the open-plan floor. These two peaks for the upper and lower temperature are combined in Figure 53.

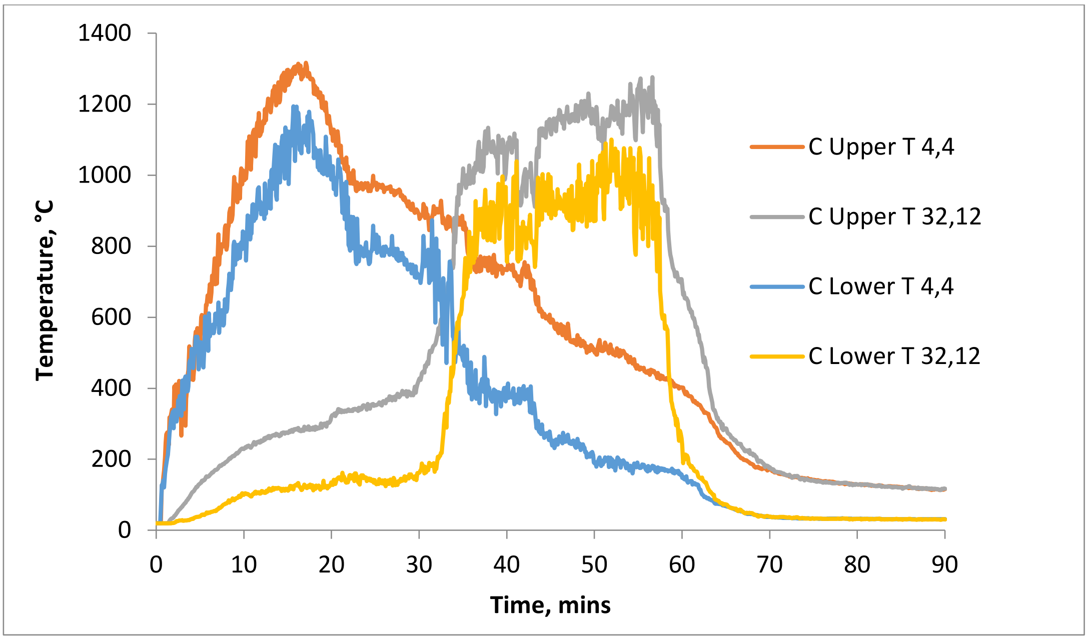


Figure 53. Upper and lower levels in worst regions 800 MJ/m² and 100%.

For consideration of a worst-case heating scenario, the highest peak with the greatest area under the curve is a likely candidate for closer scrutiny, and in this case, it coincides with the origin of the fire (first peak). For a concrete ceiling, Figure 54 shows the concrete ceiling temperatures at 25 mm depth at 4 m intervals around the centreline of the perimeter as calculated by FDS for the moving fire. This indicates that the highest temperature of 504°C and greatest exposure is where the two fire fronts converge at the opposite end of the space remote from where the fire originated, as opposed to 462°C above the fire origin.

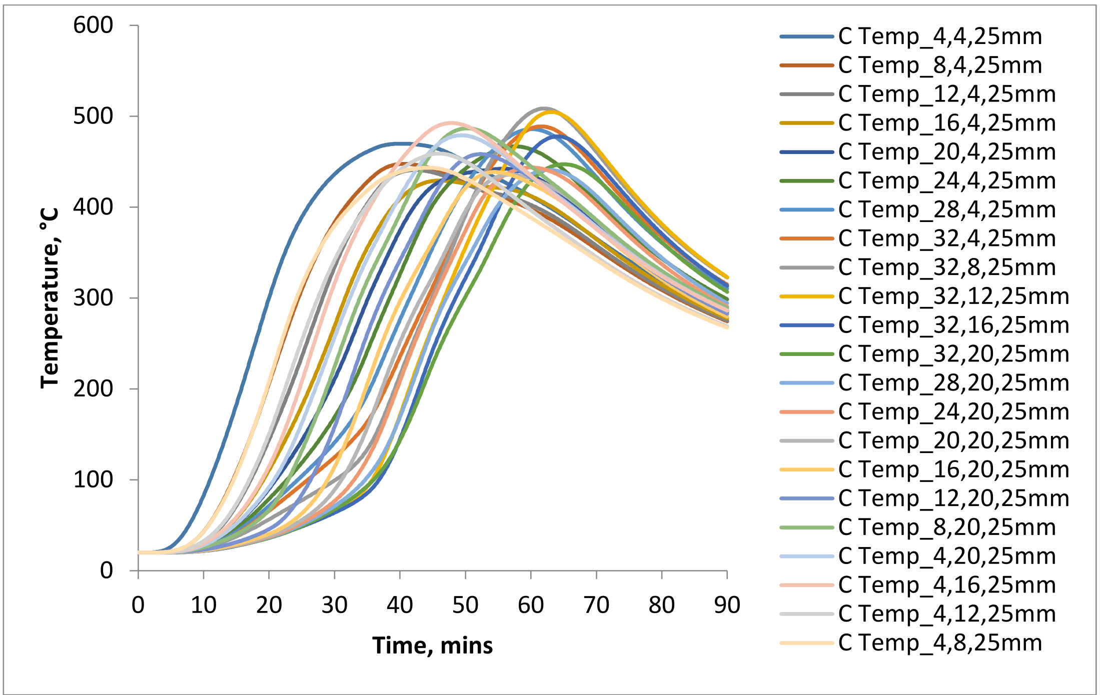


Figure 54. Concrete temperatures with FLED 800 MJ/m² and 100% ventilation at 25 mm depth in moving fire according to FDS.

Figure 55 compares the two fire peaks from Figure 53 and calculates concrete temperatures using a 1D FD (finite difference) method. Peak temperatures at 25 mm depth for the fire origin and remote location are 495°C and 483°C respectively.

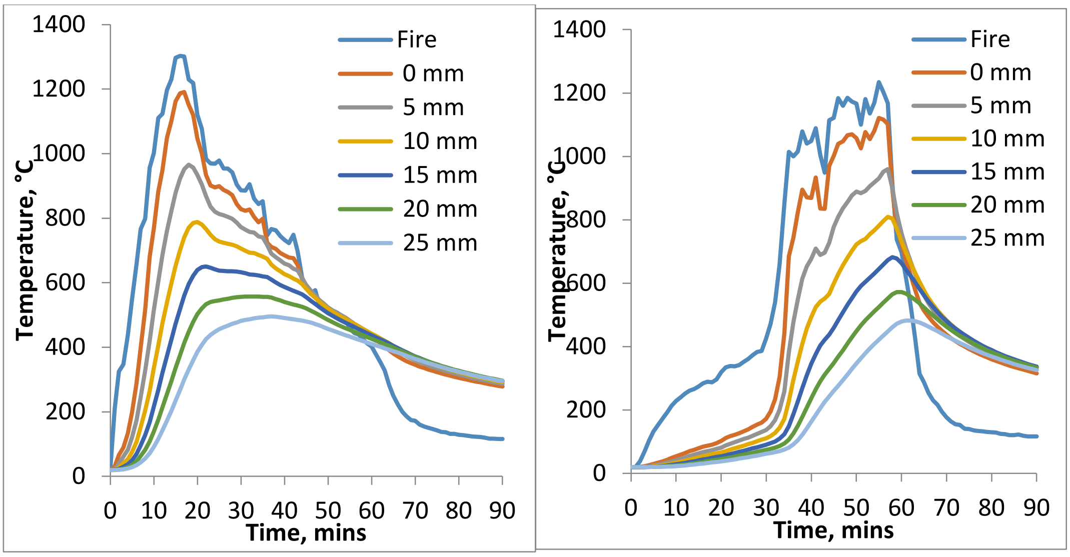


Figure 55. Concrete ceiling temperatures 800 MJ/m² at 100% ventilation.

A similar comparison of steel temperatures for the two fire peaks in Figure 56 shows the opposite trend to the concrete where the unprotected steel in the remote location reaches a higher temperature of 928°C compared with 829°C at the fire origin.

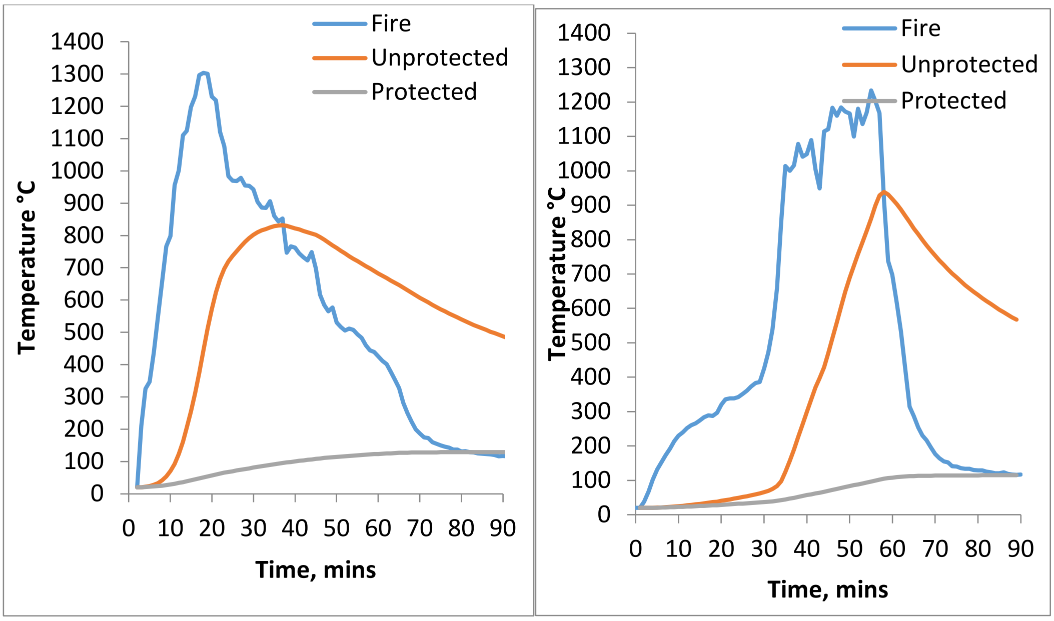


Figure 56. Steel temperatures in upper layer (H_p/A=30) 800 MJ/m² at 100% ventilation.

FLED 800 MJ/m² 50% ventilation

The two highest temperature peaks for the upper and lower layers occur at the original fire location and at the point where the two fire fronts meet again at the right-hand end of the open-plan floor. These two peaks for the upper and lower temperature are combined in Figure 57.

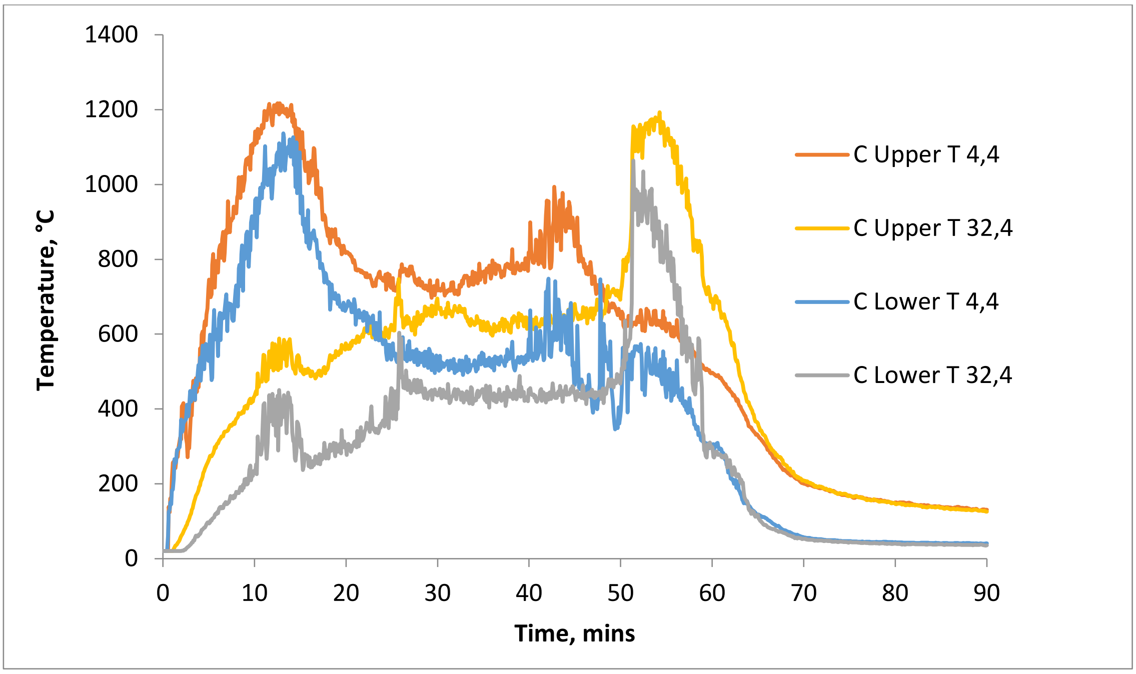


Figure 57. Upper and lower levels in worst regions 800 MJ/m² and 50% (2 maximums).

For consideration of a worst-case heating scenario, the highest peak with the greatest area under the curve is a likely candidate for closer scrutiny, and in this case, it coincides with the origin of the fire (first peak). For a concrete ceiling, Figure 58 shows the concrete ceiling temperatures at 25 mm depth at 4 m intervals around the centreline of the perimeter as calculated by FDS for the moving fire. The maximum temperature of 444°C occurs above the fire origin, and the next peak of 434°C is where the two fire fronts converge at the opposite end of the front side remote from where the fire originated.

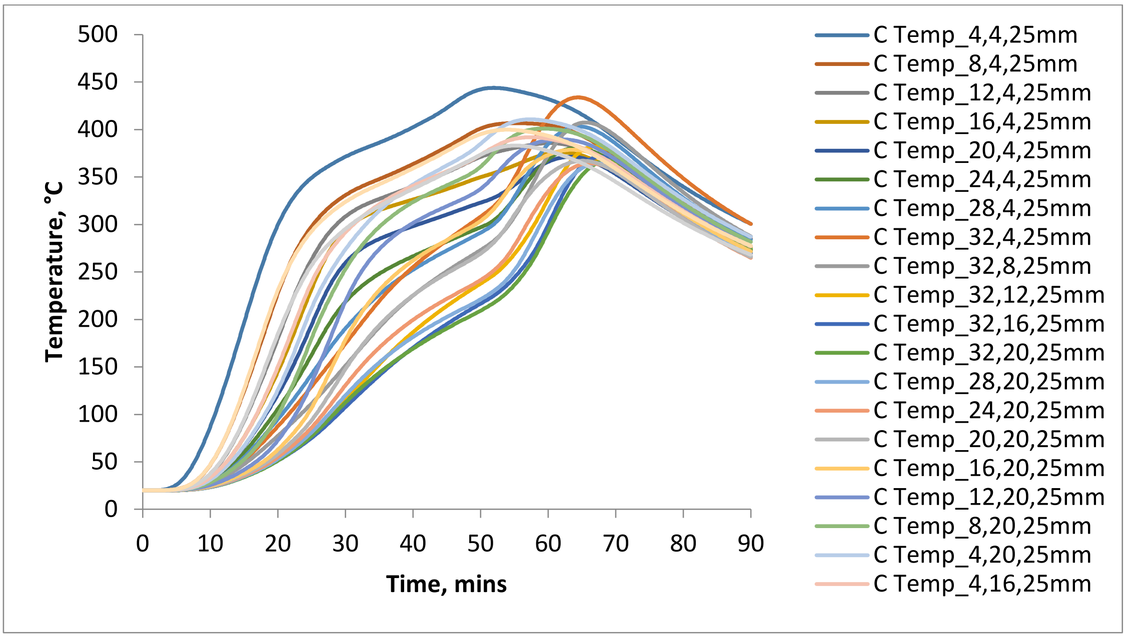


Figure 58. Concrete temperatures with FLED 800 MJ/m² and 50% ventilation at 25 mm depth in moving fire according to FDS.

Figure 59 compares the two fire peaks from Figure 57 and calculates concrete temperatures using a 1D finite difference method. Peak temperatures at 25 mm depth are 457°C and 381°C for the fire origin and remote location respectively.

