# The Fire Performance of Engineered Timber Products and Systems

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Dedicated to L.J. Humphrey, P. Guy and R. Rupasinghe.

# THE FIRE PERFORMANCE OF ENGINEERED TIMBER PRODUCTS AND SYSTEMS

By
Danny James Hopkin

A dissertation thesis submitted in partial fulfilment of the requirements for the award of the degree Doctor of Engineering (EngD), at Loughborough University

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# ABSTRACT

Timber is an inherently sustainable material which is important for future construction in the UK. In recent years many developments have been made in relation to timber technology and construction products. As the industry continues to look to construct more efficient, cost effective and sustainable buildings a number of new engineered timber products have emerged which are principally manufactured off-site. In terms of light timber frame, products such as structural insulated panels (SIPs) and engineered floor joists have emerged. For heavy timber construction, systems such as glulam and cross laminated timber (CLT) are now increasingly common.

Despite many of the obvious benefits of using wood as a construction material a number of concerns still exist relating to behaviour in fire. Current fire design procedures are still reliant upon fire resistance testing and ‘deemed to satisfy’ rules of thumb. Understanding of ‘true’ fire performance and thus rational design for fire resistance requires experience of real fires. Such experience, either gathered from real fire events or large fire tests, is increasingly used to provide the knowledge required to undertake ‘performance based designs’ which consider both fire behaviour and holistic structural response. At present performance based structural fire design is largely limited to steel structures and less frequently concrete buildings. Many of the designs undertaken are in accordance with relevant Eurocodes which give guidance on the structural fire design for different materials. For the same approaches to be adopted for timber buildings a number of barriers need to be overcome.

Engineered timber products, such as SIPs and engineered joists, are innovative technologies. However, their uptake in the UK construction market is increasing year on year. Little is known about how such systems behave in real fires. As a result the development of design rules for fire is a difficult task as failure modes are not well understood. To overcome this

barrier the author has undertaken a number of laboratory and natural fire tests on SIPs and engineered floor joists to establish how such products behave and fail in real fires. The data gathered can be used to develop design approaches for engineered timber products in fire, thus negating the need to rely upon fire resistance testing. The development of design rules from the data gathered would be a progressive step towards performance based design.

Realising performance based fire design for timber structures at present has three obvious barriers. Firstly, thermo-physical properties for timber exposed to natural fires are not well defined. Current guidance in standards such as EN 1995-1-2 provides data for standard fire exposure only. Movement towards design for parametric fires requires a better understanding of timber thermo-physical behaviour under different rates of heating and durations of fire exposure. Secondly, particularly in the UK, the fire performance of timber buildings is heavily influenced by the behaviour of gypsum plasterboard which is commonly used as passive fire protection. The thermal behaviour of gypsum under both standard and natural fire conditions is still not well understood. The majority of research available relating to gypsum in fire is dated, whilst board products continually evolve. Finally, the whole building behaviour aspects utilised in the fire design of steel and other structures have arisen as a result of complex numerical simulations. At present most commercial finite element codes are not appropriate for modelling entire timber buildings exposed to fire due to complexities relating to the constitutive behaviour of timber. Timber degrades differently depending upon stress state (i.e. tension or compression), temperature and importantly temperature history.

In recognition of the above barriers, the author has made a number of developments. Firstly, a modified conductivity model for softwood is proposed which is shown to give acceptable depth of char and temperature predictions in timber members exposed to the heating phase of parametric fires. This model is suitable for adoption in any computational heat transfer model.

Secondly, the finite element software TNO DIANA has been modified, via user supplied subroutines, to simulate large timber buildings exposed to fire by considering stress state, temperature and state history.

The developments made in this engineering doctorate are intended to facilitate the progression of performance based design for timber structures. The numerical approaches adopted herein have been supported using multi-scale experimental approaches. As a result a number of novel tools for implementation in FEA models are proposed which should ultimately lead to a more rational approach to the fire design of timber buildings.

**KEY WORDS**

Timber, Fire, Structural Fire Engineering, SIPs, Joists, Finite Element Analysis, Heat Transfer

# PREFACE

The research presented within this thesis was conducted in partial fulfilment of the requirements for the award of an Engineering Doctorate (EngD) degree at the Centre for Innovative and Collaborative Engineering (CICE), Loughborough University. The EngD is, in essence, a PhD based in industry, designed to produce doctoral graduates that can drive innovation in engineering with the highest level of technical, managerial and business competence. This EngD research project was sponsored by BRE Global, and the Engineering and Physical Sciences Research Council (EPSRC). The EngD is examined on the basis of a thesis containing at least three (but not more than five) research publications and/or technical reports. This thesis contains a discourse which is supported by five technical publications, located in Appendices A to E.

# USED ACRONYMS / ABBREVIATIONS / NOMENCLATURE

**Acronyms and Abbreviations**

| | |
|---|---|
| BRE | Building Research Establishment |
| BSI | British Standards Institution |
| CICE | Centre for Innovative and Collaborative Engineering |
| CLG | Department for Communities and Local Government |
| CLT | Cross Laminated Timber |
| CPB | Cement Particle Board |
| DFP | Department for Finance and Personnel |
| EngD | Engineering Doctorate |
| EPS | Expanded Polystyrene |
| EPSRC | Engineering and Physical Science Research Council |
| FE | Finite Element(s) |
| FEA | Finite Element Analysis |
| FPL | Forest Products Laboratory (USA) |
| GLULAM | Glue Laminated Timber |
| ISO | International Organisation for Standardisation |
| IStructE | Institution of Structural Engineers |
| LVL | Laminated Veneer Lumber |
| MCM | Modified Conductivity Model |
| MOE | Modulus of Elasticity |
| NA | National Annex |
| NHBC | National House Building Council |
| NRC | National Research Council of Canada |
| OSB | Oriented Strand Board |
| PF | Phenol Formaldehyde |
| PFP | Passive Fire Protection |
| PIR | Polyisocyanurate |
| PRF | Phenol Resorcinol Formaldehyde |
| PS | Polystyrene |
| PUR | Polyurethane |
| SBS | Scottish Building Standards |
| SCI | Steel Construction Institute |
| SIP | Structural Insulated Panel |
| SV | State Variable |
| TNO | Netherlands Organisation for Applied Scientific Research |
| Type A | Standard gypsum plasterboard according to BS EN 520:2004 (BSI 2004d) |
| Type F | Fire resistant gypsum plasterboard according to BS EN 520:2004 (BSI 2004d) |
| UKSIPSA | UK SIPs Association |
| UL | Underwriters Laboratory |
| USS | User Supplied Subroutine |

## Nomenclature

Symbols are split between heat transfer, charring and mechanical/structural analysis.
(-) Denotes a dimensionless variable.

Heat transfer and charring

| | |
|---|---|
| $\beta_0$ | One dimensional charring rate under ISO 834 heating (mm/min) |
| $\beta_n$ | Notional charring rate under ISO834 heating (mm/min) |
| $\beta_{par}$ | Notional parametric charring rate (mm/min) |
| $\beta_{par,\ 1D}$ | One dimensional parametric charring rate (mm/min) |
| $\beta_{\omega\rho 0}$ | Density and moisture modified 1D charring rate (mm/min) |
| $d_{ch}$ | Depth of char (mm) |
| $d_0$ | Depth of zero strength layer (mm) |
| $\Gamma$ | Heating rate relative to the standard fire curve (-) |
| $q_{td}$ | Fire load density related to the total area of the enclosure (MJ/m$^2$) |
| O | Opening factor (m$^{0.5}$) |
| b | Compartment thermal inertia (J/m$^2$s$^{0.5}$K) |
| $t_0$ | Time period with a constant charring rate (min) |
| $t_{max}$ | Duration of the heating phase of a given parametric fire (min) |
| $\lambda$ | Conductivity (W/m.K) |
| c | Specific heat of timber (J/kg.K) |
| $\varepsilon$ | Emissivity (dimensionless) |
| $\alpha$ | Convection coefficient (W/m$^2$.K) |
| $k_{\lambda,mod}$ | Heating rate and fire dependent conductivity modification factor (-) |
| $k_{\Gamma,mod}$ | Heating rate dependent conductivity modification factor (-) |
| $k_{qtd,mod}$ | Fire load dependent conductivity modification factor (-) |
| $\rho_0$ | Ambient density of timber at 12% moisture content (kg/m$^3$) |
| G | Density ratio (-) |
| $\omega$ | Moisture content of timber (%) |
| $\omega_f$ | Moisture content mass fraction (-) |

Structural analysis / mechanics of materials

| | |
|---|---|
| $i$ | Time step / increment number (-) |
| $n$ | Element number (-) |
| $m$ | Integration point number (-) |
| $t$ | Time (s) |
| $T$ | Temperature (°C) |
| $E$ | Modulus of elasticity (Young's modulus) (MPa) |
| $\varepsilon_{nn}$ | Strain in the nn direction (-) |
| $\varepsilon_{E}$ | Elastic strain (-) |
| $\varepsilon_{cr}$ | Crack strain (-) |
| $\sigma_{nn}$ | Stress in the nn direction (MPa) |
| $\sigma_{C,\theta,i}$ | Compressive strength at temperature θ and time I (MPa) |
| $\sigma_{T,\theta,i}$ | Tensile strength at temperature θ and time I (MPa) |
| $\tau_{nn}$ | Shear stress in the nn direction (MPa) |
| $G_f$ | Fracture energy (Pa) |
| $\rho$ | Density (kg / $m^3$) |
| $\delta$ | Deflection / displacement (mm) |
| $h$ | Crack bandwidth (-) |
| $K_{EC,mod,\theta,i}$ | Temperature dependent compressive MOE reduction factor at time i (-) |
| $K_{ET,mod,\theta,i}$ | Temperature dependent tensile MOE reduction factor at time i (-) |
| $K\sigma_{C,mod,\theta,i}$ | Temperature dependent compressive strength reduction factor at time i (-) |
| $K\sigma_{T,mod,\theta,i}$ | Temperature dependent tensile strength reduction factor at time I (-) |

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# LIST OF PAPERS

The following papers, included in the appendices, have been produced in partial fulfilment of the award requirements of the Engineering Doctorate during the course of the research.

**PAPER 1 (SEE APPENDIX A)**

LENNON, T., HOPKIN, D.J., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. Large scale natural fire tests on protected engineered timber floor systems. *Fire safety journal*, **45**(3), pp 168–182.

**PAPER 2 (SEE APPENDIX B)**

HOPKIN, D.J., LENNON, T., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. Failure mechanisms of structural insulated panel (SIP) systems under natural fire conditions, V.R. KODUR and J.M. FRANSSEN, eds. In: Proceedings of the sixth international conference on structures in fire, $2^{nd}$-$4^{th}$ June 2010, DEStech, pp 520-527.

**PAPER 3 (SEE APPENDIX C)**

HOPKIN, D.J., EL-RIMAWI, J, SILBERSCHMIDT, V., and LENNON, T., 2011. An effective thermal property framework for softwood in parametric design fires: Comparison of the Eurocode 5 parametric charring approach and advanced calculation models. *Journal of construction and building materials*, **25**(5), pp 2584-2595.

**PAPER 4 (SEE APPENDIX D)**

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2011. A numerical study of gypsum plasterboard behaviour under standard and natural fire conditions. *Fire and materials*. DOI: 10.1002/fam.

**PAPER 5 (SEE APPENDIX E)**

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., Effect of fire-induced damage on the uniaxial strength characteristics of solid timber: A numerical study. *Journal of Physics: Conference Series,* **305**(1), doi: 10.1088/1742-6596/305/1/012039.

**ADDITIONAL PUBLICATIONS**

In addition to the above papers the author has co-authored a number of other publications which are not included for assessment. These are listed below.

HOPKIN, D.J., LENNON, T., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. A laboratory study into the fire resistance performance of structural insulated panels (SIPs), V.R. KODUR and J.M. FRANSSEN, eds. In: Proceedings of the sixth international conference on structures in fire, $2^{nd}$-$4^{th}$ June 2010, DEStech, pp 611-618.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2010. Modelling the fire performance of structural insulated panels: Heat transfer, S. GRAYSON, ed. In: Interflam 2010: Proceedings of the twelfth international conference, $5^{th}$-$7^{th}$ July 2010, Interscience, pp 95-106.

LENNON, T. and HOPKIN, D., 2010. *Fire performance of Structural Insulated Panel systems*. IP21/10. Garston, UK; BRE.

LENNON, T. and HOPKIN, D., 2010. *The performance in fire of Structural Insulated Panels*. BD 2710. London, UK; Department for Communities and Local Government.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2011. Adaptation of FEA codes to simulate the heating and cooling process of timber structures exposed to fire, F. WALD, ed. In: Proceedings of the 2nd International Conference on applications in structural fire engineering, 29th April 2011, Czech Technical University, Prague, pp 307-312.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2011. A numerically derived modified conductivity model for softwood exposed to parametric design fires , F. WALD, ed. In: Proceedings of the 2nd International Conference on applications in structural fire engineering, 29th April 2011, Czech Technical University, Prague, pp 331-336.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2011. The impact of assumed fracture energy on the fire performance of timber beams, F. WALD, ed. In: Proceedings of the 2nd International Conference on applications in structural fire engineering, 29th April 2011, Czech Technical University, Prague, pp 349-354.

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2011. Fire performance of structural insulated panel and engineered floor joists assemblies exposed to natural fires. *Fire safety journal*. doi:10.1016/j.firesaf.2011.07.009.

# 1 INTRODUCTION

This Engineering Doctorate (EngD) research project discusses the fire performance of engineered timber products and systems. It was sponsored by BRE Global which is part of the Building Research Establishment (BRE) group of companies. The BRE Group is a charitable organisation specialising in construction research and education and are owned by the BRE Trust. The project was part funded by the Engineering and Physical Science Research Council (EPSRC) through the Centre for Innovative Construction Engineering (CICE), Loughborough University.

In recent years it has become apparent that the building fabric, in particular the load bearing structural elements, has changed significantly relative to what would be termed ‘traditional’ construction. This change is largely driven through sustainability policies and demands. Due to a shortage of skilled labour, time and budget constraints and environmental drivers, buildings are increasingly constructed from engineered timber products formed off-site and assembled on-site. The introduction of such products and systems has been rapid whilst research associated with their fire performance has lagged behind. In recognition of this, BRE Global commissioned this EngD research project, which ran in parallel with an experimental programme commissioned by CLG, to address the increasing knowledge gap associated with modern building fire performance. As a result, and in line with the organisational ethos, this project advances knowledge in the area of building fire safety which will ultimately impact on the safety of the public and the fire and rescue services.

## 1.1 BUILDING FIRES

Fire is a destructive force that each year in the UK alone accounts for hundreds of deaths and billions of pounds worth of losses through damage to property and business disruption. Concerns for fire safety and measures to protect people and property from fire have been present in the United Kingdom for centuries. Early fire codes were promoted by insurance companies for which the prime focus was property protection (Bryant 2006). Today the focus of the regulatory framework is life safety (CLG 2007), with the issue of building damage the responsibility of insurers or the building owner.

### 1.1.1 LEGISLATIVE FRAMEWORK

In England and Wales guidance on the fire performance of buildings is found in the Approved Documents to the Building Regulations (CLG 2007). Regional differences are covered in separate documents for construction in Scotland (SBS 2010) and Northern Ireland (DFP 2005). In the regulations underpinning such guidance all buildings must meet certain functional requirements covering means of escape, internal fire spread, external fire spread and provisions for the fire service. It is important to note that the Building Regulations are only intended to ensure reasonable standards of health and safety for persons in or around the building. They are not designed to limit structural damage other than where it may pose a risk to human life, nor do they exist to minimise financial losses arising from a fire. This has important implications for the fire engineering design of buildings where the requirements of the Building Regulations may not be sufficient to meet the needs of the client.

In terms of structural performance, life safety is the minimum legislative requirement that should be addressed. In consideration of this, in the event of a fire, a building should provide for (IStructeE 2007):

- Safe egress of the occupants from the building, or reasonable safe movement of occupants to designated refuge areas within the building
- Safe operating conditions for fire fighters
- Safety of people within, or in the proximity of, the building (including fire fighters) from the threat of possible collapse of the building.

In terms of the structural fire safety functional objectives set out in the Building Regulations for England and Wales (CLG 2007), the following are required:

- The building shall be designed and constructed so that in the event of a fire its stability will be maintained for a reasonable period

- The building shall be sub-divided into fire resisting compartments of size appropriate to the scale and intended use of the building to inhibit fire spread

The interpretation and the specification of provisions to meet these functional requirements fall into the domain of designers, more specifically, architects, fire engineers and structural (fire) engineers.

### 1.1.2 THE CONCEPT OF PRESCRIPTIVE FIRE DESIGN AND 'FIRE RESISTANCE'

Until recently, building fire engineering activities were largely based on the application of prescriptive codes of practice. Accordingly, the design engineer was expected to meet pre-determined requirements based on generic occupancies or classes of fire risk (Khoury 2000). For example, in terms of egress this may take the form of prescribed minimum doorway widths, distances to the nearest point of escape, etc. In building performance terms, and in terms of structural performance, this refers to the requirement to meet minimum levels of 'fire resistance' specified as units of time for a given risk class or occupancy type. This concept extends beyond the primary structure to cover building components such as fire doors, glazing, etc, all of which have roles in fulfilling the functional requirements of the regulations.

The ability of a building component to satisfy its required fire resistance is assessed using a standardised fire resistance test procedure (BSI 1987). During the test, an element almost exclusively in isolation, is exposed to a pre-defined consistent thermal exposure condition often referred to as the "standard fire curve" (ISO 1999). The fire resistance 'rating' of a building component, measured in time, is determined as the duration for which the component is able to perform its fire resistance function under these very specific test conditions before failure occurs. For non-load bearing elements of a building, the ‘performance requirements’ are different to that of a structural element. From a more general perspective, fire resistance can be defined as:

“*The measure of time that an element, whether it is a structural element, fire door or non-structural compartment wall, will survive in a standard test*” (Bailey 2004).

In a generic form, this definition refers to the ability of a building component to satisfy the load bearing (R), integrity (E) and insulation (I) requirements appropriate to the element’s end use (BSI 1987). Depending upon the nature of the component and its required function, either all or some of these criteria should be satisfied. With specific reference to a part of a structure, for example a load bearing slab, all of these criteria should be satisfied.

From such fire resistance tests prescriptive detailing rules can be determined for elements of structure to achieve required levels of fire resistance.

**Prescriptive rules for structural fire resistance**

A common misconception, particularly related to steel and timber structures, is that elements require fire protection to satisfy the requirements of the Building Regulations. This is not the case. They need ‘fire resistance’ (Bailey 2004). Fire resistance, based on years of experimental experience, can be achieved on a prescriptive basis using simplified design rules for a given structural material. Such rules, if followed, would allow for a structural element or a building product to survive the required amount of time, in a fire resistance test, under the very specific heating regime and boundary conditions inherent in the process. For example, with reference to concrete structures, a beam may survive for 60 minutes in a fire resistance test if minimum cover to reinforcement and beam dimensions are implemented, as specified in British Standard BS 8110 (BSI 1997). Similarly, for solid timber joist floors, 60 minutes fire resistance can be achieved if 25mm of gypsum plasterboard is used to underline the ceiling (Cherry 2010). Such rules are considered ‘prescriptive’ as they give a single pre-determined means of achieving a given fire resistance requirement.

**Fire resistance versus real fire performance**

Fire resistance and the use of the standard fire test are widely understood by designers and regulators. However, there is a continued misconception that a fire resistance rating, for example 60 minutes, refers to the ability of a building to survive for the same period in a real fire. This correlation cannot be made for a number of simple reasons:

- Firstly, the stochastic nature of fire, due to variability in fire loadings and ventilation from building to building, means that a standardised fire curve cannot represent the full, or even majority, of the types of fires that would occur in practice.

- Secondly, the standard fire test is a measure of the fire resistance of a single component acting in isolation. However, in a real structure, complex interactions occur which affect the behaviour of a component and may enhance (or degrade) its expected fire resistance. An example of this is the interaction between floors, walls, and the elements connecting them (bolts, rivets, nailing plates, etc) which are not considered in a standard fire test.

- Thirdly, real fires do not continuously increase in temperature; they will eventually cool down. However, the thermal response used in fire resistance tests is that of an ever increasing temperature without consideration of cooling. This implies that the fire has an infinite fuel source, which is of course not the case.

It follows that the prescribed detailing rules derived on the basis of such tests do not accurately consider the shortcomings of the fire resistance test procedure and the potential implications they may have in real fire events. Although the prescriptive philosophy has proved adequate over the years, the prescriptive nature of such an approach also provides a barrier to innovation and increases costs (Lennon 2004). In addition, as far as safety is concerned, the robustness of a structure in a real fire remains uncertain, as a result of adopting prescriptive based detailing and design (Bailey 2004).

### 1.1.3 BACKGROUND TO 'PERFORMANCE BASED DESIGN' AND FIRE ENGINEERING

In recognition of the limitations of the prescriptive based approach to achieving fire resistance, the concept of performance based design has evolved. This has resulted in a marked increase in the number of projects utilising the concepts of fire engineering and structural fire engineering (Bryant 2006).

The concept of performance based design is simple. A building is designed based upon fires that are unique to the characteristics (occupancy, location, height etc) of the given project. It is seen above that the standard fire curve is 'non-physical' as it continually heats for all time. In reality, fires have a finite life governed by a combination of compartment thermal inertia, ventilation and fire loading conditions all of which are unique to a given building. Intuitively, all of the products associated with fire (smoke, radiation, and toxic gases) are then also unique to a given scenario. From a fire safety standpoint it follows that, if a characteristic design fire can be defined for a given set of circumstances, then the tenability time for a given fire can be determined and a fire strategy derived. Similarly, from a structural fire engineering perspective, a structure can be designed to survive a given fire without collapse or significant irreversible damage, during both the heating and cooling phases, thus fulfilling its functional requirement.