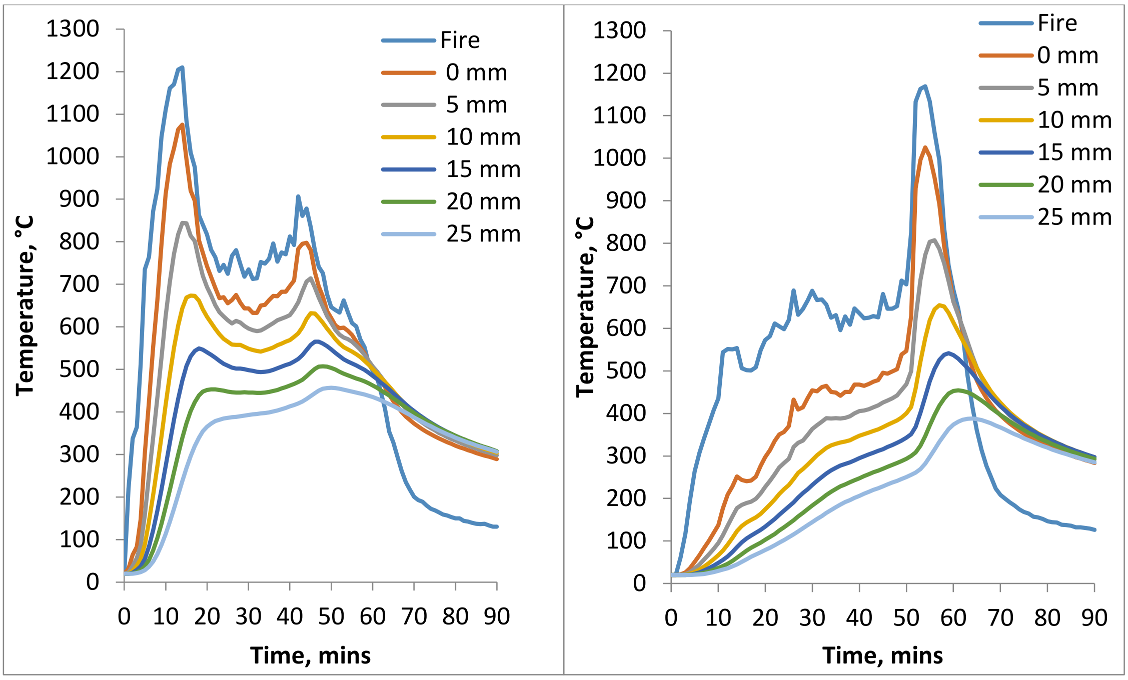


Figure 59. Concrete ceiling temperatures 800 MJ/m² at 50% ventilation.

A similar comparison of steel temperatures for the two fire peaks in Figure 60 shows the same trend to the concrete where the unprotected steel in the location of the fire origin reaches a higher temperature of 709°C compared with 614°C.

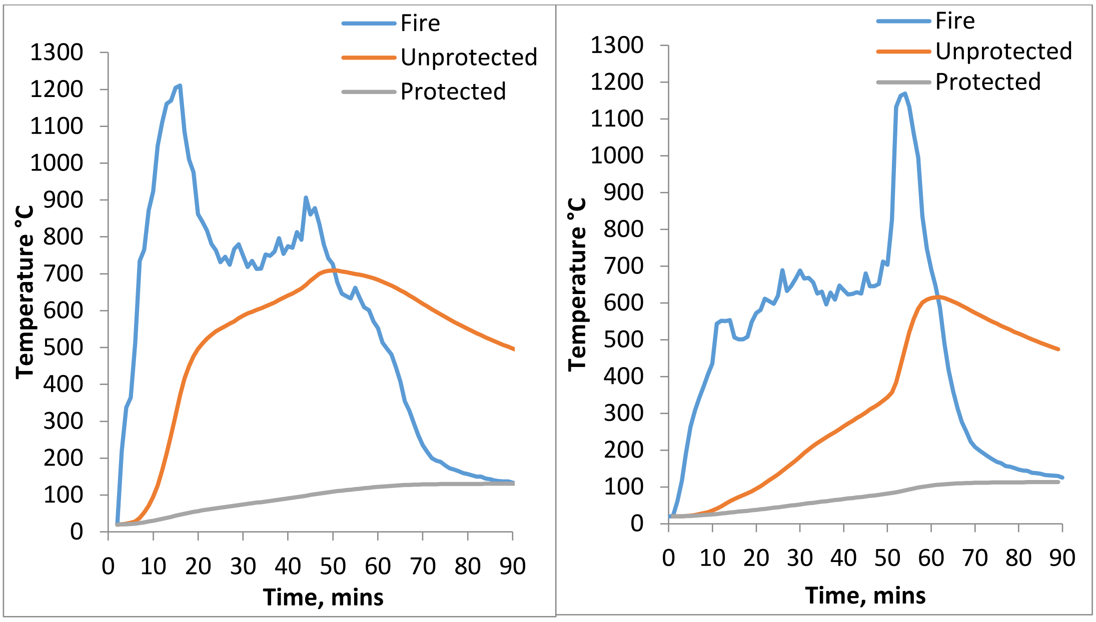


Figure 60. Steel temperatures in upper layer ($H_p/A=30$) 800 MJ/m² at 50% ventilation.

Maximum exposure may not necessarily occur at the fire origin, and some judgement is required to ensure that the most challenging exposure is captured to access the material response.

In the above examples where the ventilation is more limited (50%), the greater exposure tends to be remote from the fire origin.

B.2.3 Travelling fires limited by progressive window breakage

In the previous section on travelling fires, it is assumed that the ventilation condition remains constant throughout the passage of the fire scenario. In other words, all windows are 100% open or broken at the beginning. The more likely scenario is that the windows are initially intact and then progressively break as the fire develops.

To model progressive breaking of windows in FDS, a trigger condition is required such as the temperature or heat flux exceeding a predetermined level in the vicinity of the window. Trial runs (FDS) have shown that there is a fairly well defined break point where the windows break and the fire progresses (under ventilation-controlled conditions), or the windows do not break and fire development is inhibited. The break point in the trials conducted indicated the range to be 475–500°C or 9–10 kW/m^2 – in other words, a complete dichotomy where the fire either develops or not. This seemingly well defined line defies what might actually happen in practice. In FDS, the treatment of combustion chemistry is rudimentary and behaves in a burn/no burn fashion depending on the oxygen availability. In reality, combustion of any particular fuel comprises of not just dozens but hundreds of chemical reactions requiring different limiting conditions of temperature and oxygen to proceed and that may apply to just a relatively simple single pure substance as opposed to a mixture of a myriad of more complex chemicals (fuels) in a real building. On this basis, it is reasonable to assume that a supposedly well defined break point is actually exceedingly blurred, making any predictions of window breakage unreliable.

To arrive at a credible worst-case scenario, it should be assumed that window breakage occurs at a sufficient rate to sustain fire progress in a relatively uninhibited fashion, and this can be achieved with FDS by selecting the window breakage parameter at a level that the fire continues to develop.

Example of windows breaking

Trial runs with FDS as presented below based on the heat flux incident on each window indicate a very well defined demarcation line or dichotomy at the point at which windows break. This results either in continuing development of the fire throughout a compartment and high temperature exposure to the structure, or with limited or no window breakage, the fire becomes ventilation controlled. Vent fires may occur due to limited ventilation, meaning that the heat is released externally to the compartment, not impacting the internal structure. If no windows break, the fire development is subject to the available air within the compartment and will eventually extinguish once the available oxygen is consumed. Therefore, in this scenario, the temperatures within a compartment and the temperature of the exposed structure are considerably reduced as is the risk of structural damage.

Using the previous example with an FLED of 800 MJ/m^2 and potentially 100% ventilation, in the event that all windows break, trials were conducted to determine the conditions between limited window breakage and total window breakage.

HRR results are presented in Figure 61 where 9–10 kW/m^2 is the limiting heat flux on the window breakage and similarly the temperature range for the same result was 475–500°C. For the 9 kW/m^2 case, the windows all break, whereas for the 10 kW/m^2 case, a limited number break near the fire origin as the fire grows initially on the oxygen available within the compartment. Once that is partially consumed, the HRR levels off and is insufficient for further window breakage as the (limited) fire travels through the compartment.

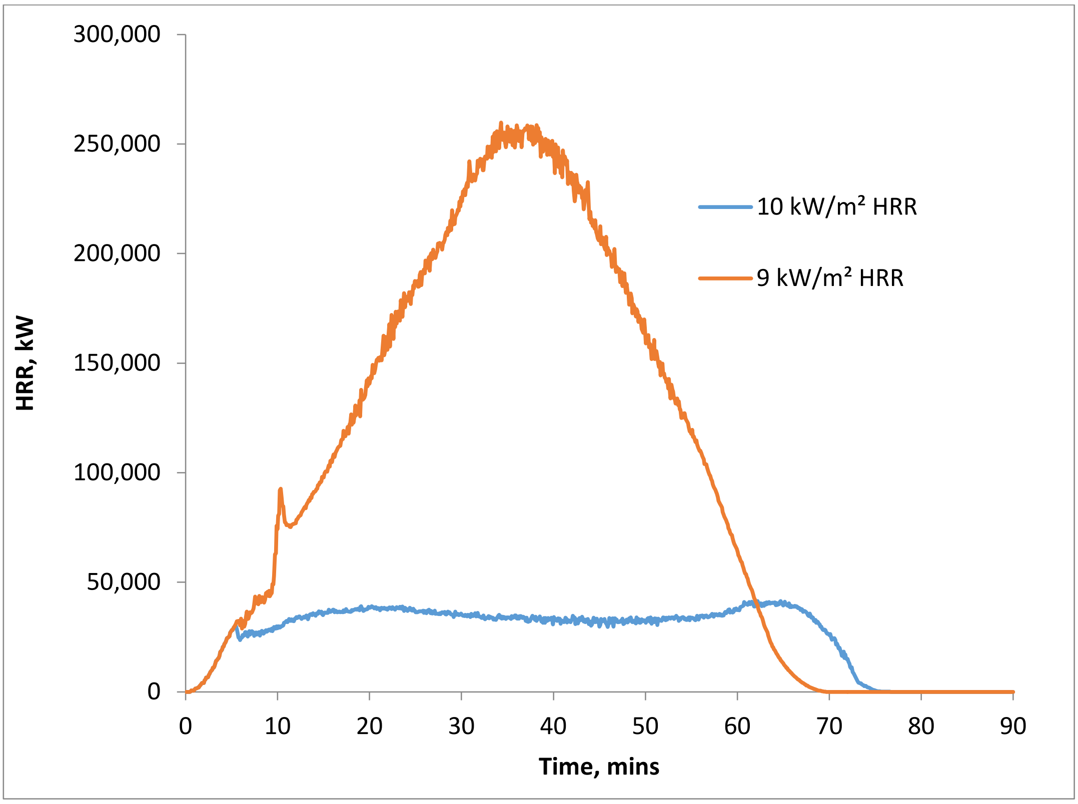


Figure 61. The influence of window breakage on HRR.

Similarly, an energy balance is presented in Figure 62 that, for the fully developing case, compares very well with the case for 100% open windows initially that was presented earlier in Figure 32.

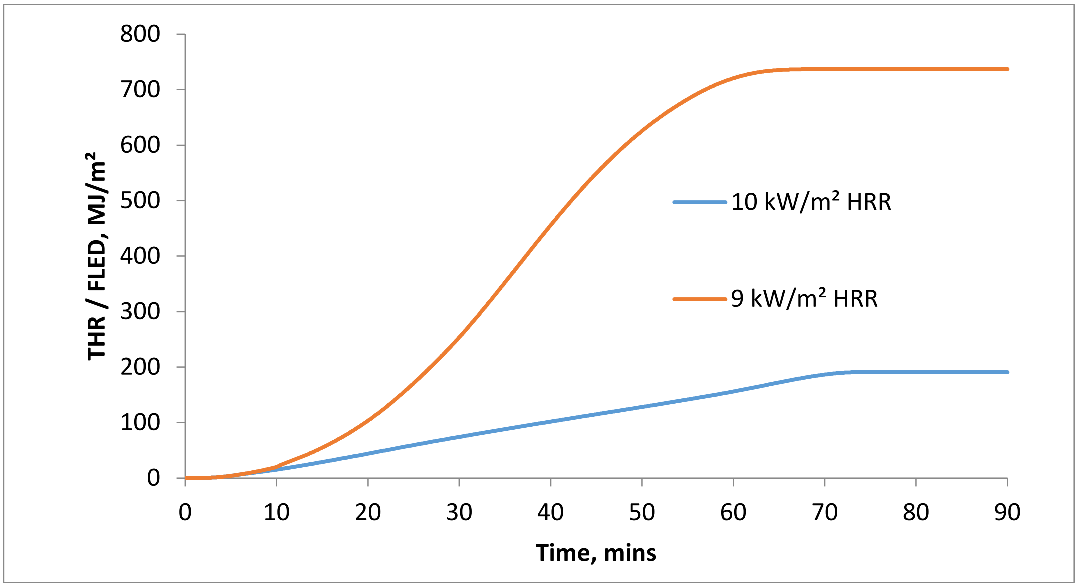


Figure 62. THR versus window breakage on basis of heat flux exposure.

Using as a base case the scenario of an FLED of 800 MJ/m^2 and 100% ventilation, FDS modelling trials were conducted to compare the two scenarios of limited window breakage with total window breakage. These two scenarios are then compared with the base case of all windows being open initially in Table 14.

It was apparent that limiting the window breakage has the effect of significantly reducing the temperatures of the structural materials, whereas it does not make very much difference whether the windows are all open initially or they break progressively as the fire spreads. In conclusion, a creditable worst-case scenario from a structural perspective is represented by just having all windows open from the beginning.

Table 14. Summary maximum temperatures with window breaking scenarios.

Scenario	Fire, upper layer max	Concrete @ 25 mm FDS	Concrete @ 25 mm 1D FD	Steel H_p/A=30 upper layer	Steel H_p/A=60 upper layer	Steel H_p/A=130 upper layer
Limited window breakage	837	206	148	177/70	267/107	341/166
Total window breakage	1300	498	495	**815**/127	**1034**/207	**1249**/334
Base case open windows FLED 800/100%	1297	504	498	**831**/129	**1082**/210	**1288**/338

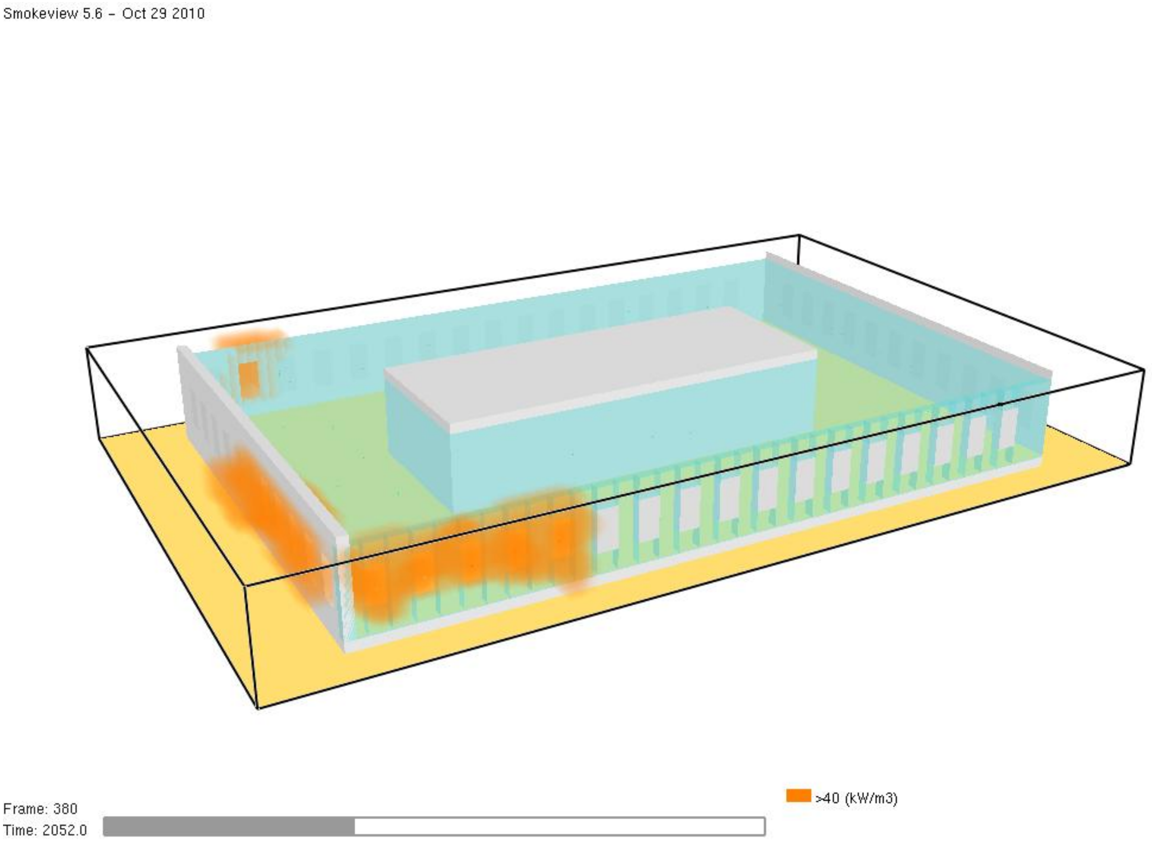


Figure 63. Smokeview image at ~34 minutes where limited window breakage has inhibited fire spread.

The temperatures in the upper layer for the two scenarios are shown in Figure 64 and Figure 65 where the difference is hundreds of °C.