### 1.1.4 STRUCTURAL FIRE ENGINEERING

The concept of structural fire engineering has been alluded to in the previous section. Bailey (2009) defines the discipline as follows:

"*Structural Fire Engineering deals with specific aspects of passive fire protection in terms of analysing the thermal effects of fires on buildings and designing members for adequate load bearing resistance and to control the spread of fire*."

It follows that the process of structural fire engineering is complex as it requires levels of understanding in relation to fire dynamics, heat transfer and non-linear structural-mechanical behaviour. More frequently, the 'fire dynamics' aspect of the discipline falls within the domain of the fire engineer who, using advanced tools like zone and CFD models, is able to characterise a design fire more accurately. However, where this is not possible, simplified rules for determining fire behaviour are available (BSI 2002). These are based on extensive research (Twilt and Van Oerle 1999) and are often adopted by structural fire engineers. In summary, the process of structural fire engineering can be visualised as shown in Figure 1.1.

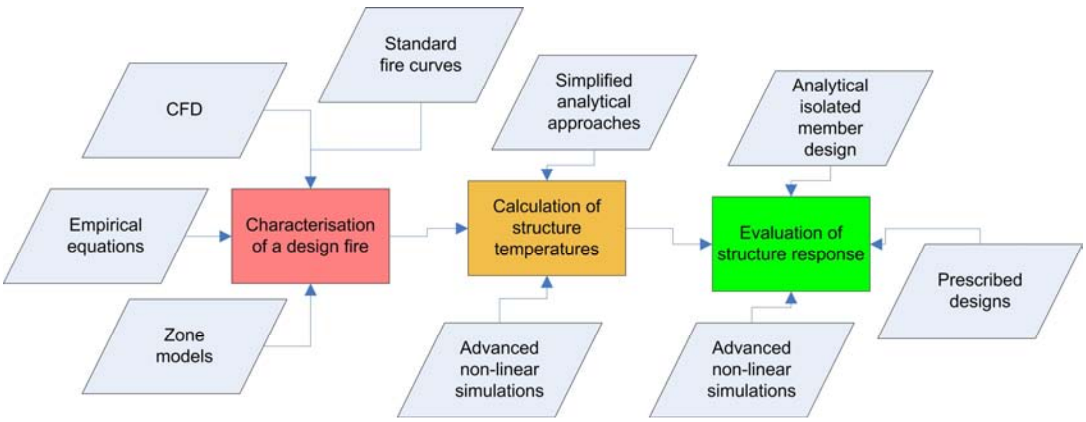



**Figure 1.1 The structural fire engineering process**

Much of the evolution of the structural fire engineering discipline has been made possible through advanced numerical simulations, for both heat transfer and mechanical behaviours.

This has allowed for the holistic behaviour of structures to be considered in design and research. Many commercial finite element computer packages such as DIANA (Manie 2010), ABAQUS (Simulia 2008) and ANSYS (ANSYS Inc 2006) have been made sufficiently flexible to be applied to structural fire engineering problems. However, more specific numerical fire codes, such as SAFIR (Franssen 2005) and VULCAN (SUEL 2010), have also evolved and are widely used.

The scope of applicability of the structural fire engineering concept is limited by the amount of research performed in any one given area. The concrete and steel industries have benefited, through efficiency and cost savings, as a result of structural fire engineering which has been promoted in these areas due to a large amount of fundamental research. The Cardington tests undertaken by BRE in the 1990's allowed for the development of numerical models which are now available for the design of concrete and steel structures (Bailey 2002 and Newman, *et al.* 2006). Similarly, the TF2000 project, also undertaken by BRE (Lennon, *et al.* 2000), advanced knowledge in the area of timber in fire performance. However, the focus and the instrumentation adopted in the latter research programme was not intended to be used in the validation of the numerical simulations of timber structures. As a result, the 'performance based design' of timber structures is less frequently undertaken relative to more researched materials.

**Structural Design and the Eurocodes**

As the concept of structural fire engineering and performance based design evolved, new codes and guidance needed to be developed to ensure a harmonised approach to both structural and structural fire design in Europe. At the turn of the century structural Eurocodes, which presented a framework for the structural and structural fire design of buildings, were published. In the overall framework, each major material has its own Eurocode complete with recommendations for fire design. Concrete, Timber, Steel, Composite, Masonry and

Aluminium are all represented in the Eurocodes at ambient temperature (BSI 2004a/b, 2005b/d/f and 2007a) and at elevated temperatures (BSI 2004c, 2005a/c/e/g and 2007b). The codes, when utilised with the fire loading element of the series (BSI 2002), give a framework for undertaking structural fire design for all of the aspects noted in Figure 1.1. However, the range of applicability of each code is limited by the extent of its underpinning fundamental research. Therefore, the concrete and steel codes are much more complete and offer more flexibility in design. In comparison, the timber in fire code is less advanced and its scope for application, particularly in the UK, is very limited. However, through applied research, which is inclusive of emerging technologies, the applicability of the code can be improved as the fundamental behaviour of timber in fire will be better understood. This would lead to not only more efficient timber buildings (thus increasing their market share) but also to more fire safe timber buildings which benefits the wider public.

## 1.2 INTRODUCTION TO TIMBER STRUCTURES AND SYSTEMS

Timber structures broadly fall into two categories; heavy timber construction and light timber frame (Buchanan 2001). Within both types of construction, there is often another sub-division which distinguishes between structures that are formed from solid section timber and those made of ‘engineered’ timber. Heavy timber construction typically refers to structures formed from large section (generally in excess of 300mm) sawn wood or glulam beams, columns, trusses and slabs. Due to the large cross section of such members, passive fire protection is generally not applied and any fire resistance is ‘inherent’. Light timber frame structures differ as they are formed from smaller timber elements in the shape of stud walls and joist floors. Such small sections would be engulfed in a fire in very little time. As a result, they are typically protected with timber product boards, fire retardant treatments, gypsum plasterboards or a combination of these methods.

Timber is a non-homogenous material as it is organic. The formation of knots and the direction of the grain are among a number of important factors that heavily influence the strength of a timber structural element. Members formed from large continuous pieces of timber contain many defects. As a result, the timber strengths quoted in the grading process can be extremely conservative as they are based on observed defects rather than on the true strength of the wood. In recognition of this, many engineered wood products are now available which reduce waste through more efficient use of material, whilst allowing for the 'true' strength of timber to be utilised. As a result, in terms of light timber frame, products like engineered floor joists incorporating systems such as timber I-Joists, steel truss web joists and timber truss girders are beginning to replace traditional solid section studs and solid flexural elements. Similarly, off-site components like Structural Insulated Panels (SIPs) are used to form building envelopes and vertical structural members such as walls. In the case of large section 'heavy' timber construction, glue laminated (Glulam) or laminated veneer lumber (LVL) are often adopted in favour of solid members. Similarly, cross laminated timber (CLT) is often used as a direct substitute for solid section timber slabs and panels. Each of these technologies is outlined below for completeness.

### 1.2.1 ENGINEERED FLOOR JOISTS (EFJS)

In the UK, engineered floor joists are frequently used as a direct substitute for traditional 45 x 220 mm solid timber joists. This is largely because they are more lightweight and structurally efficient, requiring less material to achieve the same or similar stiffness to that of a solid timber joist. The three most common types of engineered floor joists are 'timber I-Joists', 'steel truss web joists' and 'timber truss girders' (see Figure 1.2).

Timber I-Joists refer to narrow I-sections formed from two solid or LVL timber 'flanges' joined via glue to a slender oriented strand board (OSB) or a similar board product web. The adopted glues are typically phenol-formaldehyde (PF) or phenol-resorcinol-formaldehyde (PRF) based adhesives (Richardson 2004). Typically, the flanges are made of 45 mm square sections, however, variations exist depending upon the loads and spans required. The timber board web will typically measure 8-10 mm in thickness but variations also exist depending upon the requirements of a given project.

In principle, steel truss web joists are formed in a similar manner to timber I-Joists. However, the slim timber board web is replaced by discrete pressed steel truss sections which are mechanically fixed to the top and bottom flanges via nailing plates. The top and bottom flanges in this instance may be a little wider than that of a timber I-joist and would typically measure 45x100mm. Again, variations exist depending upon specific requirements.

Timber truss girders are similar in shape to traditional steel truss girders. They are formed from two timber or LVL flanges separated by a network of timber bracing. The various components are joined via mechanical methods such as nailing plates.

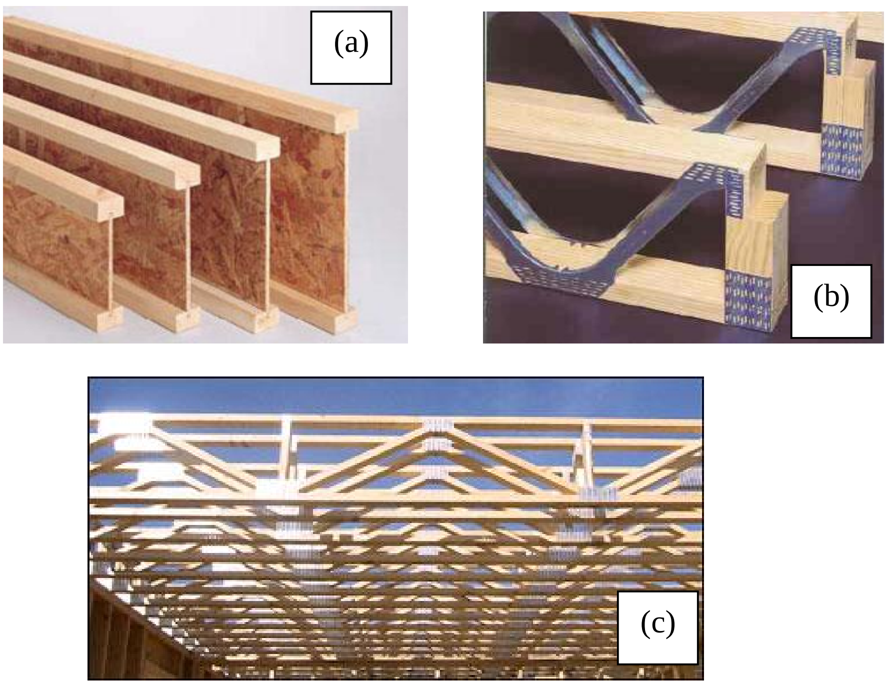



**Figure 1.2 Engineered floor joists (a) I-Joists (b) Truss steel web (c) Truss girders**

All engineered floor joists are fixed to supporting walls, or similar supports, in an identical manner to that of traditional joists. Typically, this would be done using either 'joist hangers' or through a cassette whereby joists are fixed mechanically to a perimeter ring beam.

### 1.2.2 STRUCTURAL INSULATED PANELS (SIPS)

SIPs are prefabricated lightweight building units used as principal load-bearing components (Bregulla and Enjily 2004). Essentially, they are a sandwich construction comprising two layers of high-density sheet material bonded to a low-density cellular core (Figure 1.3). The most commonly used insulated cores in the UK are expanded polystyrene (EPS) and polyurethane (PUR). The facing boards are most commonly oriented strand board (OSB). However, cement particle board (CPB), or similar boards, may less frequently be adopted as a facing material. SIPs constructed with EPS cores are formed by cold-pressing layers of OSB and EPS with a polyurethane based adhesive. In the case of a PUR core, the insulation material is poured between OSB sheets and allowed to cure forming an autohesive bond between panel layers.

**Figure 1.3 Section from a structural insulated panel with a PUR core**

In Europe, SIPs are normally used as load-bearing internal and external walls and less frequently as roofs. In addition, SIPs are occasionally used to form floors. In the UK, SIPs typically form all principal load-bearing walls in conjunction with an engineered timber floor joist system. However, there have been instances where SIP walls have been used to support precast concrete floor planks and as alternatives to timber roof trusses.

### 1.2.3 LAMINATED TIMBER

Glulam and LVL are interchangeable terms used to describe structural components formed from thin laminations of kiln dried sawn timber which are compressed together and fixed using an external type adhesive (Figure 1.4). Typically, in traditional laminated products the grain orientation of each veneer is aligned (Thelandersson and Larsen 2003 and Aghayere and Vigil 2007). The laminated cross-section forms a stronger structural unit when compared to solid section timber as defects such as knots and other natural weaknesses are eliminated from the cross-section. Laminated products may also be used for much larger spans and loads when compared to solid section timber. This is because cross-sections of any size (or shape within reasonable limits) can be manufactured from multiple laminations.


**Figure 1.4 (a) Glue laminated timber (b) Laminated veneer lumber**

When the grain orientation of the laminations (commonly termed 'lay up') varies with depth CLT is formed (Figure 1.5). Layers of veneers are gradually built up with perpendicular grain orientations to form a product which has better orthogonal properties than glulam or LVL. As a result, the system is commonly used to form slabs which may span in multiple directions. In addition, due to the cross lamination of the timber plies, CLT exhibits much better dimensional stability than glulam or LVL sections as thermal expansion is more even in all directions (Kolb 2008).

**Figure 1.5 Cross laminated timber**

## 1.3 INTRODUCTION TO THE FUNDAMENTALS OF TIMBER BEHAVIOUR IN FIRE

Large timber sections are generally regarded as having good fire resistance, despite the inherent combustibility of wood. Upon heating timber undergoes a number of ‘phase changes’. At ambient temperature, the moisture content of typical softwoods is approximately 10-14% of its weight. With gradual heating, this percentage changes dramatically as the timber temperature approaches 90-120°C. Most of the moisture evaporates from the heated surface, whilst some flows towards the non-heated zones before re-condensing. With increasing heat energy the timber locally dehydrates and begins to degrade. At temperatures approaching 300°C timber starts to combust. Volatiles generated below the surface of the unaffected wood begin to flow both along and perpendicular to the grain orientation, before igniting at the hot surface (Drysdale 1998).

At approximately 300°C wood begins to rapidly degrade, forming a carbonaceous char layer. This layer is full of cracks and fissures which allow heat to permeate deeper into the timber section. However, in terms of fire resistance, the formation of the char layer is a positive phenomenon as it serves to insulate the unaffected wood below. As temperatures continue to rise, the char layer becomes increasingly friable and ablates. The burning process is shown graphically in Figure 1.6.

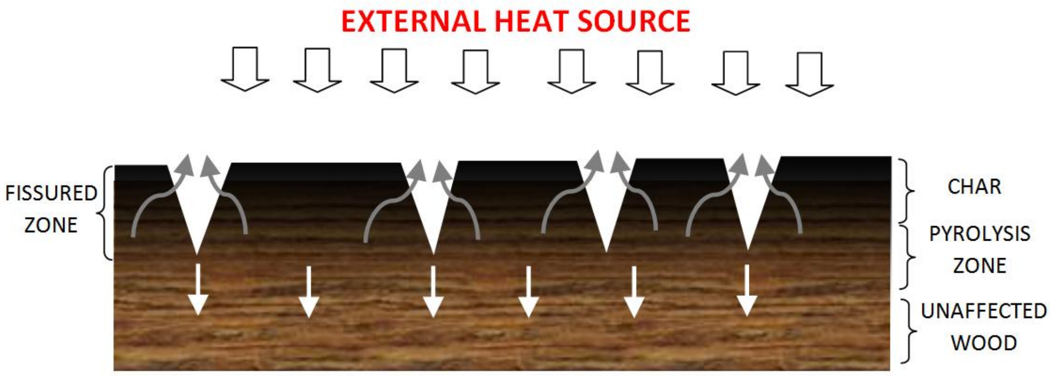



**Figure 1.6 Burning of a timber section in one dimension. Solid arrows indicate flow of volatiles. Adapted from Drysdale (1998)**

Below the increasingly hotter char layer, the interface between the pyrolysis zone and the unaffected wood gradually creeps through the cross section of a timber member. This transient movement of the interface between burnt and un-burnt wood is referred to as the charring rate which is often expressed as a depth per unit time. The rate and extent at which char forms is dependant upon a number of parameters including species, heat flux, density and moisture content.

### 1.3.1 CHARRING

The charring rate of different types of wood has been the subject of much research. Studies as early as 1971 give correlations between heat flux exposure and the rate of char formation (Butler 1971). Various accepted charring rates are available in codes and standards, such as EN 1995-1-2 (BSI 2004c) and NZS303:1993 (SNZ 1993), which are widely adopted. The value typically quoted for softwood under standard fire exposure conditions is 0.65 mm.min-1 (SNZ 1993 & BSI 2004c).

The rate of char formation is understood to be influenced by parameters such as heating rate. As a result, charring rates determined from real fires are often significantly different to those derived based upon standard fire exposure.

The formation of char is one obvious process timber undergoes upon heating. However, a number of reactions/processes that occur prior to the formation of char influence both the heat transfer and mechanical behaviour of timber at high temperature. These aspects are discussed in greater detail below.

### 1.3.2 THERMO-PHYSICAL CHARACTERISTICS OF SOLID TIMBER

Upon heating, timber undergoes two clearly defined physical changes. First, free moisture is driven off as timber temperature approaches 100°C. Second, timber combusts and forms char at temperatures of around 300°C. These physical changes are clearly identifiable in the thermo-physical characteristics of softwood at high temperature.

The thermo-physical properties of timber with increasing temperature have been the topic of much research. A few better known examples are shown in the figures that follow. Relationships between temperature and apparent conductivity, specific heat and density ratio are shown in Figures 1.7 to 1.9 respectively. It is important to note that variations exist between authors due to both the nature and method of derivation of the properties presented. It is understood that these properties represent the heat transfer characteristics normal to the grain orientation. The thermal conductivity along the grain orientation is approximately twice that normal to the grain direction.

The earlier studies of Knudson, *et al.* (1975), Gammon (1987) and Fuller, *et al.* (1992) give thermal properties for softwood which are experimentally determined. In these instances the thermal properties are derived from the direct measurement of conductivity and specific heat using specialist instrumentation. In such cases the thermal characteristics of dry wood are evaluated. As a result, behaviour such as mass transfer is often not implicitly included and hence must be considered explicitly in any subsequent thermal analyses.

Alternatively, properties such as those given by Hadvig (1981), Janssens (1994), Thomas (1997) and König and Walleij (2000) are derived numerically. In these cases the thermal properties are determined on the basis of numerical calibrations against experimental results. In such cases the thermal properties are adjusted to accurately simulate the temperatures observed at various locations through a section of timber. In undertaking such a calibration, the properties implicitly include a number of complex mechanisms for heat transfer. This is so because the consequences of behaviour, such as moisture flow, moisture evaporation and ablation are included within basic thermal characteristics, such as conductivity. This approach has become common as it allows for heat transfer to be simulated through simpler finite element thermal models.