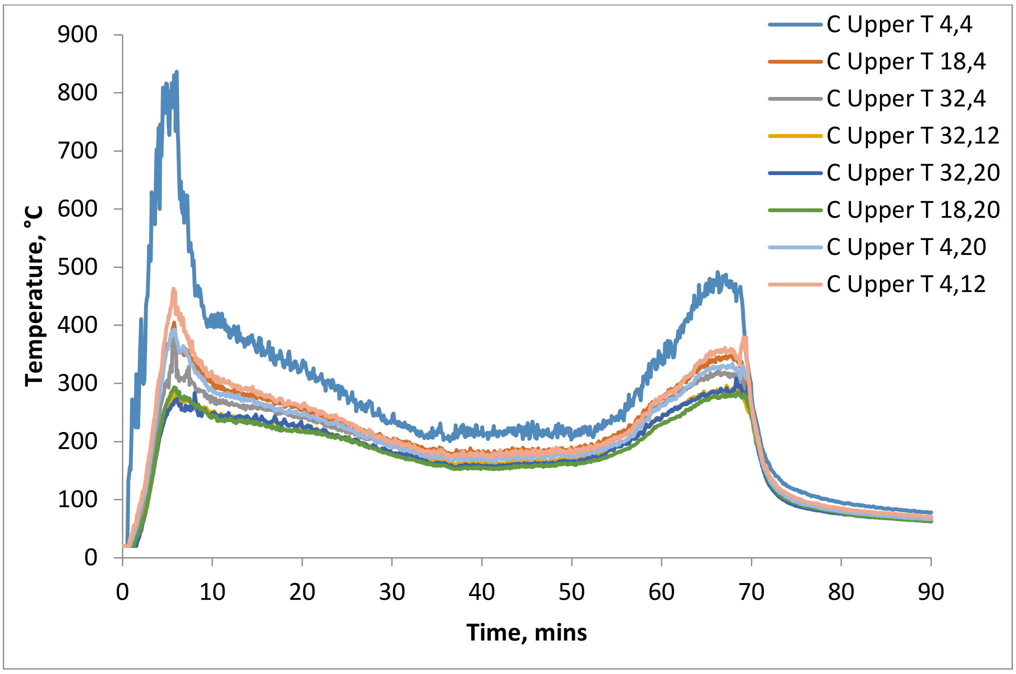


Figure 64. Upper layer gas temperatures with limited window breakage.

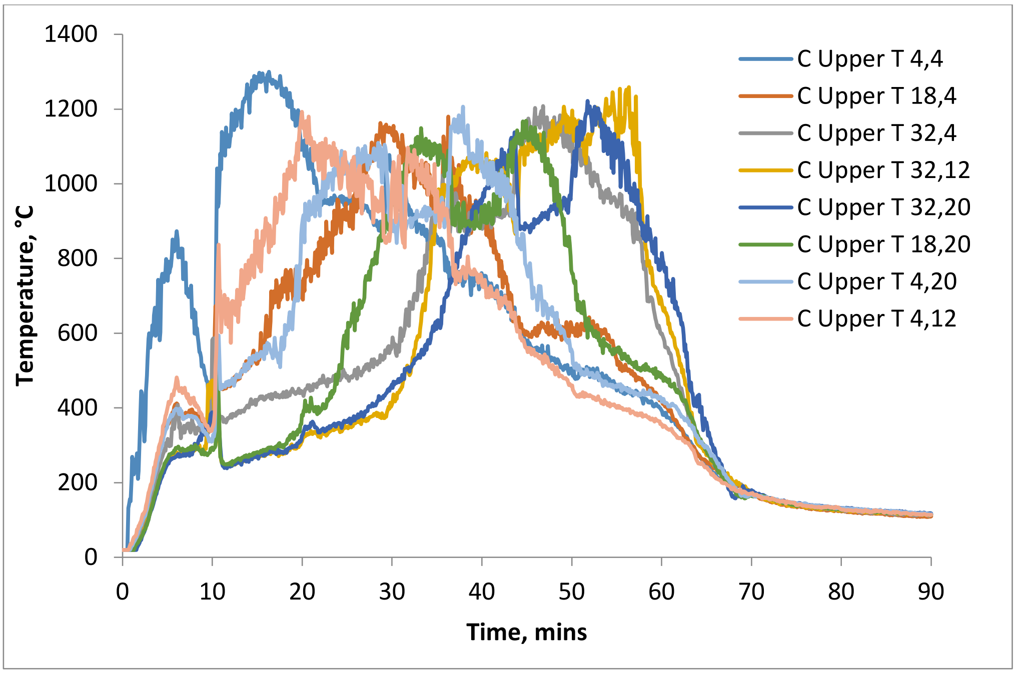


Figure 65. Upper layer gas temperatures with total window breakage.

The trend is the same for the lower layer in Figure 66 and Figure 67.

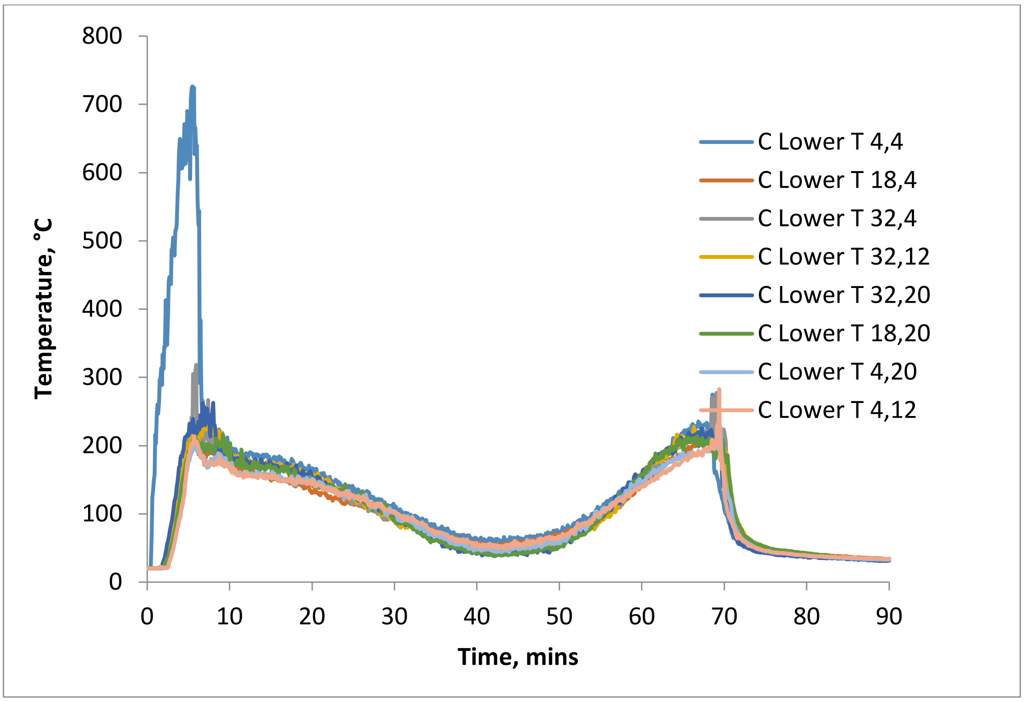


Figure 66. Lower layer gas temperatures with limited window breakage.

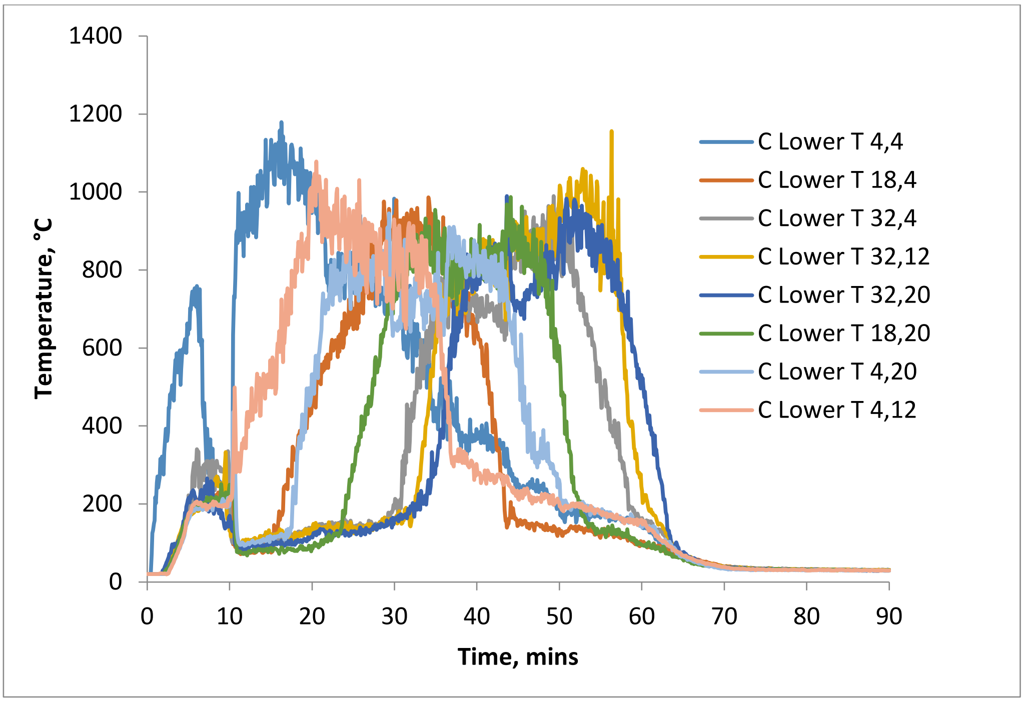


Figure 67. Lower layer gas temperatures with total window breakage.

The response of the concrete to the upper layer temperatures as calculated by FDS is shown in Figure 68 and Figure 69.

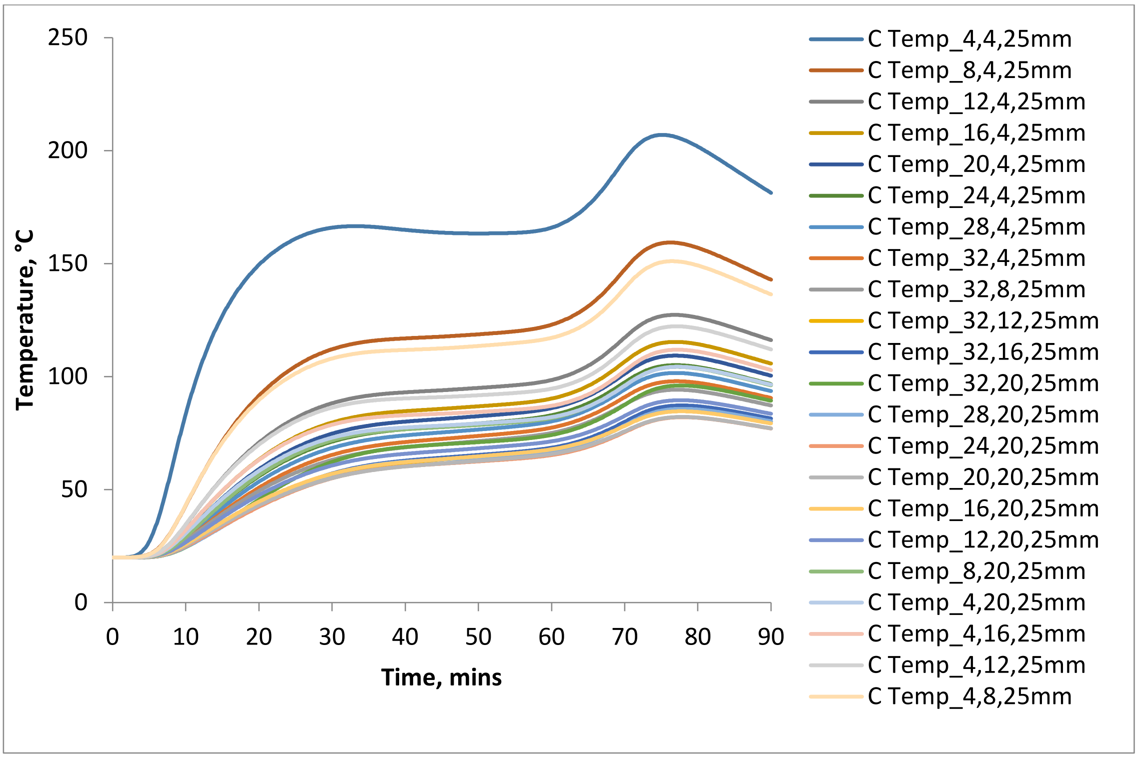


Figure 68. Concrete temperatures with limited window breakage.

The concrete temperatures at 25 mm depth for progressive window breakage are only marginally less than the case where the windows are all broken at the beginning of the fire scenario.

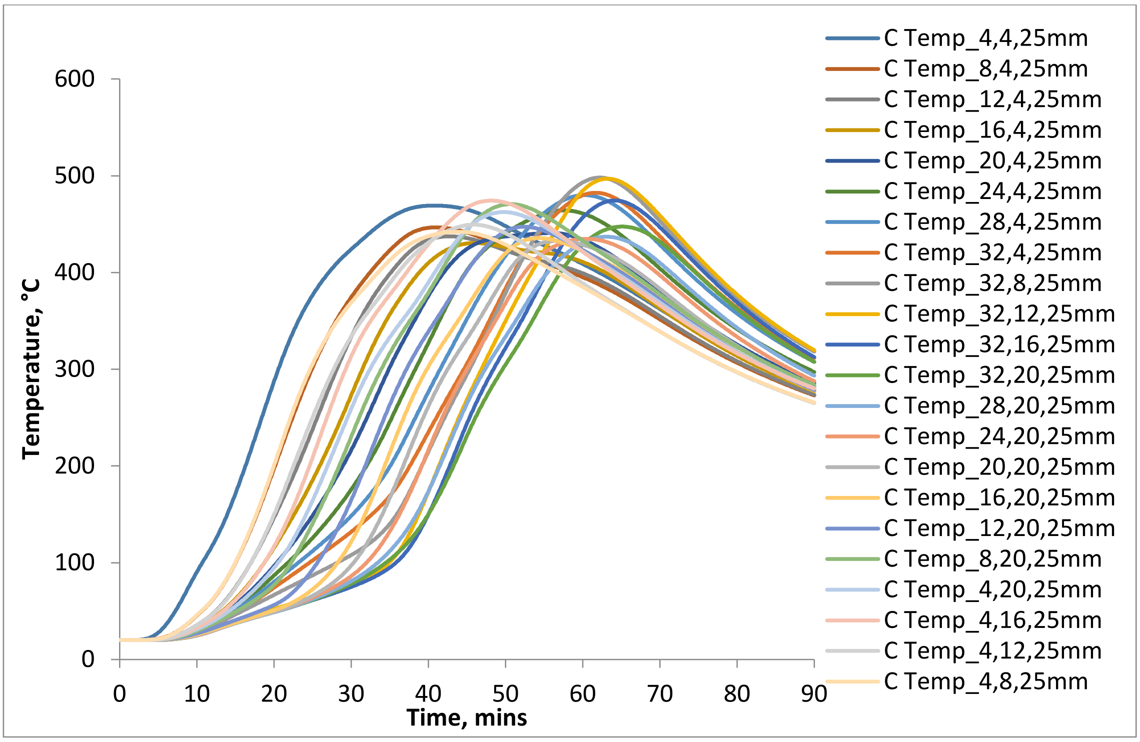


Figure 69. Concrete temperatures with total window breakage.

The temperature exposures are shown in Figure 70 for the fire origin and highest remote location in the scenario where window breakage does not follow the fire path.

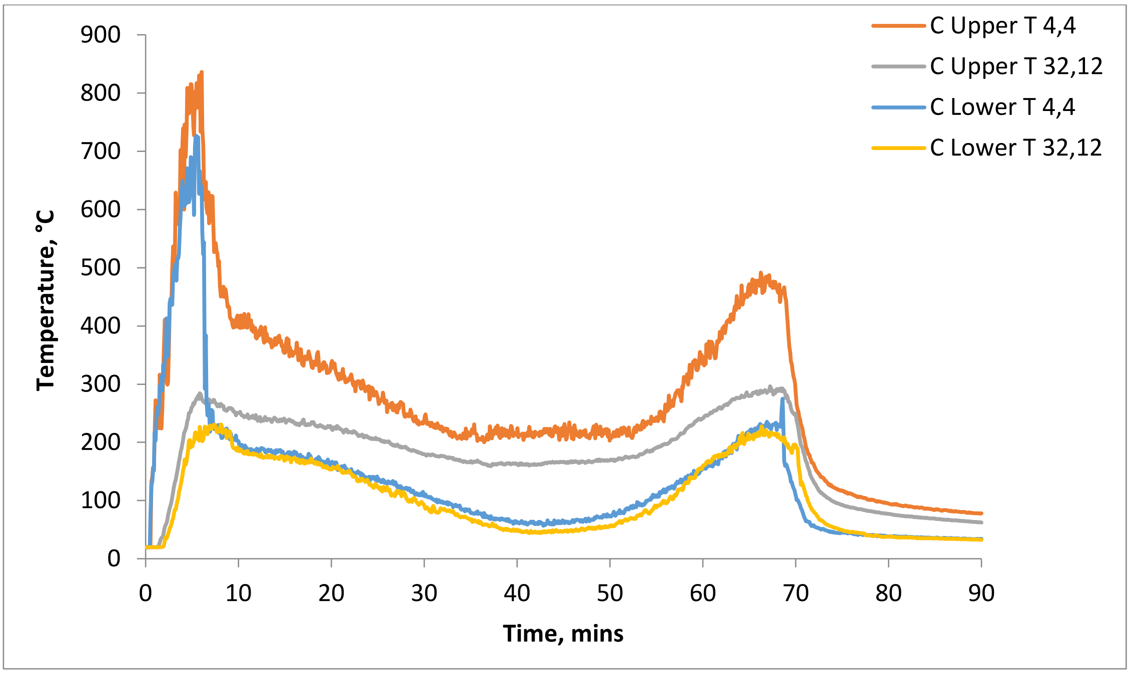


Figure 70. Upper and lower level temperatures in worst regions, with limited window breakage.