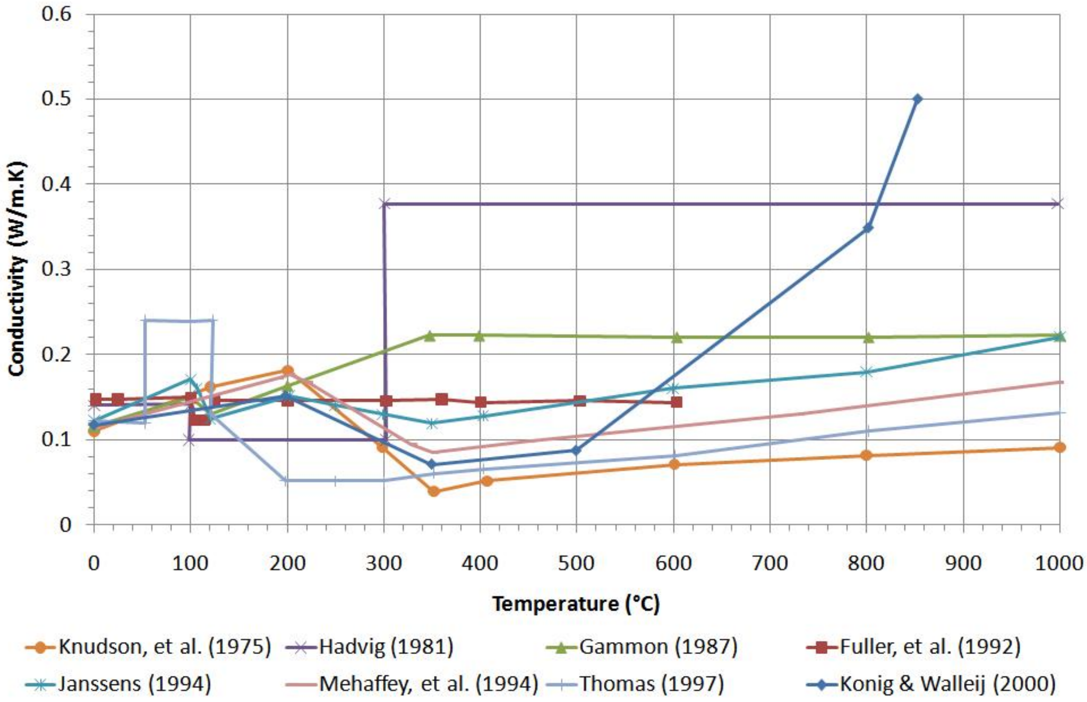



**Figure 1.7 Transverse to the grain conductivity of timber versus temperature from various sources**

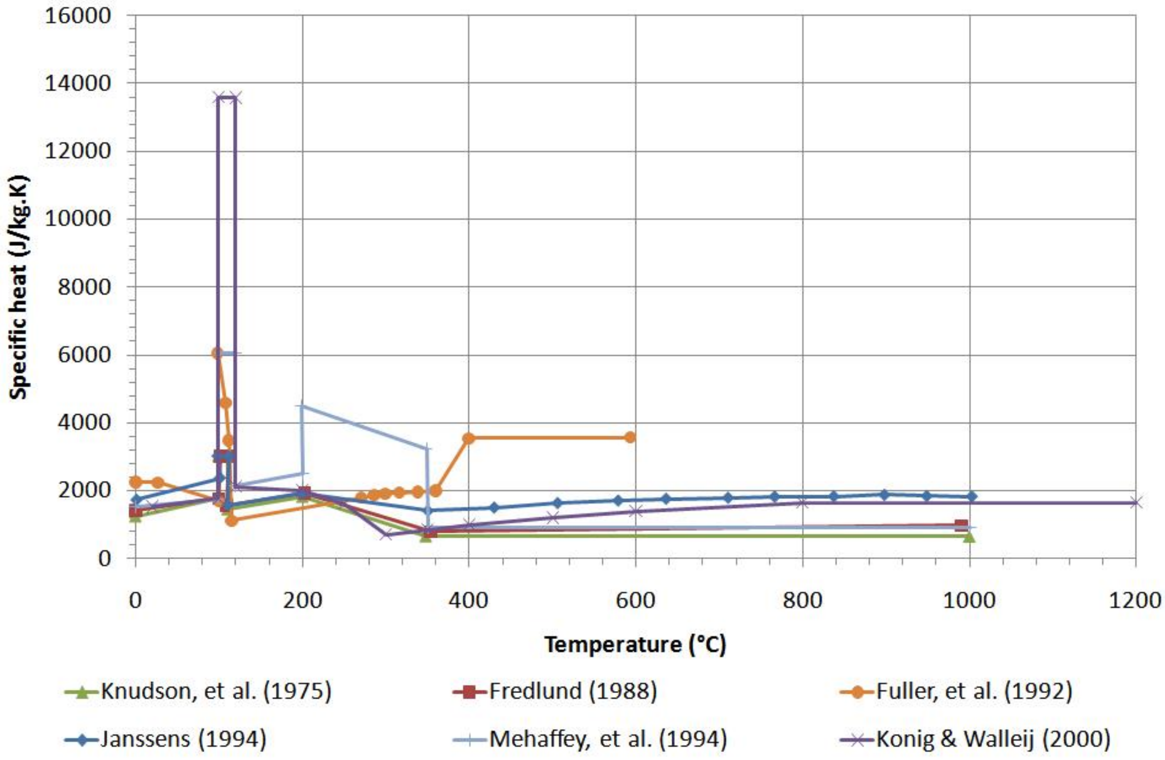



**Figure 1.8 Transverse to the grain specific heat of timber versus temperature from various sources**

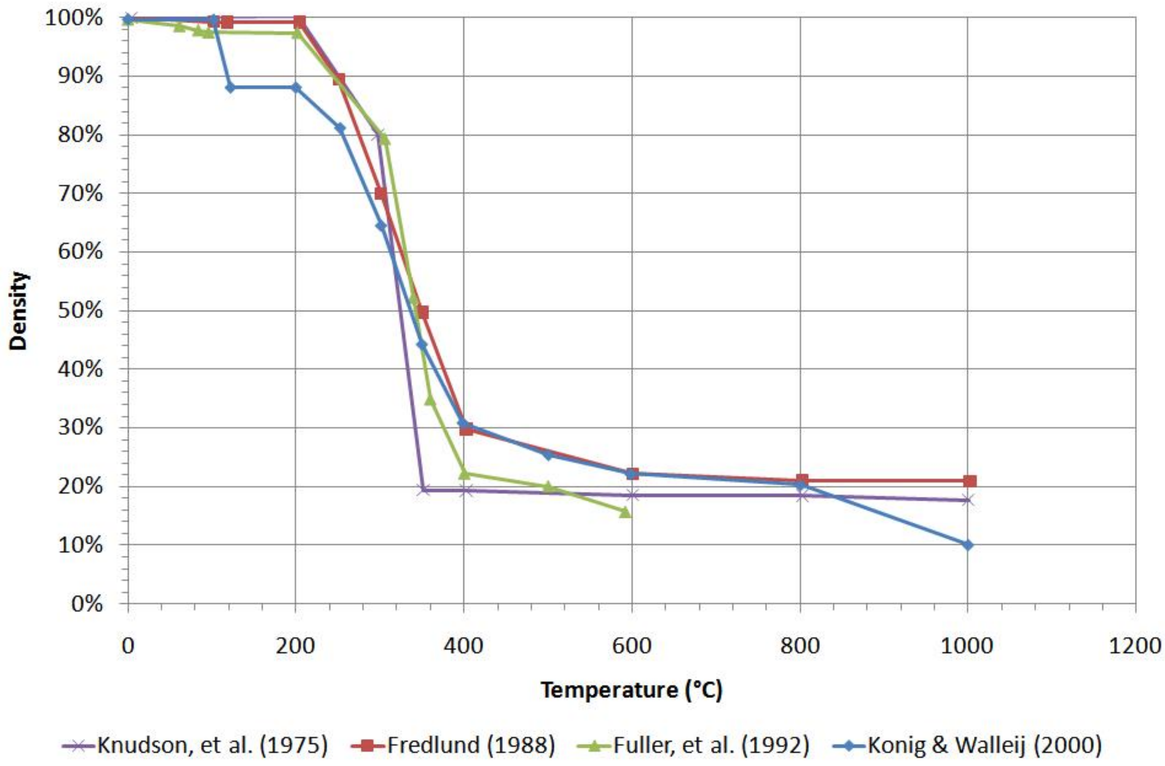



**Figure 1.9 Density of timber versus temperature from various sources**

### 1.3.3 MECHANICAL ASPECTS OF TIMBER BEHAVIOUR AT HIGH TEMPERATURE

Timber is a complex structural material that is not only orthotropic but also has strength characteristics that depend upon species, density, moisture content, stress state and loading duration (Harte 2009). Timber exhibits its best mechanical characteristics when loaded parallel to the grain orientation and is generally weaker when loaded perpendicular to the grain orientation.

At ambient temperature, under compressive loading, timber can be considered as a ductile material which deforms plastically. Under tensile loading timber is brittle and fractures. Typical constitutive behaviour at ambient temperature can be visualised in Figure 1.10.

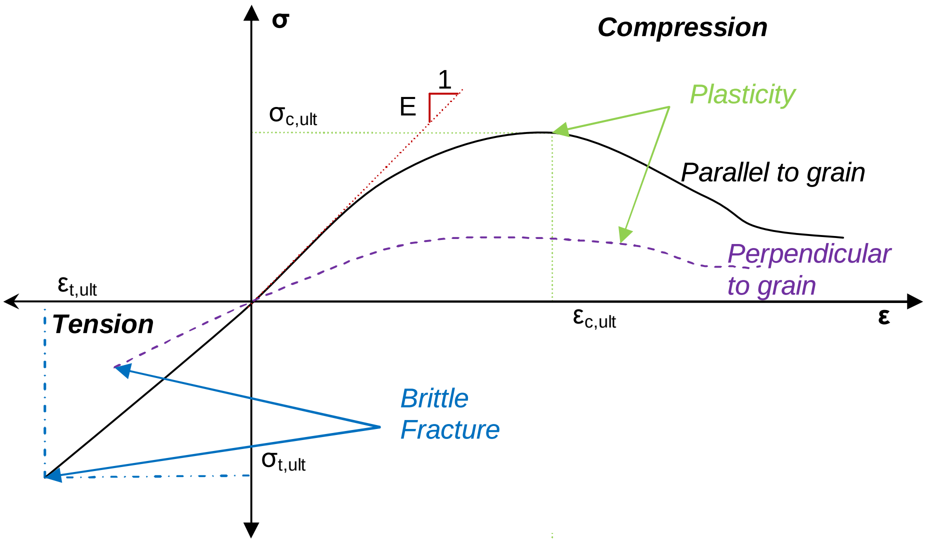



**Figure 1.10 Typical stress strain behaviour of timber (adapted from Buchannan 2001)**

The impact of temperature on the strength and stiffness of timber is widely reported in the literature. In general, the strength and stiffness of wood is highly influenced by the physical changes that occur as a result of de-hydration and combustion. A number of strength, stiffness and temperature relationships reported by different researchers are shown below. Figure 1.11 indicates how tensile strength is affected with increasing temperature. The compressive strength reduction with temperature is shown in Figure 1.12. The impact of temperature on the Modulus of Elasticity (MOE) of wood under compressive and tensile stresses is shown in Figure 1.13. In all instances the relationships apply to strength and stiffness characteristics when loaded parallel to the grain orientation. For structural members resisting loads mostly by flexure it is sufficient to assume an isotropic material model with properties corresponding to that of the parallel to the grain orientation. This is because measures are taken when designing timber structures to ensure they are loaded such that the principle stress is aligned with the grain orientation.

In a similar manner to the thermo-physical aspects of behaviour, obvious trends are apparent where physical changes in the timber occur. In particular it is clear that char has little or no strength and stiffness.