The response of the concrete to the fire in Figure 71 indicates that, when ventilation is limited, the temperature rises are similarly limited.

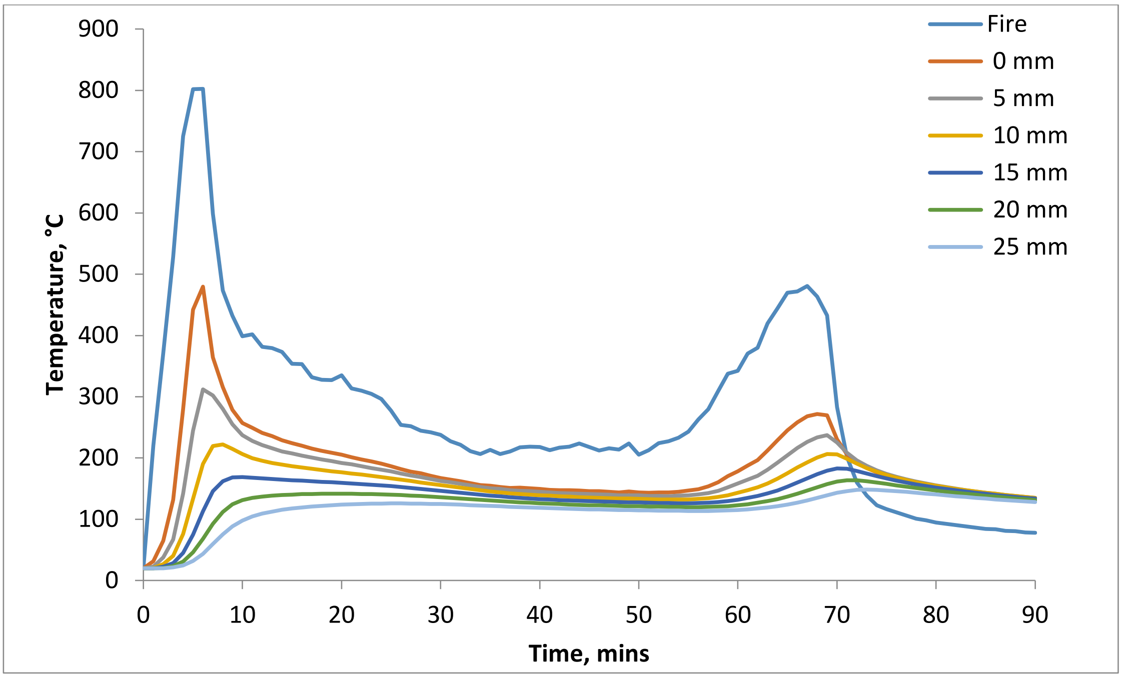


Figure 71. Concrete ceiling temperatures in worst region, with limited window breakage.

However, progressive window breakage following (or leading) the fire development results in almost identical temperatures when compared with the case of the windows being open at the beginning for Figure 72 and Figure 73.

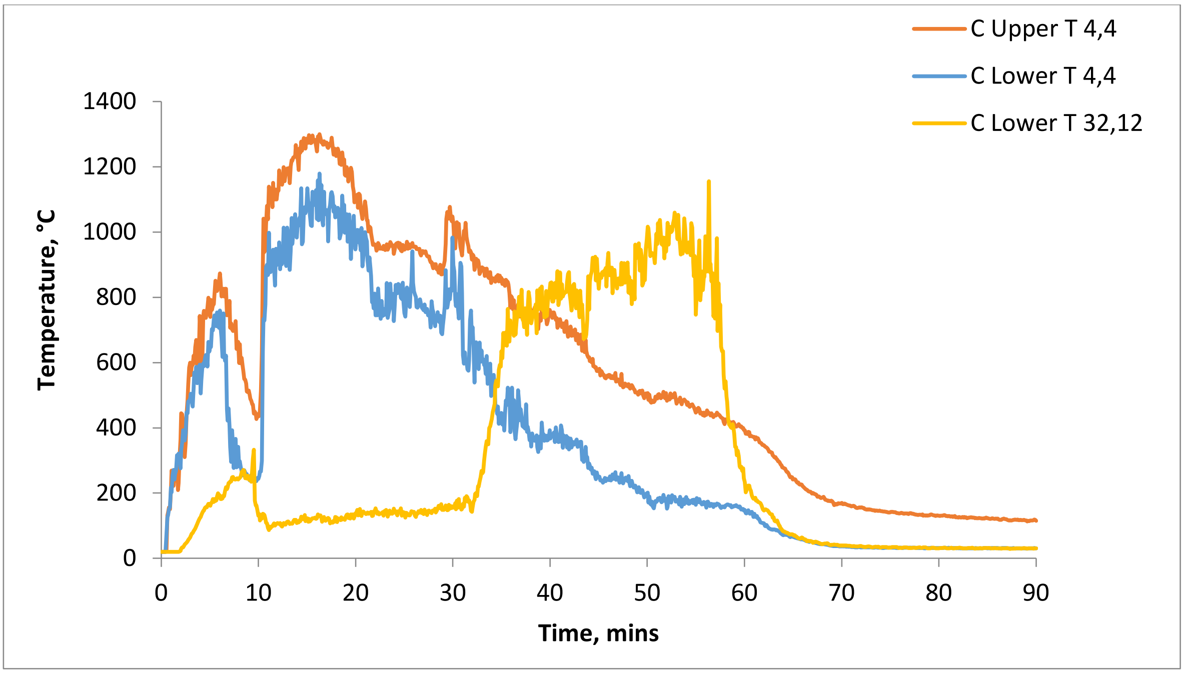


Figure 72. Upper and lower level temperatures in worst regions, with total window breakage.

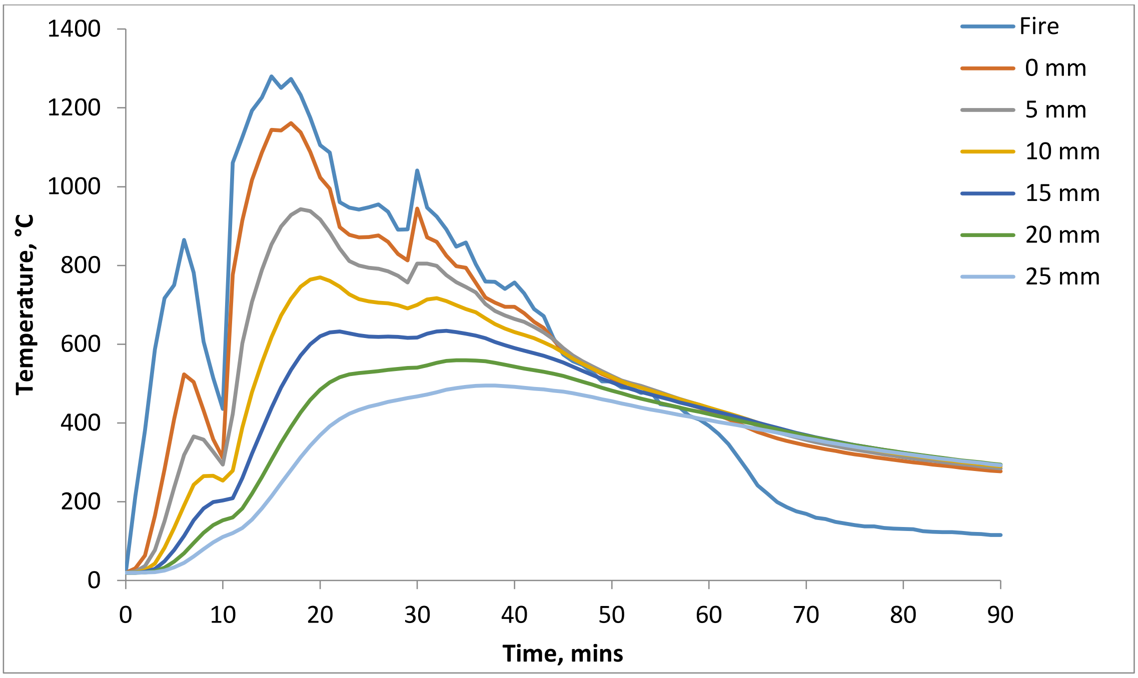


Figure 73. Concrete ceiling temperatures in worst region, with total window breakage.

Considering steel member temperatures for the range H_p/A=30–130 and two fire scenarios where there is limited window breakage and total progressive window breakage, the results are presented in Figure 74 to Figure 79.

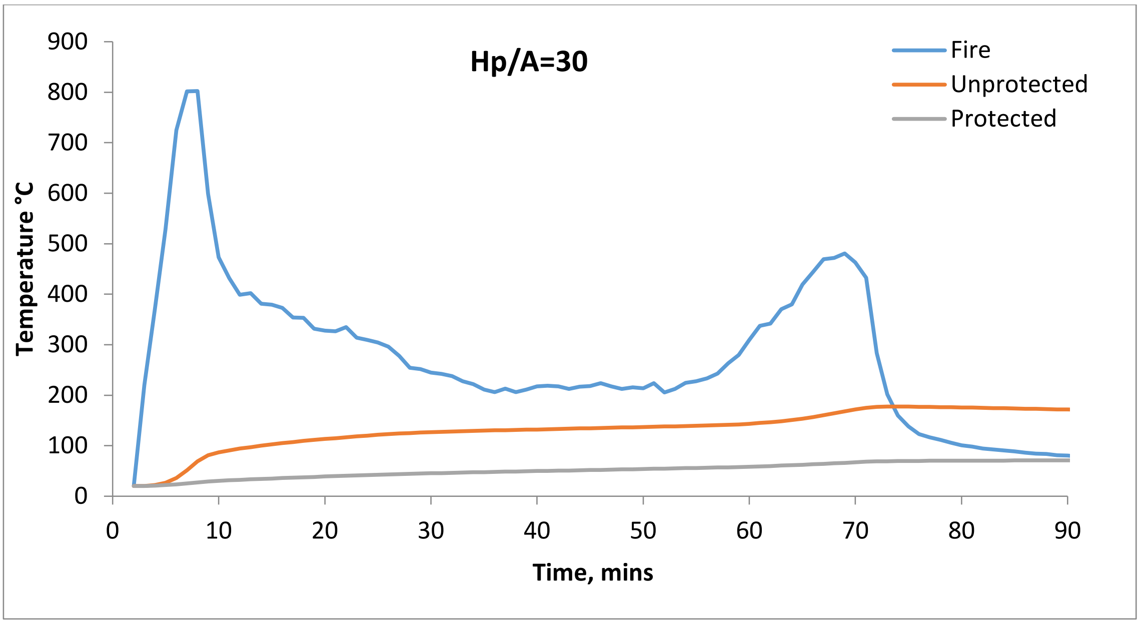


Figure 74. Steel temperatures (H_p/A=30), with limited window breakage, upper layer.

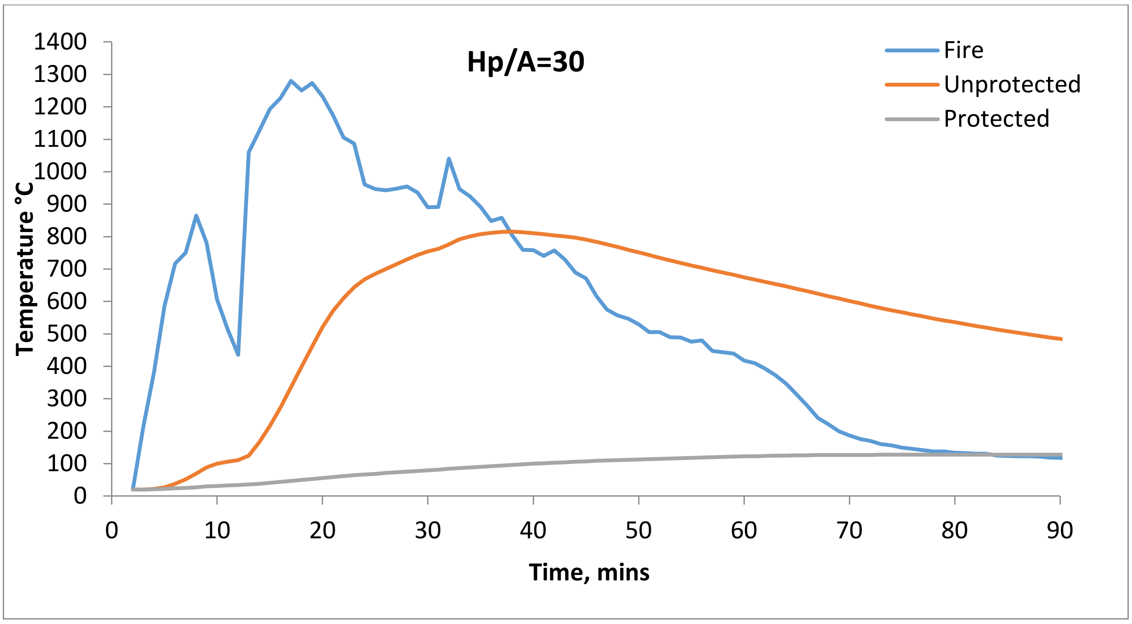


Figure 75. Steel temperatures (H_p/A=30), with total window breakage.

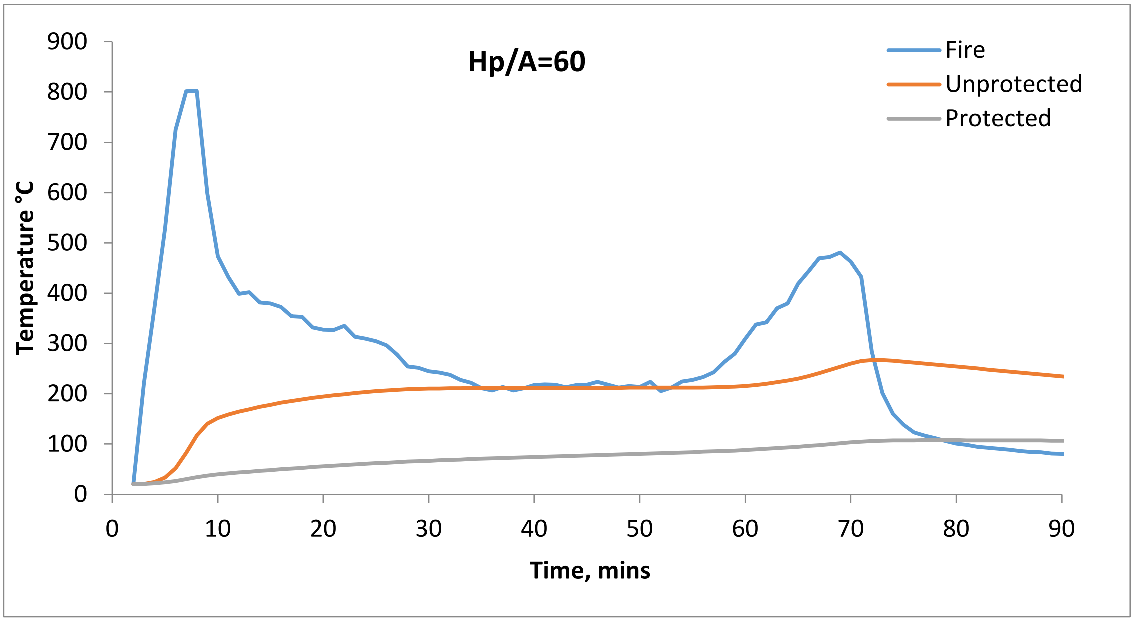


Figure 76. Steel temperatures (H_p/A=60), with limited window breakage, upper layer.

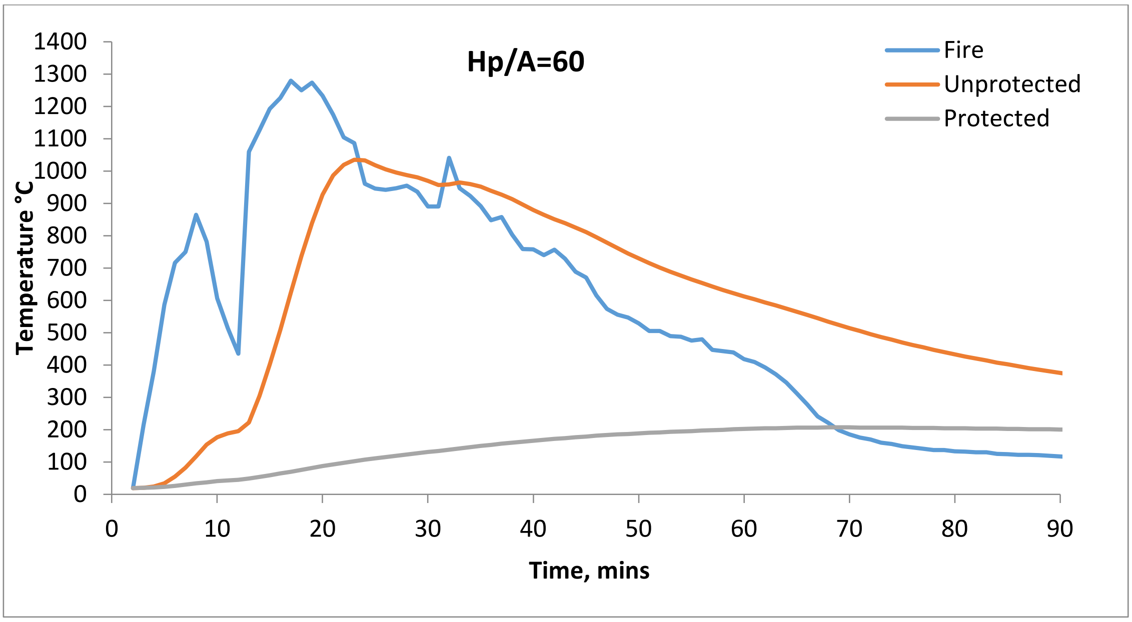


Figure 77. Steel temperatures (H_p/A=60), with total window breakage.

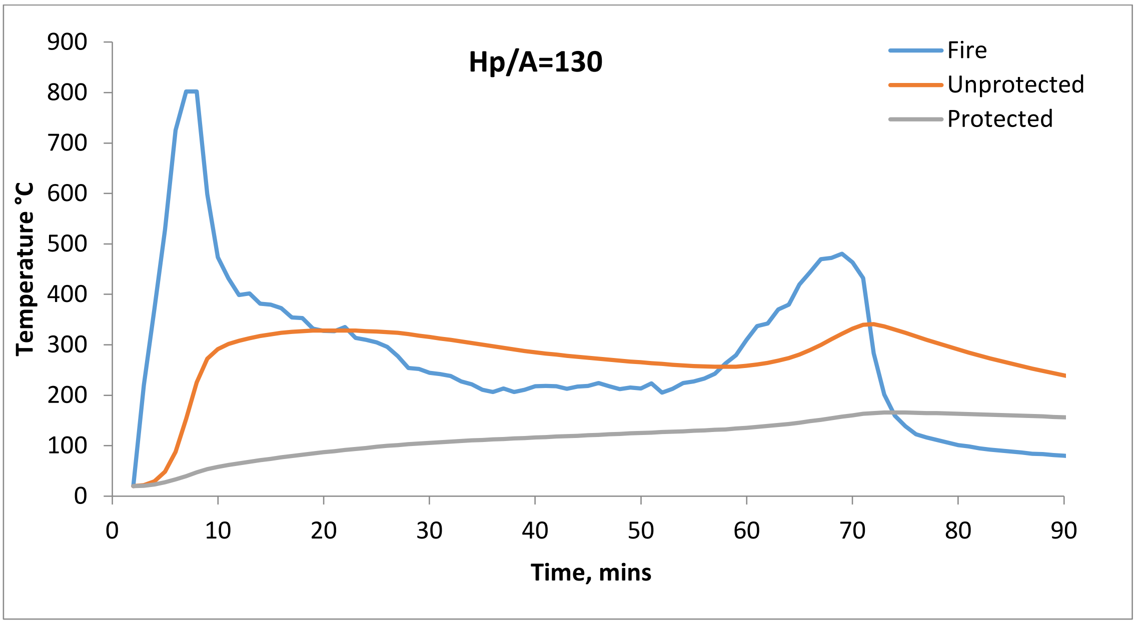


Figure 78. Steel temperatures (H_p/A=130), with limited window breakage, upper layer.

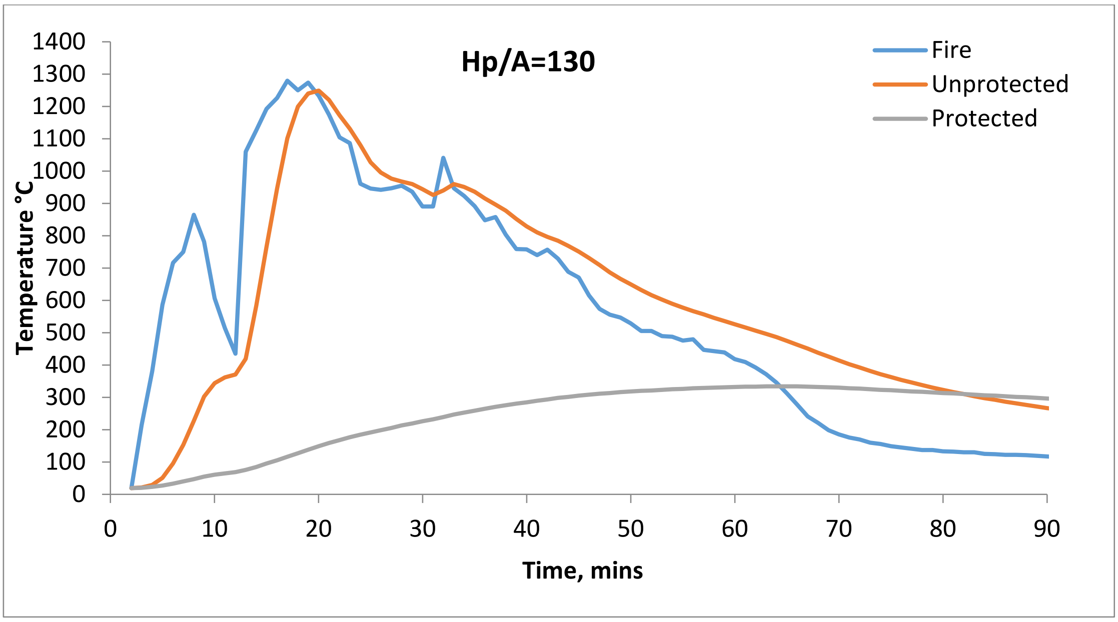


Figure 79. Steel temperatures (H_p/A=130), with total window breakage, upper layer.

Whether the windows break or not by whatever means they are opened in a modelling scenario has a significant effect on the temperature of the structural elements, so much so that it should always be considered that windows will partially or fully break for the worst-case scenario from a structural element consideration (of temperatures).

B.3 Travelling fires (Stern-Gottfried, 2011)

Depending on the material (concrete or steel). several trials with the fire exposure may be required to determine the worst-case exposure, such as:

- 25% of floor fire involvement with a 800°C far field for a relatively short exposure (56, 400 MJ/m^2)
- 10% of floor fire involvement with a 550°C far field for a relatively long exposure (140, 400 MJ/m^2)
- the timing of the arrival on the near field (1200°C) makes a difference where generally the longer the period of preheating, the higher the material temperature reached.

Possible exceptions are that:

- unprotected steel may reach a plateau before the arrival of the near field
- concrete within a member draws heat away from surface towards the cooler centre after the near fire has passed, which results in a peak temperatures after the fire (near field) has passed.

Example 1 continued from section A.3.4

To evaluate the temperature response in example 1, Eq.(4) to Eq.(9) are encoded into spreadsheets and used to evaluate the temperature response of concrete and steel members to the stepwise exposure of the travelling fires.

Considering a fire scenario with 25% of floor area involvement and an 800°C near field, Figure 80 shows the concrete temperature contours for the near field of 1200°C arriving after 14 minutes compared with a much later arrival at 42 minutes.

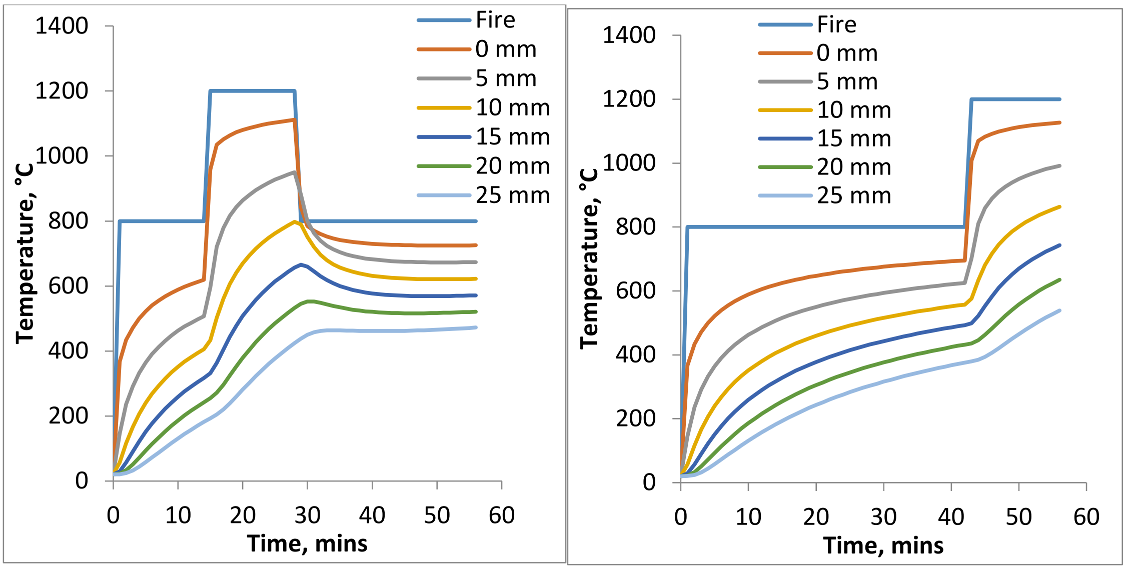


Figure 80. Concrete temperature contours with 25% floor area, travelling fire.

Table 15. Maximum concrete temperatures for 25% floor involvement.

Depth mm	Early °C	Late °C
20	549	634
25	467	538

The maximum temperatures reached are different for the two scenarios. The later arrival of the near field at 42 minutes is a worst case and likely to be the worst case as the fire event ends at 56 minutes. On the basis of the concrete temperatures achieved and a limitation of 550°C for the concrete and hence reinforcing steel temperature, 20 mm will be enough concrete cover (549°C) for the arrival of the fire (near field) at 14 minutes, while for the arrival of the fire (near field) at 42 minutes, 25 mm cover (539°C) would be required.

The alternative scenario with 10% of floor area involvement is considered to determine if this represents a worst case.

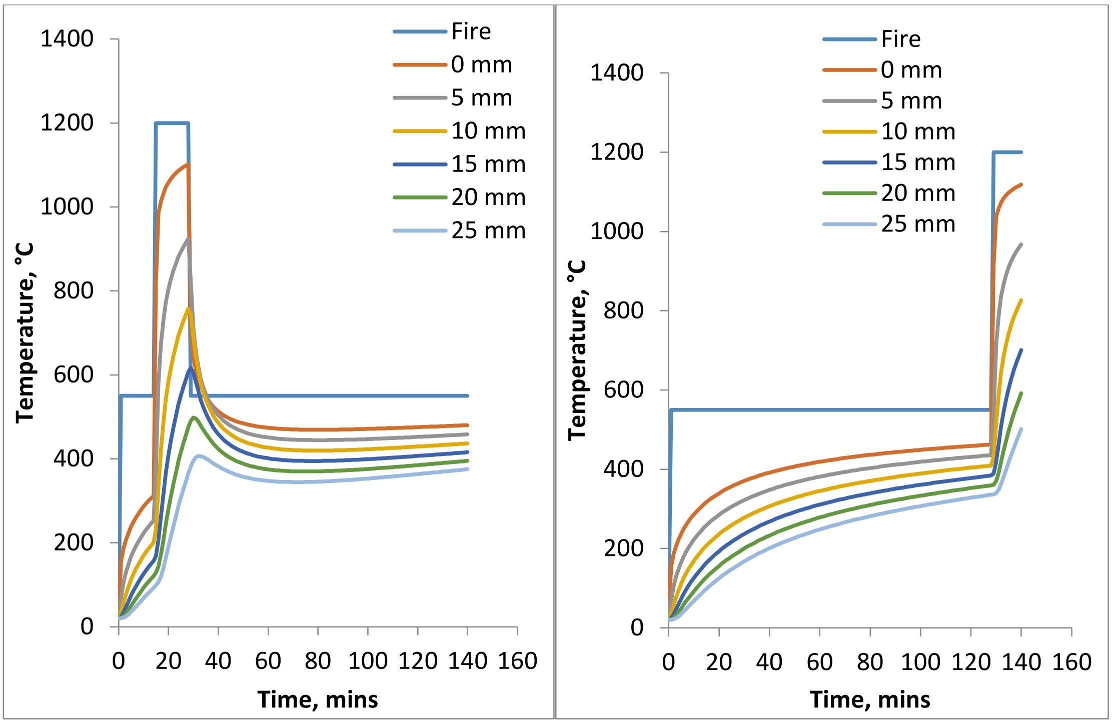


Figure 81. Concrete temperature contours with 10% floor area, travelling fire.

Table 16. Maximum concrete temperatures for 10% floor involvement.

Depth mm	Early °C	Late °C
20	490	591
20	381	501

For a 140-minute fire exposure, the temperature response of concrete is shown in Figure 81, with similar results to the 25% floor area case, where 20 mm depth of cover is adequate for the earlier arrival of the near field, but to cover the worst case, 25 mm is required to limit the temperature rise to below 550°C.

The scenario with 25% floor involvement represents a slightly worse case.

Similarly, the steel temperatures in Figure 82 and Figure 83 for a column with an H_p/A of 30 show that a later arrival of the fire (near field) results in higher temperatures for both the unprotected and protected steel. In each case, the unprotected steel temperature of 550°C is exceeded at 21 and 38 minutes respectively, so some applied protection (13 mm of mineral fibre) is required to limit the steel temperature below 550°C.

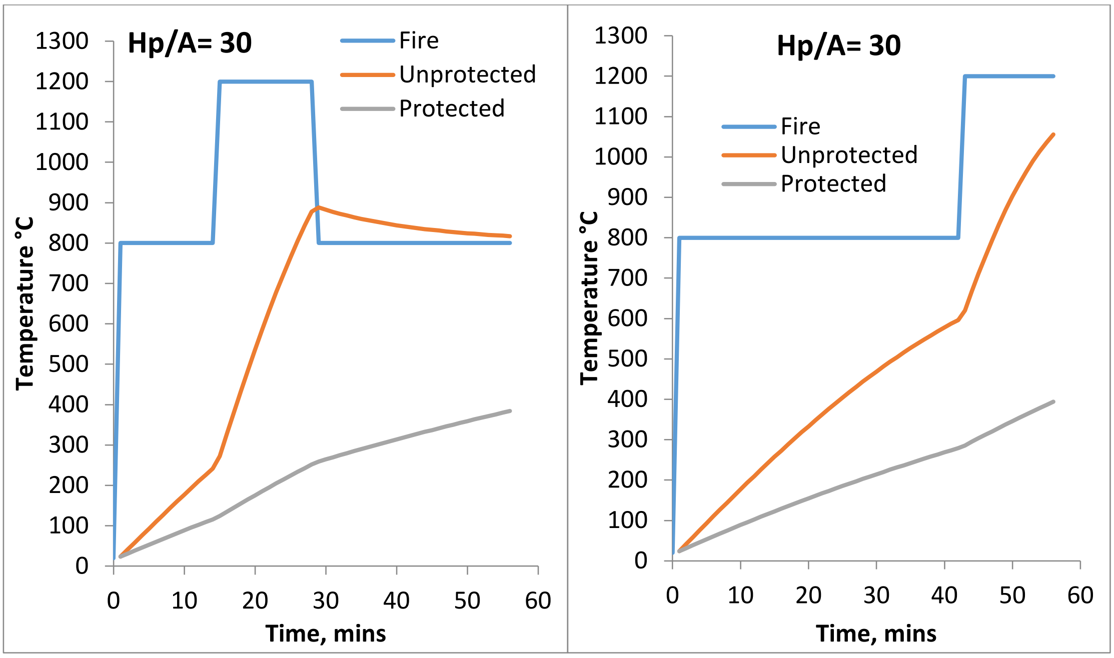


Figure 82. Steel member temperatures with 25% floor area, travelling fire.

Table 17. Maximum steel temperatures for 25% floor involvement.