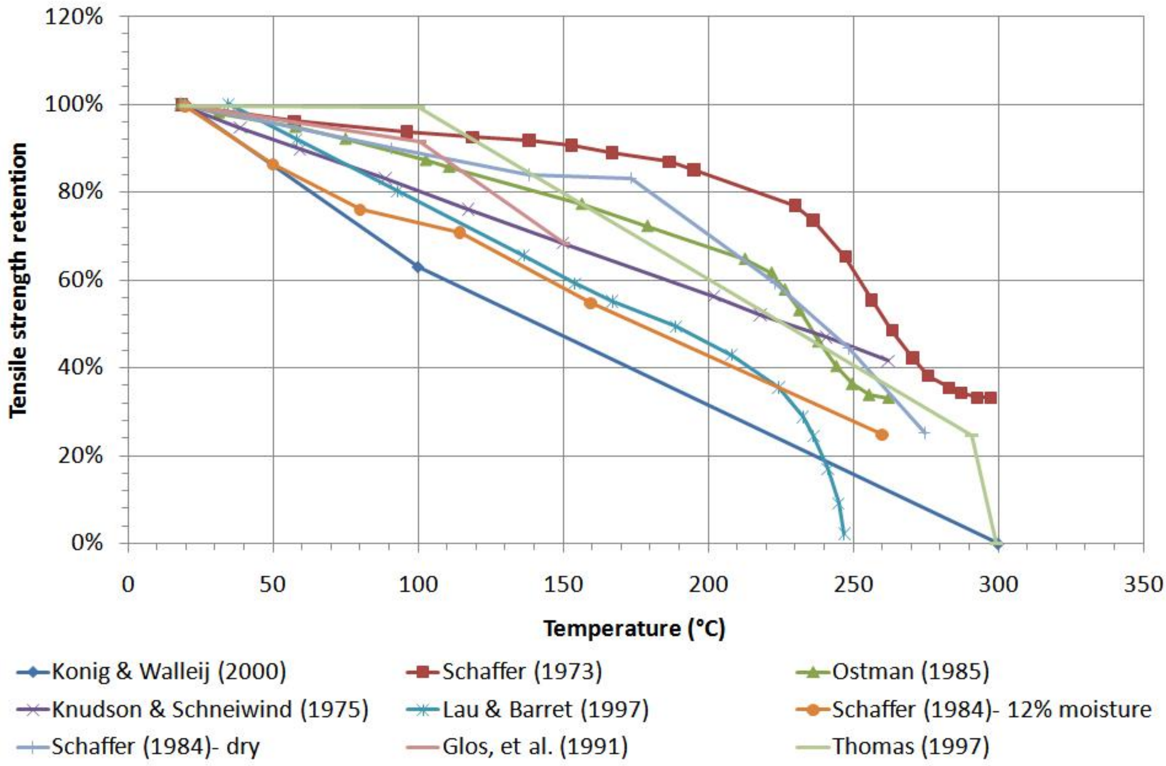



**Figure 1.11 Tensile strength reduction in softwood exposed to elevated temperature (Parallel to the grain)**

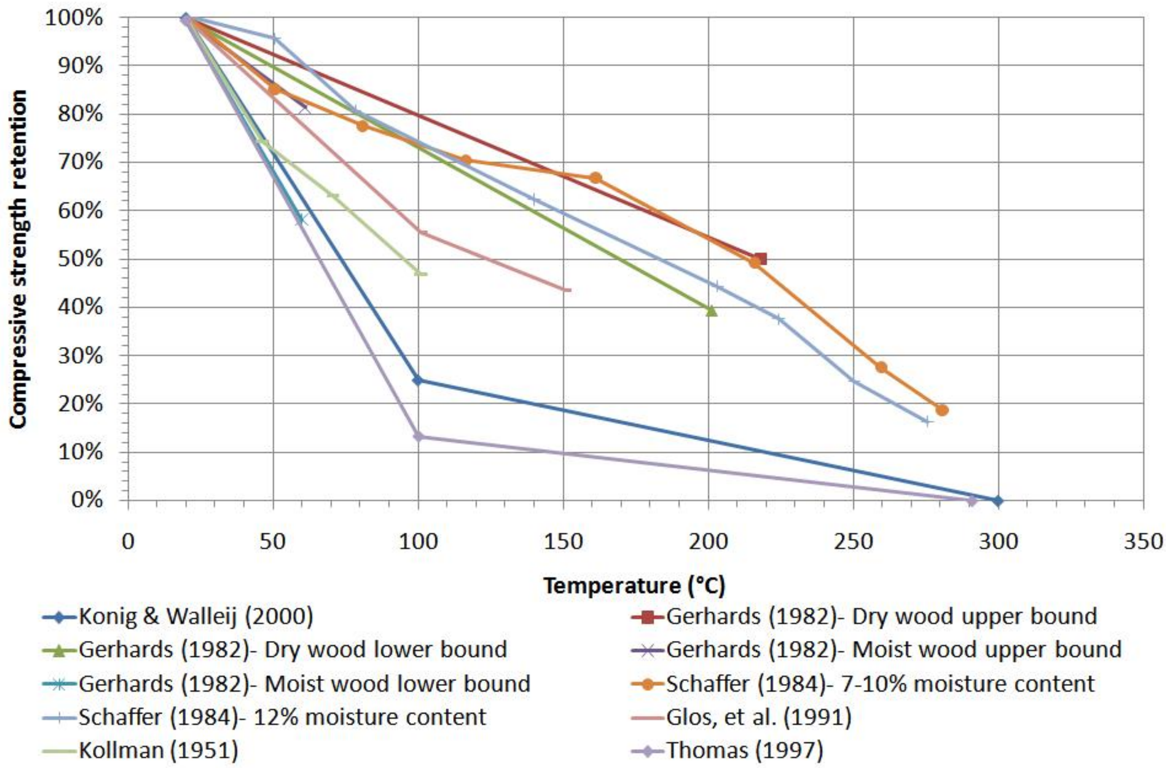



**Figure 1.12 Compressive strength reduction in softwood exposed to elevated temperature (Parallel to the grain)**

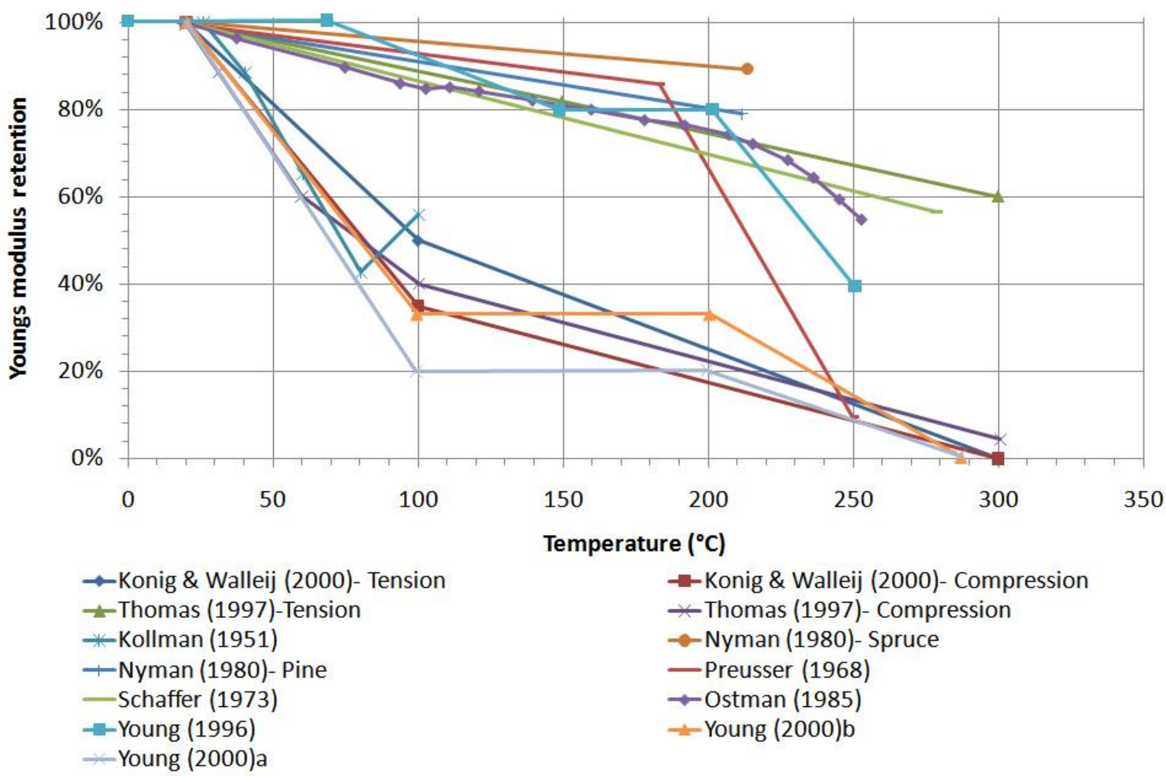



**Figure 1.13 Elastic modulus reduction in softwood exposed to elevated temperature (Parallel to the grain)**

### 1.3.4 PROPERTIES ADOPTED IN DESIGN

EN 1995-1-2 gives guidance on both the thermal and mechanical properties that should be adopted for the fire design of timber structures. These are largely based upon the work of König and Walleij (2000), which in turn are based on the works of other authors including Janssens (1994) and Thomas (1997). In the determination of properties for design purposes, König and Walleij (2000) derived both thermal and mechanical properties from numerical simulations, calibrated against well defined standard fire experiments. As a result, the properties in EN 1995-1-2 are limited to the design of timber structures exposed to the standard fire curve.

Inspection of Figures 1.11 to 1.13 indicates some difference between the mechanical property proposals of König and Walleij (2000) (as published in EN 1995-1-2) and those conducted by other researchers using Quasi-static steady state experiments. This is most likely to be due to the calibration process undertaken by König and Walleij (2000), whereby numerical simulations were calibrated against experiments conducted on flexural timber members.

In the design of timber structures it is common to assume an ideally elasto-plastic constitutive regime under compressive stresses and ideally brittle behaviour under tensile conditions. Adopting these assumptions and utilising the strength and stiffness reduction factors proposed in EN 1995-1-2 by König and Walleij (2000), the constitutive relations for timber proposed in Figure 1.14 can be derived.

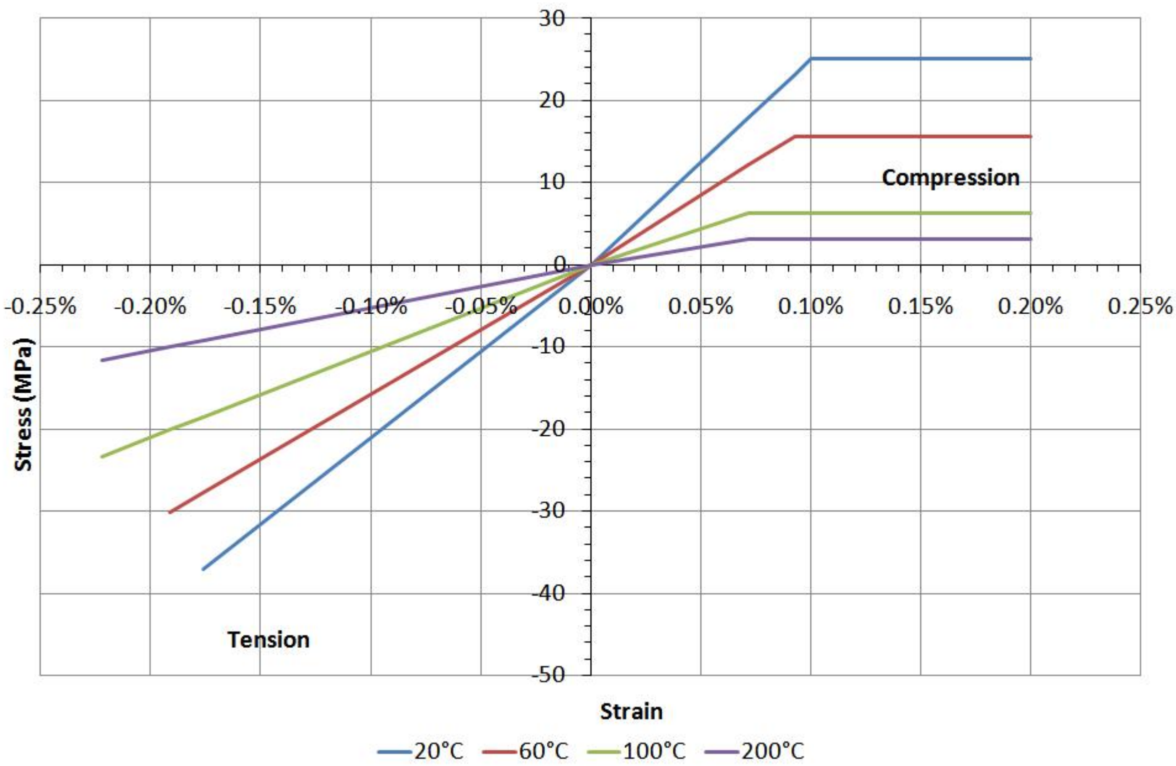



**Figure 1.14 Typical idealised stress strain relationship for timber at elevated temperature (after König & Walleij 2000)**

## 1.4 AIMS AND OBJECTIVES OF THE RESEARCH PROJECT

The overall aim of this project is to advance knowledge in relation to the fire safety performance of buildings formed from engineered timber products and systems. Within this overarching aim are a number of objectives which are investigated through a combination of small scale laboratory experiments, full-scale fire testing, heat transfer modelling and coupled thermo-mechanical modelling. These methods combine in the development of tools, procedures and computational models aimed at designing timber structures to withstand fire. The objectives of the research project are as follows:

1. To observe the heat transfer characteristics of foam insulation, plasterboard and timber board products exposed to fire. Such products are common in timber based technology such as Structural Insulated Panels (SIPs), closed and open light timber frame systems.