	Early °C	Late °C
Unprotected	888	1055
Protected	380	394

For the 10% of floor area involvement, the steel temperatures in Figure 83 show the unprotected temperatures are not the worst case. If protection is added, it is worse.

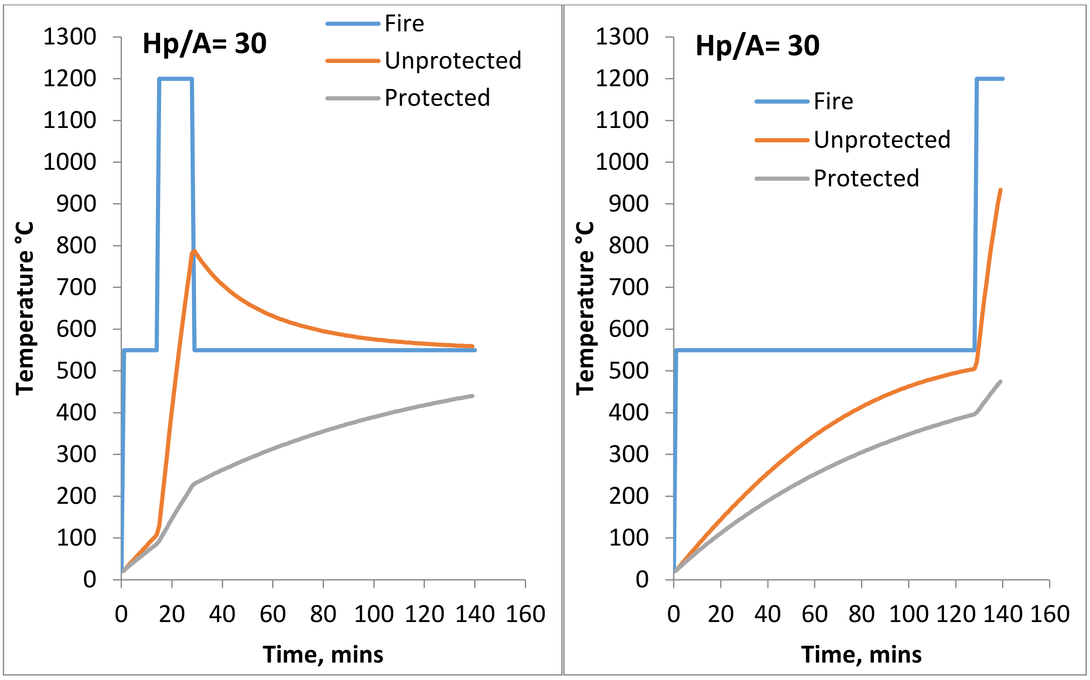


Figure 83. Steel member temperatures with 10% floor area, travelling fire.

Table 18. Maximum steel temperatures for 10% floor involvement.

	Early °C	Late °C
Unprotected	781	933
Protected	438	475

The worst scenario is if the arrival of the near field is later, whereby the structure has undergone the longest period of preheating before the arrival of the near field.

Compare this with the most favourable scenario where the initial exposure to the structural element under consideration is to the near field (1200°C) and then the far field as the fire moves away. The initial temperature rise of the steel member may not reach 550°C. Thereafter, the maximum temperature it will reach is 550°C (Figure 84).

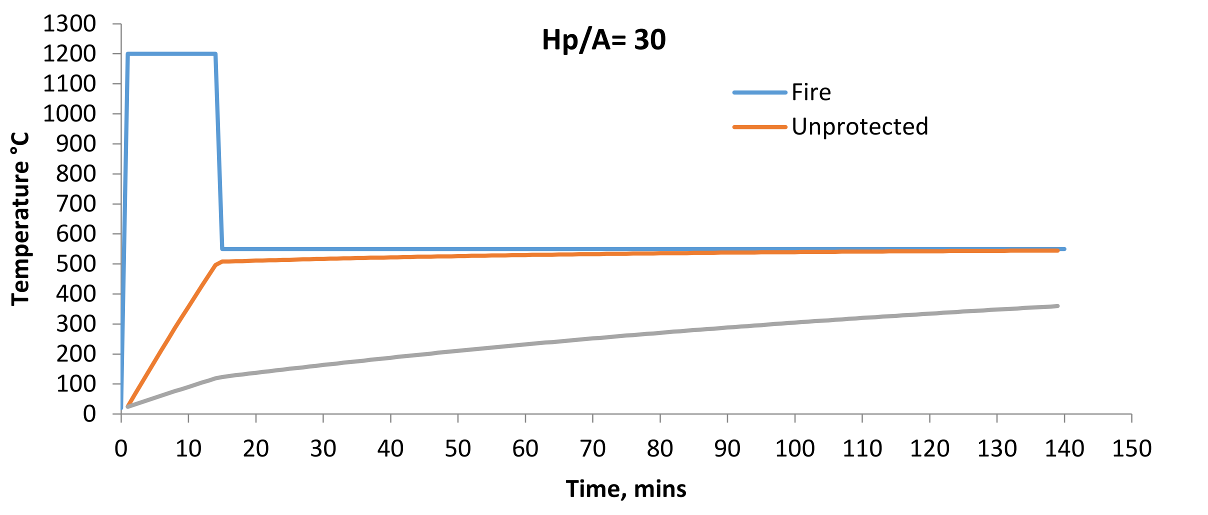


Figure 84. Steel temperatures with early arrival of fire.

Summary of maximum temperatures

Table 19 shows the maximum temperatures of the structural elements for the various fire scenarios, the temperatures in bold representing the worst case. In the majority of cases, the highest temperature is attained with the late arrival of the near field and 25% floor involvement. The exception is that a longer heating time at a lower (far field) temperature represents a worst case for steel that is protected. The rationale for that is that lower-level exposure for a longer period results in a steady build-up of temperature compared with a shorter period at a higher temperature. Therefore, both the 10% and 25% scenarios need to be compared to determine a worst case.

Table 19. Maximum structure temperatures.

Time of near field	Early °C		Late °C	
Floor involvement	10%	25%	10%	25%
Concrete 20 mm	490	549	591	**634**
Concrete 25 mm	381	467	501	**538**
Steel unprotected	781	888	933	**1055**
Steel protected	438	380	**475**	394

B.4 Energy balance

The energy released in the fire scenarios considered above are compared in Table 20 as a means of reconciling and justifying that a travelling fire scenario covers off the majority of the other methods of assessing fire exposure.

Table 20. Energy balance of enclosure fires.

Parametric fires				
FLED, MJ/m²	400	400	800	800
Ventilation, %	100	50	100	50
THR, MJ/m²	119	156	266	355
Peak heat flux, kW/m²	90	65	130	91
Travelling fires (FDS)				
FLED, MJ/m²	400	400	800	800
Ventilation, %	100	50	100	50
THR, MJ/m²	237	233	366	302
Peak heat flux, kW/m²	316	264	349	272
Travelling fires (Stern-Gottfried, 2011)				
FLED, MJ/m²	400	400	800	800
% floor	25	10	25	10
THR, MJ/m²	425	431	838	849
Peak heat flux, kW/m²	267	267	267	267
ISO fires				
FRR, min	60	120	180	240
THR, MJ/m²	293	836	1,524	2,320
Peak heat flux, kW/m²	124	173	207	234

In order to justify the application of travelling fires (Stern-Gottfried et al., 2011) for the assessment of structural elements as opposed to parametric fires and FDS modelled fires, an energy balance is presented in Table 20. The parameters considered for the comparison are the energy (THR) over the fire duration in MJ/m^2 impinging on an element surface and peak heat flux in kW/m^2. The THR was calculated using Eq.(10) for the time-temperature exposures, where the emissivity ε is assumed to be unity.

$$Energy(THR) = \varepsilon\sigma \int T^4 dt \qquad \text{Eq.(10)}$$

The peak heat flux was determined from the stepwise increases in the THR. In all cases, the THR of the travelling fire exceeds the other modelling scenarios. A comparison with the ISO fire exposure is included showing that 60 and 120 minutes are comparable with FLEDs of 400 and 800 MJ/m^2.

The stepwise travelling fires for the 400 and 800 MJ/m^2 FLED and 25% floor involvement conditions are compared with the FDS modelled fires in Figure 85, Figure 86 and Figure 87. In each case, it is clearly evident that travelling fire exposure exceeds by a considerable margin the FDS modelled fires. The 25% level was chosen as the fire durations of 56 or 112 minutes are comparable with the FDS scenario duration.

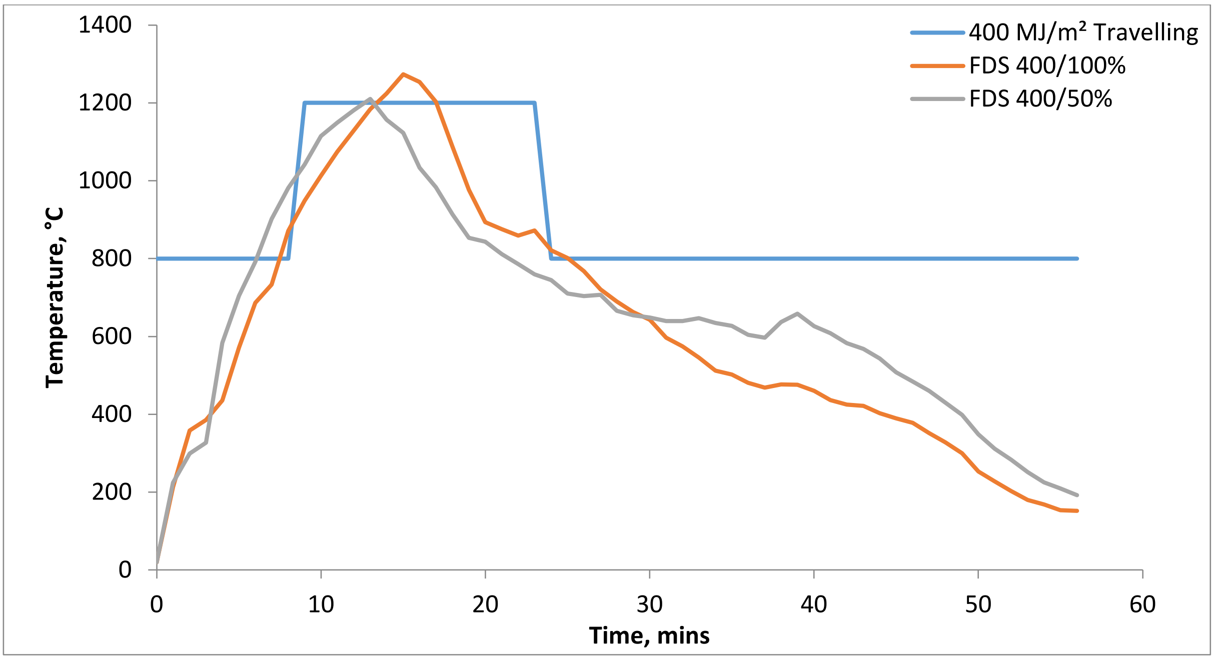


Figure 85. Travelling fire superimposed on FDS trials for FLED 400 MJ/m².

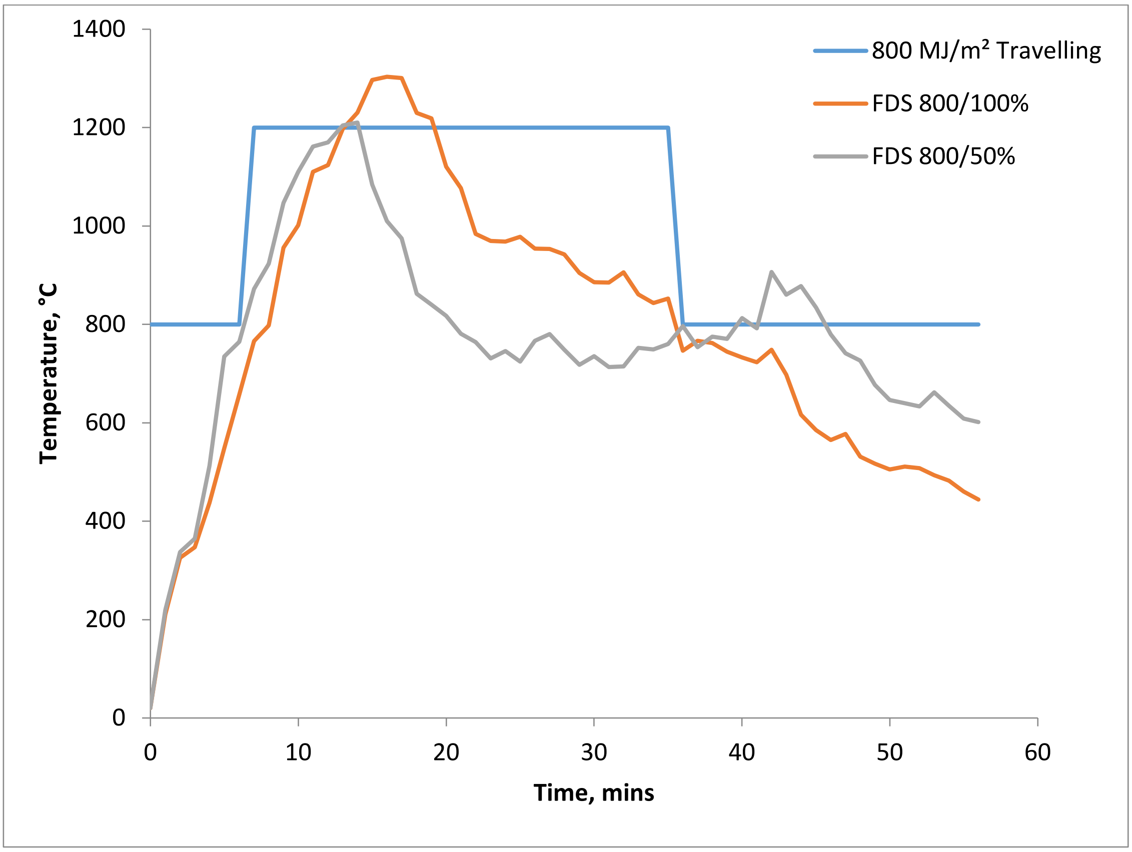


Figure 86. Travelling fire superimposed on FDS trials for FLED 800 MJ/m².

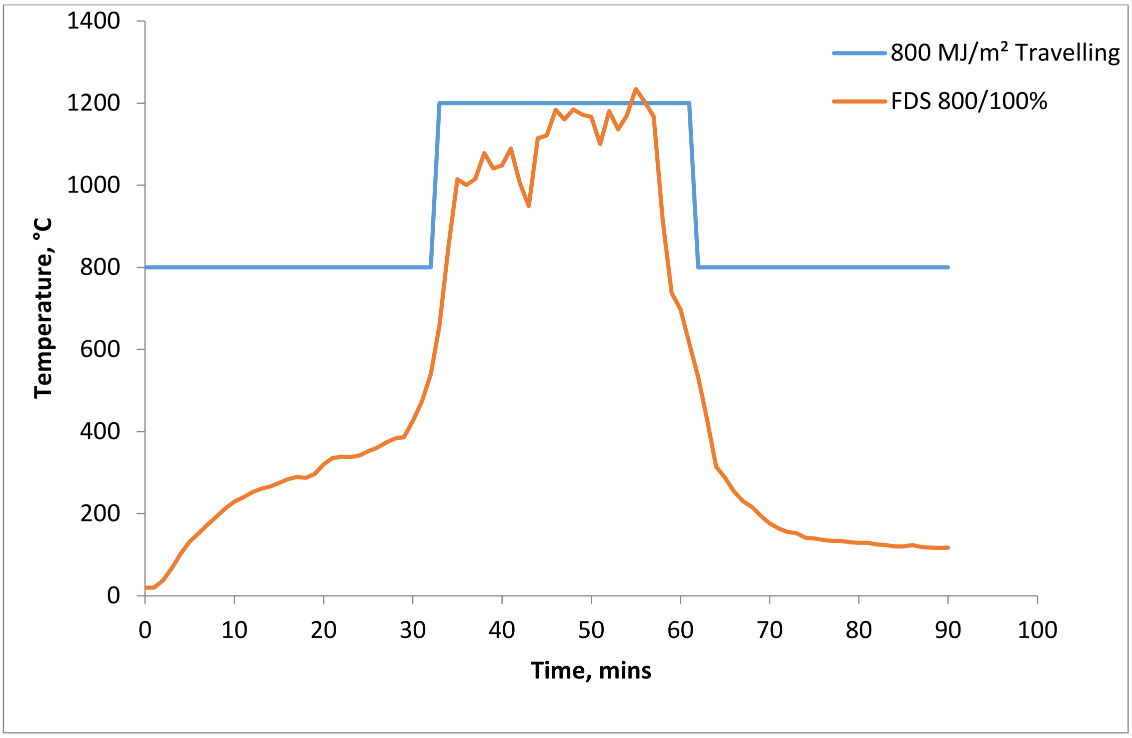


Figure 87. Travelling fire superimposed on FDS trial for FLED 800 MJ/m^2 remote from fire origin.