2. To identify ‘failure modes’ of holistic engineered timber SIP and floor joist systems in natural fire conditions.

3. To develop thermo-physical properties appropriate for the simulation of timber systems protected with gypsum plasterboard and insulated with polymeric foams.

4. To investigate the applicability of published timber thermo-physical properties in the simulation of timber structures exposed to parametric fires, and to propose modifications if they are found to be inappropriate.

5. To build a general Finite Element Analysis model, and supporting numerical procedures, suitable for simulating the behaviour of fire exposed timber structures using typical commercial finite element packages.

The methods adopted in achieving the above objectives will be discussed later in the research methodology chapter.

## 1.5 THESIS STRUCTURE

This thesis contains a number of sections and appendices. A general discussion of the research related to the subject area is firstly introduced. This is followed by an outline of the research methodology. The author’s EngD work is then fully discussed. Finally, the findings and implications of the research outcomes are highlighted. 5 papers are included in the appendices. A list of the chapters contained in this thesis, with a brief description of their content, is shown below.

**Chapter 2** - “*Related work*”: The state of the art in relation to timber system behaviour in fires is introduced in this chapter. Studies relating to material behaviour, experiments and modelling are also reviewed. In addition, a discussion of the novelty of the current research is included.

**Chapter 3** - "*Research Methodology*": In this chapter, the research methodology adopted for achieving the project objectives is outlined. This includes a discussion of laboratory experimentation, full scale fire testing, heat transfer numerical studies and thermo-mechanical simulation of engineered timber systems.

**Chapter 4** - "*EngD Research*": An outline of the studies undertaken by the author during the research period is provided in this chapter. In addition, the chapter includes a discussion of experimentally observed failure modes in wood structures exposed to fire, the simulation of temperature development in timber structures and how rational design can be realised through numerical modelling techniques.

**Chapter 5** - "*Findings and implications*": Highlights of the key conclusions of the research undertaken and the wider implications for structural fire engineering design are presented in this section. In addition, possible areas for future investigation are discussed.

**Appendices** - Five papers are included for assessment which have been published in peer reviewed journals and/or conferences.

# 2 RELATED WORK

The behaviour of timber structures in fire has been the subject of research in the building community for over half a century. Many of the world's most reputable research laboratories have studied timber in fire either from a charring perspective or a more general structural performance perspective. One of the early studies of note performed on timber structures was conducted in the USA where the charring rates of fire exposed timber were studied in California (Butler and Martin 1956 and Saur 1956). In parallel to this and subsequently, recognised research institutions, such as the Fire Research Station (now BRE), conducted research focussed on fire resistance (Lawson, *et al.* 1951), timber burning rates (Thomas, *et al.* 1967) and charring rates (Rogowski 1967 and Butler 1971). Comparable research was also undertaken in the USA, Canada and Scandanavia following similar themes (Schaffer 1977, Lie 1977 and Hadvig 1981).

The concept of designing timber structures to resist fire was not significantly advanced until a decade later. Janssens and White (1994) conducted further research into the charring behaviour of wood and the associated temperature profiles that developed in timber members. However, it was the Swedish Institute for Träteknisk Forskning (SP Trätek), and more generally Scandinavian research institutes, that significantly enhanced knowledge in the area of 'fire resistance of timber' as it is known today. Hansen and Olesen (1991) performed fundamental research on the behaviour of loaded glulam beams exposed to natural fires whilst Östman, *et al.* (1996) were the early pioneers of the performance based design of timber structures. The former was able to characterise the rate of char development in softwoods for different rates of heating. The latter introduced the concept of numerically determining the separating ability of timber structures when used to form compartmentation. Such a concept was innovative as it allowed flexibility in the makeup of timber construction which had previously been specified on the basis of rigid prescriptive guidelines (Östman 1996). In addition, SP Trätek furthered knowledge in the area of timber board products (Tsantaridis 1996 and Östman and Mikkola 1996) and gypsum based protection layers (Östman, *et al.* 1994), all of which are adopted on a large scale in UK timber construction today. Although such research significantly advanced knowledge in the area of fire damaged structures, it was still primitive compared to further advancements made by SP Trätek and other institutions towards the turn of the millennium. These are discussed in greater detail in the sections that follow.

## 2.1 FURNACE FIRE TESTING OF TIMBER SYSTEMS

Jürgen König, also of SP Trätek, conducted numerous fire resistance experiments prior to the turn of the millennium (2000). Much of the data gathered through this research currently underpins the timber in fire Eurocode (BSI 2004c). König (1994) was one of the first researchers to acknowledge the impact of load ratio (utilisation), i.e. the amount of load applied to a member relative to its capacity, on the failure time/characteristics of lightweight timber construction. Up until this point, much of the research undertaken had focussed upon heavy weight timber construction and charring rates. König (1994) also acknowledged that the failure of timber members was dependent not only on the rate of char formation but also on the degradation of strength at temperatures below that at which timber ignites and char forms. The study, through a large number of furnace experiments on stud walls at differing load levels, gave relationships between failure time (in standard fires) and the utilisation level of the wall. Predictably, fire resistance (in relation to failure time) was shown to be highly dependent upon load level. However, up until this point the relationship was not quantifiable and often not considered for light timber construction.

König and colleagues continued to advance knowledge through further studies on furnace exposed timber members following non-standard fire time-temperature relationships (König *et al.* 1995/1997 and König and Walleij 1999). Up until this point, from a design perspective, the behaviour of timber in natural fires had not been widely considered. However, their initial research in this area (König, *et al.* 1995) focussed upon the verification of charring equations derived on the basis of experiments by Hadvig (1981), and Hansen and Olesen (1992) for natural or parametric design fires. The experiments had empirically quantified the differences in charring rate when timber members are exposed to natural fires with varying heating rates compared to furnace exposure. These relationships, once verified by König and colleagues, formed the basis of the Annex A charring method in Eurocode 5 part 1.2 (BSI 2004c). SP Trätek conducted a more generalised series of experimental parametric studies for timber systems exposed to non-standard fires (König, *et al.* 1997 and König and Walleij 1999). The first of these studies comprised a number of furnace experiments heated following various time temperature regimes. Light timber frame wall and floor assemblies were investigated and the primary motive for the research was to validate a numerical model which was subsequently developed by König and Walleij (2000). The influence of load ratio and heating regime on the failure time of stud walls and joist floors, protected with varying specifications of plasterboard and insulated with different materials, provided data for the validation of early timber numerical models. The study also provided an opportunity to investigate the charring behaviour of initially protected light timber frame assemblies and the fall-off characteristics of passive fire protection in non-standard fires, which was later investigated in more thorough detail (König and Walleij 1999). Like much of König's work the experimental data were used to develop analytical methods for use by designers in EN 1995-1-2 (BSI 2004c).

In parallel to the work undertaken in Europe, similar research was conducted in the USA and Canada. The North American research institutes were early pioneers of research in relation to charring rates of timber. These institutes (NRC, FPL, etc) also conducted research into the fire performance of light timber frame assemblies. Studies by Sultan *et al.* (1994a/b and 1998) and Kodur *et al.* (1996) were focussed upon the collection of basic temperature and structural data for the validation of numerical models.

It is apparent from the literature that the decade prior to the turn of the millennium represented the computational era, particularly in relation to timber research, where many authors, both in Europe and North America, attempted to simulate the fire resistance behaviour of building elements. As a result, many small scale highly instrumented experiments were conducted on timber stud walls filled with varying insulation and protected with different types of gypsum boards. An example of this is the results reported by Kodur *et al.* (1996). Thermocouples were typically placed in key locations of interest in relation to thermal performance and the data were used in the validation of the developed simulation tools. Elemental fire resistance testing was also supplemented with materials testing to determine thermo-physical properties for implementation in models. Benichou *et al.* (2000 and 2001), also of NRC, conducted numerous material tests and gathered thermal properties such as conductivity, specific heat and mass loss as a function of temperature for various materials. The studies covered many of the materials typically used in timber construction such as insulation, gypsum plasterboard and other relevant materials.

Given the motives for the ‘fire resistance’ experiments performed in that period, the resulting developed numerical models are discussed in more detail in Section 2.3.

## 2.2 NATURAL FIRE EXPERIMENTS ON TIMBER SYSTEMS

Experiments on the natural fire resistance of any type of structure were, and still are, rare. The most high profile ones conducted to date are those completed by the Building Research Establishment (BRE) in the 1990's on concrete (Bailey 2002) and steel framed structures (Newman, *et al.* 2006). An image of the steel building can be seen in Figure 2.1. However, there have been instances where natural fire research has been conducted on timber structures. Examples of this are compartment fire tests conducted by Hakkarainen (2002), the Timber Frame 2000 (TF2000) project undertaken by BRE (Lennon *et al.* 2000) and the recent fire testing of a cross laminated structure by Frangi *et al.* (2008).