In conclusion, the application of the stepwise travelling fire scenario presents a simplified and easily applicable means that is conservative and likely to cover a wide range of conceivable eventualities where the only inputs are the FLED and the choice of 10% or 25% occupancy of the floor area by the fire at any one time.

B.5 Finite element modelling

The temperature response of the concrete and steel structural elements as determined using the methods above (Incropera, DeWitt, Bergman and Levine, 2007; Buchanan, 2001) (1D FD and LMHT) for an ISO fire exposure and the exposure to most severe fire (800 MJ/m^2 and 100% ventilation) were compared with the finite element software SAFIR.

Results indicate some acceptably close agreement between spreadsheet-based methods in one dimension compared with SAFIR in two dimensions. Where there are differences, these can partially be attributed to the convection and radiation parameters at the boundaries and physical properties of the concrete, in particular, how water of hydration is handled.

A screen image of a 200 mm thick concrete slab supported by an I beam is shown in Figure 88.

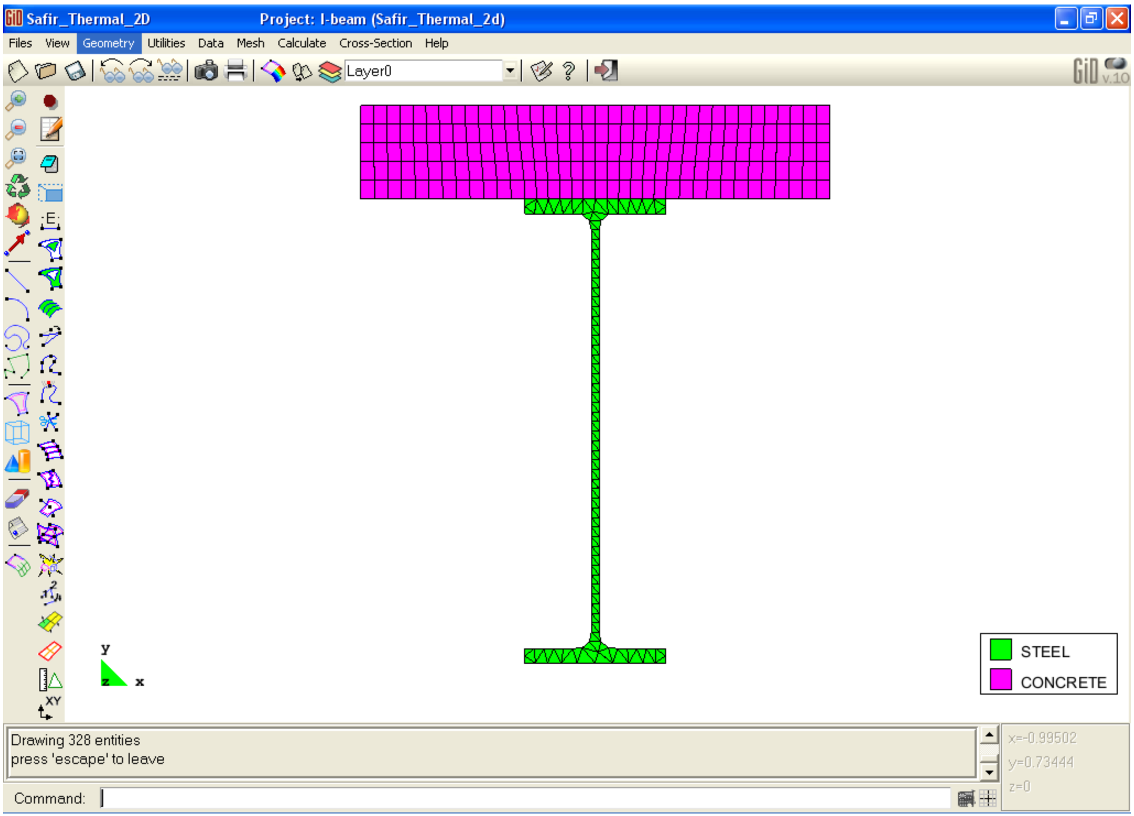


Figure 88. SAFIR analysis of 200 mm concrete slab supported by I beam.

The I beam (1016 x 305 mm, 272 kg/m, H_p/A=80) and concrete slab were subjected to an ISO fire and the travelling fire scenario with progressive window breakage (800 MJ/m^2 and 100% ventilation).

The temperature contours for the ISO fire exposure are shown at 30, 60 and 90 minutes in Figure 89, Figure 90 and Figure 91.

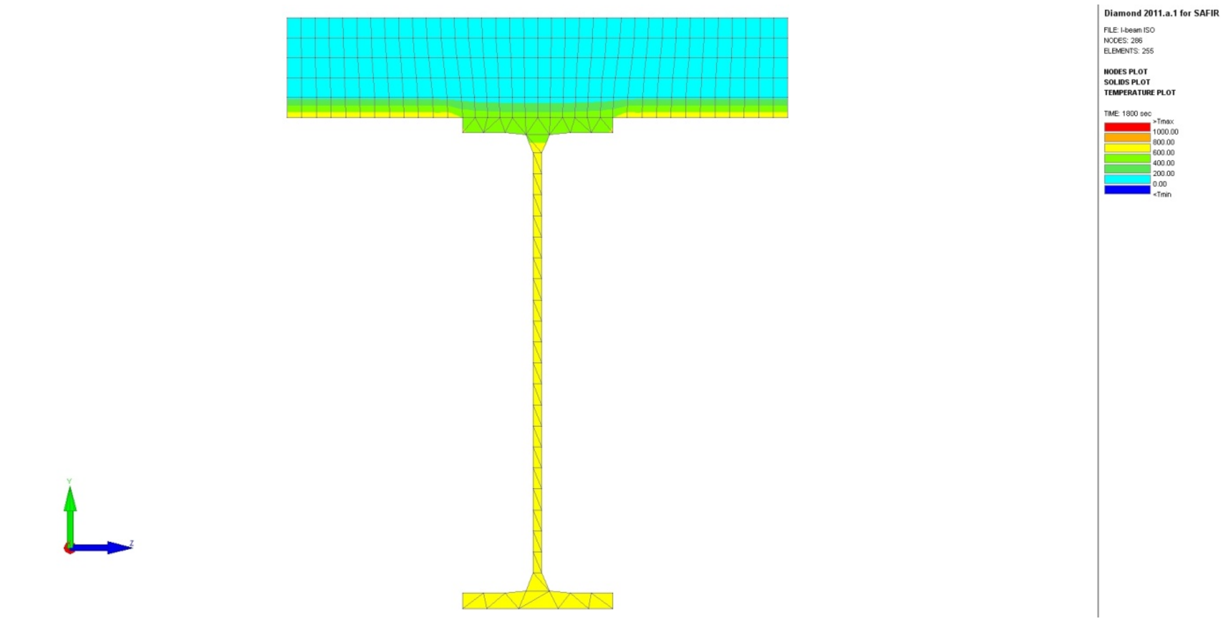


Figure 89. Temperature contours at 30 minutes exposure to ISO fire.

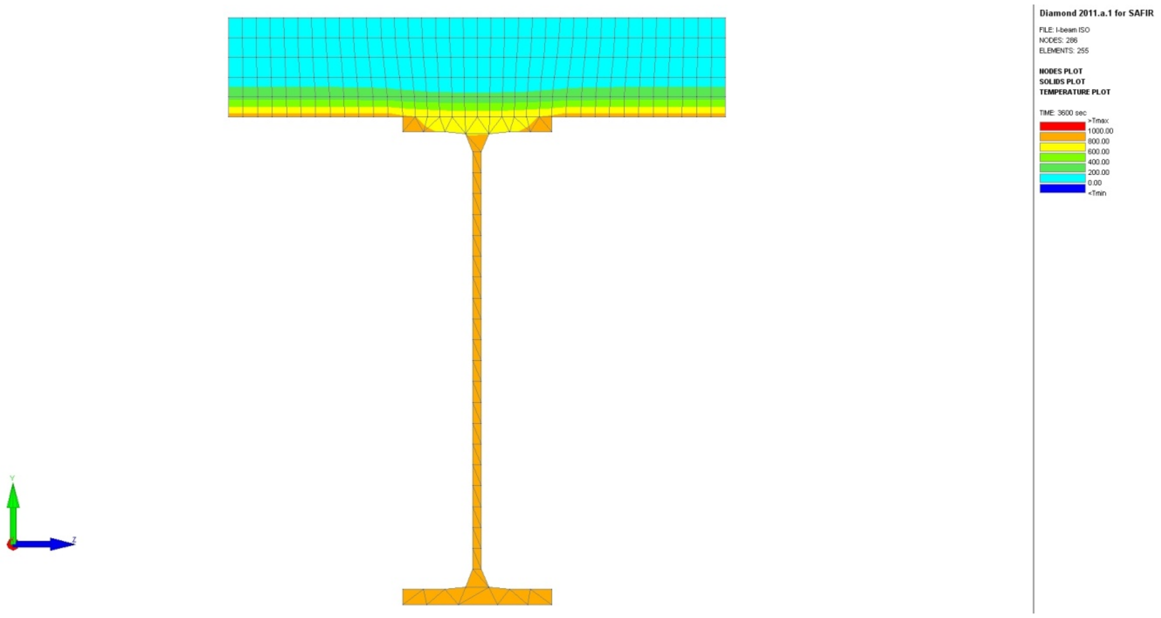


Figure 90. Temperature contours at 60 minutes exposure to ISO fire.

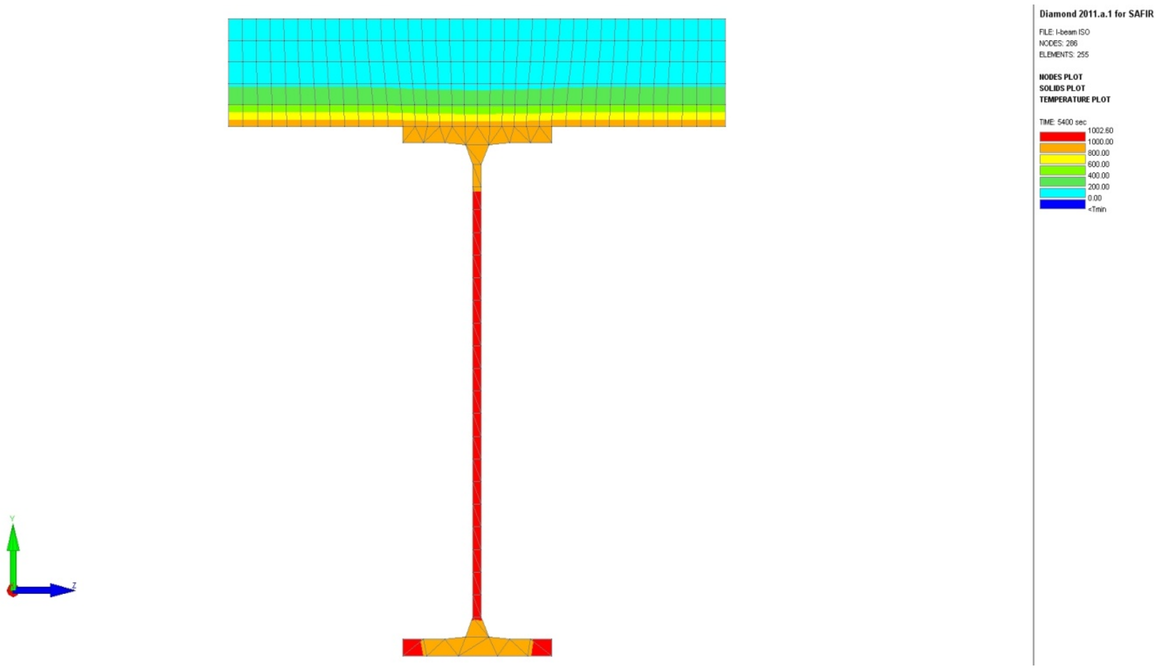


Figure 91. Temperature contours at 90 minutes exposure to ISO fire.

The temperature responses for the concrete at progressive depths are compared in Figure 92 as calculated by SAFIR and the 1D FD method.

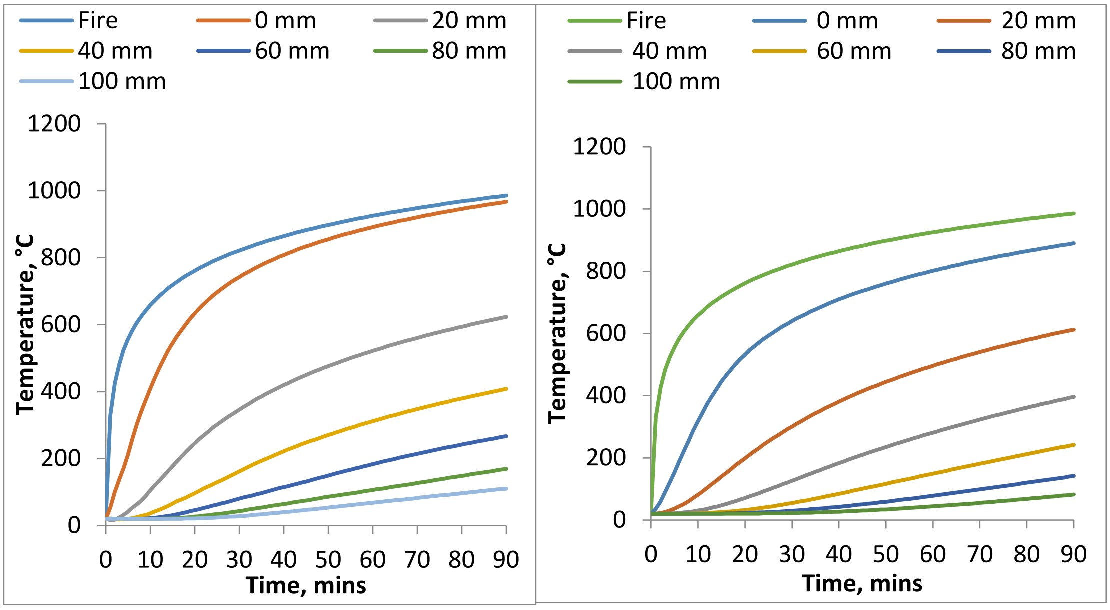


Figure 92. Comparing SAFIR (left) and 1D FD method generated concrete temperatures for ISO fire exposure.

Similarly, the steel temperatures are compared in Figure 93.

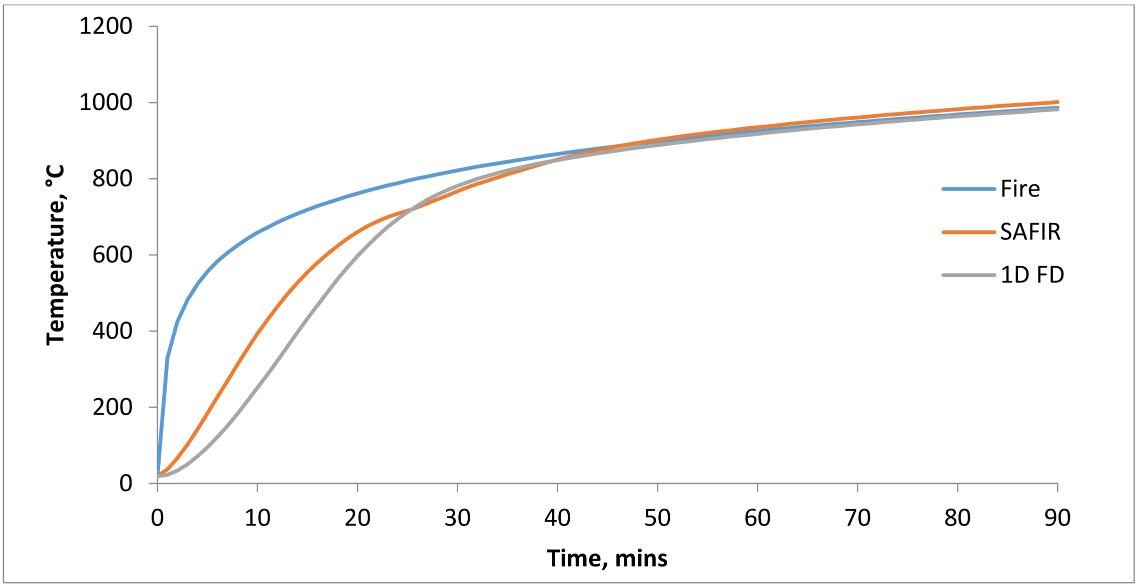


Figure 93. Comparing steel temperatures of I beam (H_p/A=80) for SAFIR and LMHT method.

For the travelling fire with an FLED of 800 MJ/m^2 and 100% ventilation, the comparison for concrete is shown in Figure 94.

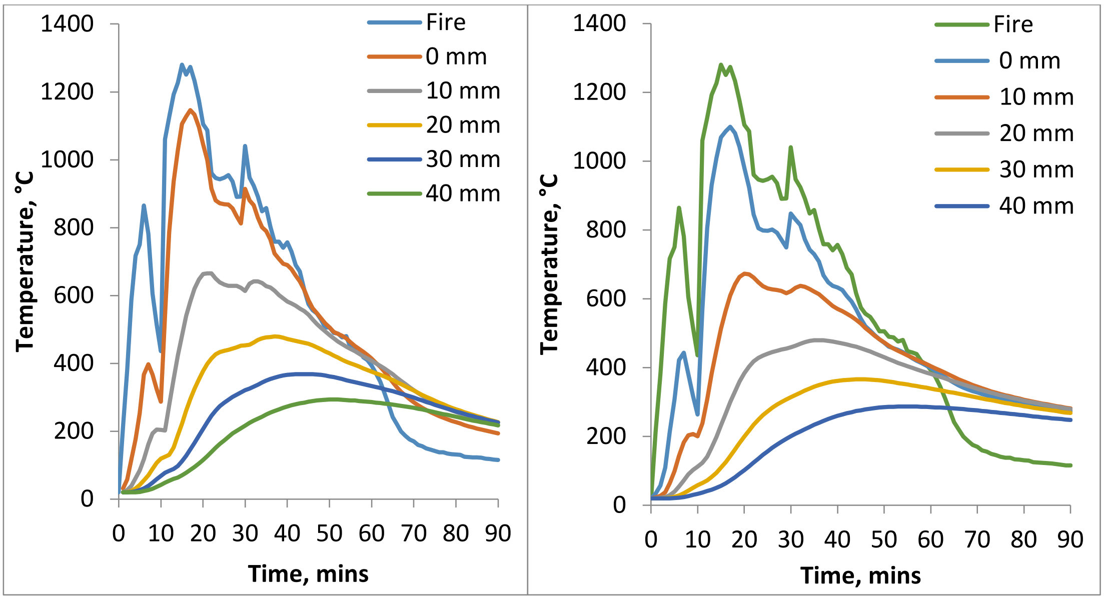


Figure 94. Comparing SAFIR-generated concrete temperatures (left) with 1D FD method (right) for 800 MJ/m² 100% ventilation fire.

The temperature responses for the steel I beam are compared in Figure 95 where, in the case of the SAFIR prediction, the steel temperature is an average for the section.

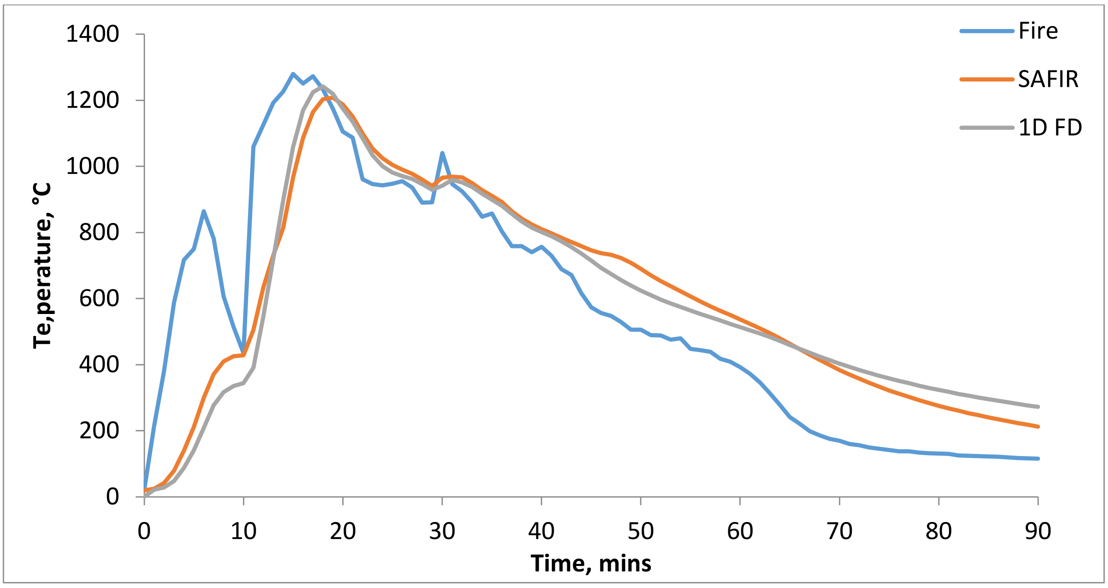


Figure 95. Comparing SAFIR-generated steel I beam ($H_p/A=80$) temperatures with LMHT method for 800 MJ/m² 100% ventilation fire.

The SAFIR predicted temperature response for concrete and steel shows generally close comparison between predictions using an SS based 1D FD and LMHT calculations, so a first pass estimation of material temperatures using the simplified methods offers a viable means of assessing structural element temperatures for design purposes.

B.5.1 Structural response

The response of structural elements to heating by fire requires the following considerations:

- Loss of strength in steel at elevated temperatures.
- Uneven heating and then cooling as a travelling fire moves through a compartment, causing uneven loading.
- Deflection or failure of some parts putting additional load and stress on other parts.

B.5.2 Other considerations

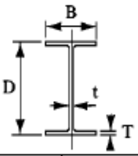
Not specifically considered in this study is the possibility of hot spots being created by air flows through gaps, perhaps between floors that entrain already hot but partially unburnt gases and air. The possibility exists that such mixing of the two may result in localised burning at very high temperatures (~1200°C) that may cause considerable damage and loss of strength to the connections holding beams to floors.

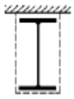
Appendix C: Selection of steel sections

Table 3: October 2006

UK Beams (UKB)

Dimensions to BS4 Part 1:2005



Designation		Depth of section D	Width of section B	Thickness		Area of section	Section factor A/V(Hp/A)			
							Profile		Box	
Serial size	Mass per metre			Web t	Flange T		3 sides	4 sides	3 sides	4sides
										
mm	kg	mm	mm	mm	mm	cm²	m⁻¹	m⁻¹	m⁻¹	m⁻¹
1016 x 305	487	1036.1	308.5	30.0	54.1	619.89	45	50	40	45
	438	1025.9	305.4	26.9	49.0	556.62	50	55	40	50
	393	1016.0	303.0	24.4	43.9	500.24	55	65	45	55
	349	1008.1	302.0	21.1	40.0	445.15	65	70	50	60
	314	1000.0	300.0	19.1	35.9	400.41	70	80	55	65
	272	990.1	300.0	16.5	31.0	346.86	80	90	65	75
	249	980.2	300.0	16.5	26.0	316.88	90	95	70	80
	222	970.3	300.0	16.0	21.1	282.82	95	110	80	90
914 x 419	388	921.0	420.5	21.4	36.6	494.22	60	70	45	55
	343	911.8	418.5	19.4	32.0	437.30	70	80	50	60
914 x 305	289	926.6	307.7	19.5	32.0	368.27	75	80	60	65
	253	918.4	305.5	17.3	27.9	322.83	85	95	65	75
	224	910.4	304.1	15.9	23.9	285.64	95	105	75	85
	201	903.0	303.3	15.1	20.2	255.92	105	115	80	95
838 x 292	226	850.9	293.8	16.1	26.8	288.56	85	100	70	80
	194	840.7	292.4	14.7	21.7	246.82	100	115	80	90
	176	834.9	291.7	14.0	18.8	224.02	110	125	90	100
762 x 267	197	769.8	268.0	15.6	25.4	250.64	90	100	70	85
	173	762.2	266.7	14.3	21.6	220.37	105	115	80	95
	147	754.0	265.2	12.8	17.5	187.19	120	135	95	110
	134	750.0	264.4	12.0	15.5	170.58	130	145	105	120
686 x 254	170	692.9	255.8	14.5	23.7	216.83	95	110	75	90
	152	687.5	254.5	13.2	21.0	194.08	105	120	85	95
	140	683.5	253.7	12.4	19.0	178.43	115	130	90	105
	125	677.9	253.0	11.7	16.2	159.48	130	145	100	115
610 x 305	238	635.8	311.4	18.4	31.4	303.33	70	80	50	60
	179	620.2	307.1	14.1	23.6	228.08	90	105	70	80
	149	612.4	304.8	11.8	19.7	190.04	110	125	80	95
610 x 229	140	617.2	230.2	13.1	22.1	178.19	105	120	80	95
	125	612.2	229.0	11.9	19.6	159.34	115	130	90	105
	113	607.6	228.2	11.1	17.3	143.94	130	145	100	115
	101	602.6	227.6	10.5	14.8	128.92	145	160	110	130
610 x 178	100	607.4	179.2	11.3	17.2	128.00	135	150	110	125
	92	603.0	178.8	10.9	15.0	117.00	145	160	120	135
	82	598.6	177.9	10.0	12.8	104.00	160	180	130	150
533 x 312	273	577.1	320.2	21.1	37.6	348.00	60	70	40	50
	219	560.3	317.4	18.3	29.2	279.00	70	85	50	65
	182	550.7	314.5	15.2	24.4	231.00	85	100	60	75
	151	542.5	312.0	12.7	20.3	192.00	105	120	75	90
533 x 210	138	549.1	213.9	14.7	23.6	176.00	95	110	75	85
	122	544.5	211.9	12.7	21.3	155.39	110	120	85	95
	109	539.5	210.8	11.6	18.8	138.86	120	135	95	110
	101	536.7	210.0	10.8	17.4	128.67	130	145	100	115
	92	533.1	209.3	10.1	15.6	117.38	140	160	110	125
	82	528.3	208.8	9.6	13.2	104.69	155	175	120	140

continued overleaf

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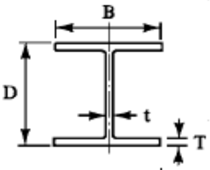
Table 4: October 2006

Columns (UKC)

Dimensions to BS4 Part 1:2005

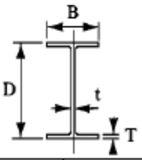


Designation				Thickness			Section factor A/V(Hp/A)			
							Profile		Box	
Serial size	Mass per metre	Depth of section D	Width of section B	Web t	Flange T	Area of section	3 sides	4 sides	3 sides	4sides
mm	kg	mm	mm	mm	mm	cm^2	m^{-1}	m^{-1}	m^{-1}	m^{-1}
356 x 406	634	474.6	424.0	47.6	77.0	807.548	25	30	15	20
	551	455.6	418.5	42.1	67.5	701.930	30	35	20	25
	467	436.6	412.2	35.8	58.0	594.909	35	40	20	30
	393	419.0	407.0	30.6	49.2	500.574	40	50	25	35
	340	406.4	403.0	26.6	42.9	433.036	45	55	30	35
	287	393.6	399.0	22.6	36.5	365.708	50	65	30	45
	235	381.0	394.8	18.4	30.2	299.432	65	75	40	50
356 x 368	202	374.6	374.7	16.5	27.0	257.219	70	85	45	60
	177	368.2	372.6	14.4	23.8	225.506	80	95	50	65
	153	362.0	370.5	12.3	20.7	194.803	90	110	55	75
	129	355.6	368.6	10.4	17.5	164.335	110	130	65	90
305 x 305	283	365.3	322.2	26.8	44.1	360.426	45	55	30	40
	240	352.5	318.4	23.0	37.7	305.789	50	60	35	45
	198	339.9	314.5	19.1	31.4	252.414	60	75	40	50
	158	327.1	311.2	15.8	25.0	201.364	75	90	50	65
	137	320.5	309.2	13.8	21.7	174.415	85	105	55	70
	118	314.5	307.4	12.0	18.7	150.202	100	120	60	85
	97	307.9	305.3	9.9	15.4	123.448	120	145	75	100
254 x 254	167	289.1	265.2	19.2	31.7	212.855	60	75	40	50
	132	276.3	261.3	15.3	25.3	168.134	75	90	50	65
	107	266.7	258.8	12.8	20.5	136.381	95	110	60	75
	89	260.3	256.3	10.3	17.3	113.311	110	135	70	90
	73	254.1	254.6	8.6	14.2	93.100	130	160	80	110
203 x 203	127	241.4	213.9	18.1	30.1	162.00	65	80	45	55
	113	235.0	212.1	16.3	26.9	145.00	75	90	45	60
	100	228.6	210.3	14.5	23.7	127.00	80	100	55	70
	86	222.2	209.1	12.7	20.5	109.636	95	115	60	80
	71	215.8	206.4	10.0	17.3	90.427	110	135	70	95
	60	209.6	205.8	9.4	14.2	76.373	130	160	80	110
	52	206.2	204.3	7.9	12.5	66.282	150	180	95	125
	46	203.2	203.6	7.2	11.0	58.731	170	200	105	140
152 x 152	51	170.2	157.4	11.0	15.7	65.20	120	145	75	100
	44	166.0	155.9	9.5	13.6	56.10	135	165	85	115
	37	161.8	154.4	8.0	11.5	47.112	160	195	100	135
	30	157.6	152.9	6.5	9.4	38.263	195	235	120	160
	23	152.4	152.2	5.8	6.8	29.245	250	305	155	210

Table 3: October 2006

UK Beams (UKB)

Dimensions to BS4 Part 1:2005



Designation		Depth of section D	Width of section B	Thickness		Area of section	Section factor A/V(Hp/A)			
							Profile		Box	
Serial size	Mass per metre			Web t	Flange T		3 sides	4 sides	3 sides	4sides
mm	kg	mm	mm	mm	mm	cm^2	m^{-1}	m^{-1}	m^{-1}	m^{-1}
533 x 165	85	534.9	166.5	10.3	16.5	108.00	140	155	115	130
	75	529.1	165.9	9.7	13.6	95.20	160	175	130	145
	66	524.7	165.1	8.9	11.4	83.70	180	200	145	165
457 x 191	161	492.0	199.4	18.0	32.0	206.00	75	85	60	65
	133	480.6	196.7	15.3	26.3	170.00	90	100	70	80
	106	469.2	194.0	12.6	20.6	135.00	110	125	85	100
	98	467.2	192.8	11.4	19.6	125.26	120	135	90	105
	89	463.4	191.9	10.5	17.7	113.76	130	145	100	115
	82	460.0	191.3	9.9	16.0	104.48	140	160	105	125
	74	457.0	190.4	9.0	14.5	94.63	155	175	115	135
	67	453.4	189.9	8.5	12.7	85.51	170	190	130	150
457 x 152	82	465.8	155.3	10.5	18.9	104.53	130	145	105	120
	74	462.0	154.4	9.6	17.0	94.48	145	160	115	130
	67	458.0	153.8	9.0	15.0	85.55	155	175	125	145
	60	454.6	152.9	8.1	13.3	76.23	175	195	140	160
	52	449.8	152.4	7.6	10.9	66.64	200	220	160	180
406 x 178	85	417.2	181.9	10.9	18.2	109.00	125	140	95	110
	74	412.8	179.5	9.5	16.0	94.51	140	160	105	125
	67	409.4	178.8	8.8	14.3	85.54	155	175	115	140
	60	406.4	177.9	7.9	12.8	76.52	170	195	130	155
	54	402.6	177.7	7.7	10.9	68.95	190	215	145	170
406 x 140	53	406.6	143.3	7.9	12.9	67.90	180	200	140	160
	46	403.2	142.2	6.8	11.2	58.64	205	230	160	185
	39	398.0	141.8	6.4	8.6	49.65	240	270	190	215
356 x 171	67	363.4	178.1	9.1	15.7	85.49	140	160	105	125
	57	358.0	172.2	8.1	13.0	72.55	165	190	120	145
	51	355.0	171.5	7.4	11.5	64.91	185	210	135	160
	45	351.4	171.1	7.0	9.7	57.33	205	235	150	180
356 x 127	39	353.4	126.0	6.6	10.7	49.77	210	235	165	195
	33	349.0	125.4	6.0	8.5	42.13	250	280	195	225
305 x 165	54	310.4	166.9	7.9	13.7	68.77	160	185	115	140
	46	306.6	165.7	6.7	11.8	58.75	185	210	135	160
	40	303.4	165.0	6.0	10.2	51.32	210	240	150	185
305 x 127	48	311.0	125.3	9.0	14.0	61.23	160	180	120	145
	42	307.2	124.3	8.0	12.1	53.40	180	200	140	160
	37	304.4	123.4	7.1	10.7	47.18	200	225	155	180
305 x 102	33	312.7	102.4	6.6	10.8	41.83	215	240	175	200
	28	308.7	101.8	6.0	8.8	35.88	250	280	200	230
	25	305.1	101.6	5.8	7.0	31.60	280	315	225	255
254 x 146	43	259.6	147.3	7.2	12.7	54.77	170	195	120	150
	37	256.0	146.4	6.3	10.9	47.16	195	225	140	170
	31	251.4	146.1	6.0	8.6	39.68	230	270	165	200
254 x 102	28	260.4	102.2	6.3	10.0	36.08	220	250	175	200
	25	257.2	101.9	6.0	8.4	32.04	250	280	190	225
	22	254.0	101.6	5.7	6.8	28.02	280	320	220	255

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