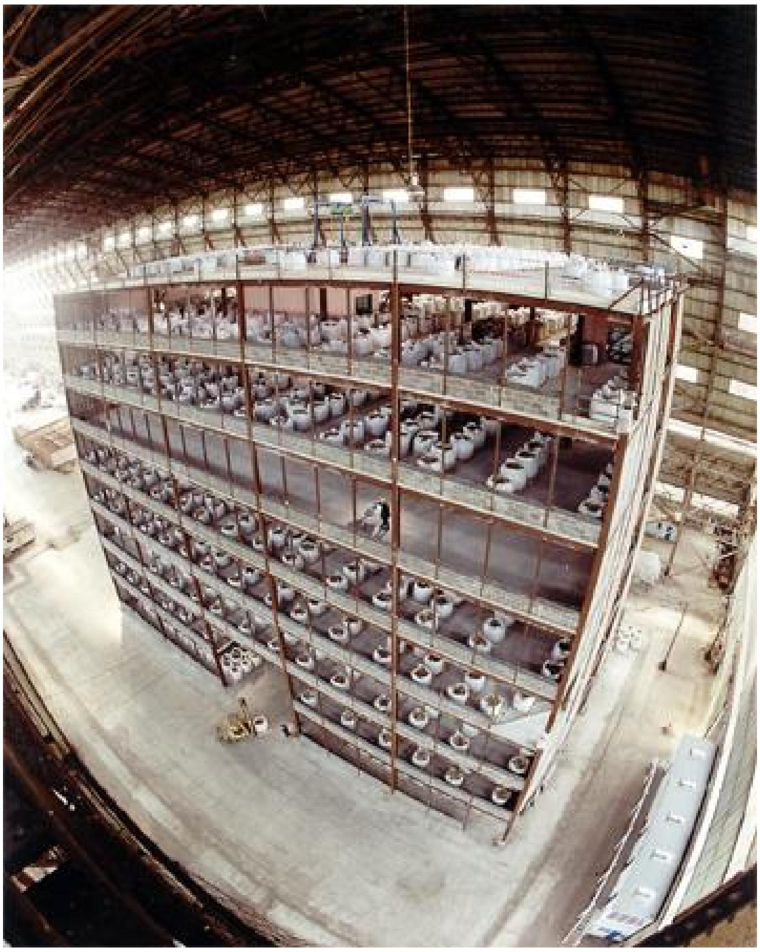


**Figure 2.1 Steel building Cardington fire experiments**

The TF2000 (Lennon, *et al.* 2000 and Grantham, *et al.* 2003) project conducted in 1999 is undoubtedly the most high profile fire test performed on a timber structure to date. A large scale compartment fire was conducted in a six storey timber framed structure representing a typical multiple occupancy residential building. A single flat on the second floor of the building incorporated a fire load, comprising timber cribs, which was ignited and allowed to burn naturally without suppression. The fire brigade were asked to intervene after approximately 60 minutes. The objectives of the experiment were to assess whether the compartmentation of the building was effective in preventing fire spread from the flat of origin to adjoining flats through party walls, windows, floors or communal stair and lift shafts. In addition, the buildings ability to maintain the integrity of the means of escape and structural stability was also assessed. The programme demonstrated that timber frame construction can meet the functional requirements of the Building Regulations for England and Wales, in terms of limiting fire spread and maintaining structural integrity, for an appropriate length of time (Lennon, *et al.* 2000). An image of the TF2000 building post test can be seen in Figure 2.2.

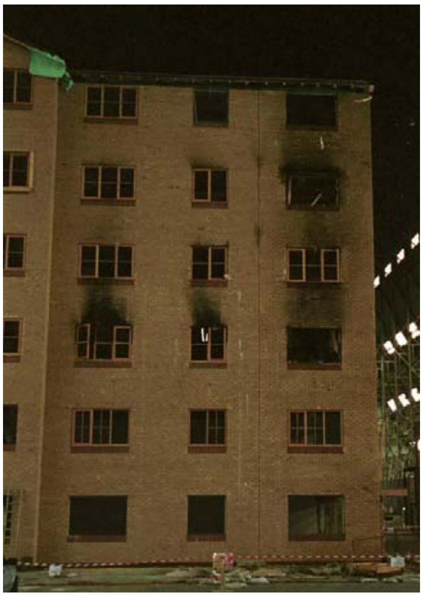


**Figure 2.2  The TF2000 building post fire test**

Hakkarainen's (2002) study of fires in timber structure compartments followed the TF2000 in 2001 and comprised four experiments. Three of these experiments were conducted in compartments formed from heavy laminated timber with varying degrees of passive fire protection. The first experiment was conducted without a lining to the timber structure, the second comprised a single layer of standard plasterboard protection (Type A) and the third comprised two layers of plasterboard, one standard (Type A) and one fire resistant (Type F). The fourth and final experiment was conducted in a light timber frame compartment protected with two layers of plasterboard as for experiment three. The overall purpose of the research was to aid the development of an analytical model for simulating the fire dynamics within timber compartments. The varying degrees of protection resulted in differing thermal inertias within the compartment which led to different fire dynamics. However, useful observations were also made in relation to plasterboard failure time, the onset of charring, charring rates and total depths of char even though the primary objective was to observe the fire behaviour.

After the research of Hakkarainen (2002), further research followed by Frangi and Fontana (2005), concerning the fire performance of full scale modular timber hotel units. The objective of the research was to establish the impact of different fire safety provisions on the fire performance of the timber units. In particular the influence of suppression systems was investigated, which included fast response sprinklers. Based upon six full scale fire experiments Frangi and Fontana (2005) were able to conclude that the fire safety requirements of timber modular hotel units can be achieved through the provision of suppression systems, despite the inherent combustibility of the structural frame and wall linings. Further to this, the experiments identified the potential for timber structures to prevent the spread of fire beyond the compartment of origin, should fire develop in unsuppressed timber buildings.

More recently, a fire test has been performed in Japan on a three storey cross laminated timber building (Frangi, *et al.* 2008). The overall building size was approximately 7m x 7 m on plan and 10m in height. The walls and the floors of the building were formed from cross laminated timber panels. A fire load comprising mattresses and timber cribs was ignited on the first floor of the building and allowed to burn for approximately 60 minutes. Much like the TF2000 project (Lennon, *et al.* 2000) the structure was shown to be able to fulfil its functional requirements in relation to preventing fire spread. However, no structural measurements were taken in relation to deflection, stresses or strains. Residual depths of char were however measured in a 300 mm grid for all surfaces bounding the fire compartment. After some 60 minutes, due largely to the protection afforded by the gypsum lining to the room, depths of char not in excess of 10 mm were measured.

## 2.3 MODELLING OF FIRE EXPOSED TIMBER SYSTEMS

Researchers have been attempting to model temperature development in wood structures exposed to fire for some 20 to 25 years. One of the first successful attempts is published by Fredlund (1993). He developed a one dimensional finite difference model named “WOOD1” which not only considered the heat transfer aspects of thermal behaviour but also mass transfer caused by the flow of pyrolysis products and moisture. In the case of heat transfer Fredlund (1993) assumed that energy was transferred via thermal conduction and convective flow of volatile pyrolysis products and water vapour. Where mass transfer was considered the model assumed gas flows driven by pressure gradients. Again, the gases considered were water vapour and pyrolysis products. This approach, although novel and shown to be consistent with experimental data, was very complex. Therefore, further efforts followed by authors both in North America and Europe to create simplified models.

Following the above, Mehaffey *et al.* (1994) and Sultan (1996) were amongst the first researchers to adopt ‘effective’ thermal properties in light timber frame fire models which included gypsum plasterboard. In the context of this research effective properties are those that are derived on the basis of material experiments and do not explicitly consider complex behaviour, such as mass transfer. These projects, although conducted in the research area of timber in fire, were largely focussed upon the heat transfer behaviour of gypsum boards. Mehaffey, *et al.* (1994) and Sultan (1996), in support of the development of numerical models, conducted material experiments to determine the conductivity, specific heat and mass loss rate of gypsum plasterboard, as a function of temperature, using tools such as conductivity meters and differential scanning calorimeters. As a result, ‘apparent’ properties were directly measured which implicitly included the effective actions of complex behaviour such as ablation and moisture flow. Unlike the model of Fredlund (1993) such an approach negated the need to consider mass transfer, cracking and other behaviour explicitly, as the properties measured represented the materials of interest as a whole, and not the constituents from which it was formed. This is the preferred approach of most practitioners and researchers today who model timber and gypsum systems as it allows the thermo-physical properties to be implemented in more general finite element formulations and software packages. Both Mehaffey, *et al.* (1994) and Sultan (1996), using ‘in-house’ developed finite element codes, were successful in accurately simulating temperature development in light timber frame stud walls. The models were extensively validated and the thermal properties presented therein have since been used in further studies (Takeda and Mehaffey 1998, Clancy 2001 and Thomas 2002).

Whilst the USA and Canadian research institutes largely focussed upon the thermo-physical characteristics of gypsum board, Thomas (1997), and König and Walleij (2000) conducted similar research into the thermal and structural characteristics of lightweight and large section timber. The modelling approach and properties adopted in the former of these studies would later be included in Annex B of Eurocode 5 part 1.2 (BSI 2004c).

In his first numerical studies, Thomas (1997) assumed, in the absence of supporting test data, a relationship between strength, stiffness and temperature. The relationship between compressive strength and stiffness was assumed to decrease rapidly as the timber temperature approached 100°C. 100°C was the temperature assumed to correspond with the temperature at which all free moisture in the timber had evaporated. The loss of tensile strength and stiffness was assumed to be less pronounced and less sensitive to moisture flow. These properties would be used to simulate earlier tests conducted by König, *et al.* (1997) on stud walls, with credible accuracy.

The study of König and Walleij (2000) built upon that of Thomas (1997) and comprised two parts. Firstly, a model to determine depth of char and timber temperature was developed called "TEMPCALC" which was an extension of the finite element code TCD 3.0. Secondly, a mechanical model was developed to determine the consequences of temperature on timber's structural performance. The latter was highly innovative as it was one of the first studies to acknowledge that plastic flow can occur in timber, under compressive stresses, in a fire. It also acknowledged the short comings of the timber grading process, presented in standards like BS EN 338:2003 (BSI 2003), as it highlighted the difference between localised strength and stiffness and those determined through grading tests.

The thermal element of König and Walleij's (2000) research included thermo-physical properties for timber, as a function of temperature, gathered from a number of sources. This includes 'effective properties' gathered by König and Walleij (1999) in an earlier study, and specific heat values proposed by Janssens (1994) for standard fire exposure. These were then implemented in a finite element code to simulate char line movement in timber sections. The results were benchmarked against the vast array of testing undertaking by SP Trätek and were found to be sufficiently accurate for inclusion in Eurocode 5 part 1.2 (BSI 2004c). However limitations were imposed in the Eurocode to confine the use of these properties to standard fire exposure only.

In the mechanical modelling of fire exposed timber structures König and Walleij (2000) coupled the thermal model “TEMPCALC” with a spreadsheet model that undertook sectional analyses of fire exposed timber cross sections. As noted earlier, the researchers acknowledged that grading strengths were limited by defects and that, for modelling purposes, these should be substituted for localised strengths. These localised values corresponded to the properties of small pieces of “clear” wood without, or almost without, defects (König and Walleij 2000). The “true strength” as defined by König and Walleij (2000) was determined using bending correlations developed by Thunnel (1941) who derived relationships between grade bending strength, determined assuming a linear stress distribution, and true tensile strength, considering plastic flow in the compression region. These “true strengths” where then coupled with the strength reduction versus temperature functions assumed by Thomas (1997). Through numerical calibration against known behaviour from experiments König and Walleij (2000) were able to refine the strength and stiffness relationships proposed by Thomas (1997) to those now included in Annex B of Eurocode 5 part 1.2 (BSI 2004c). It was these properties that resulted in the most promising correlation with earlier experiments performed by SP Trätek (König, *et al.* 1997 and König 2006).

## 2.4 STATE OF THE ART

Timber in fire research has not evolved significantly in the last decade. Most of the areas of interest pre-millennium, such as charring behaviour and numerical modelling, remain of interest today. These areas in particular were the focus of studies in the early 2000's where Frangi and Fontana (2003) further experimentally investigated charring rates and temperature development in softwood slabs. The study was different to those that had been conducted previously as it approached charring rates from a statistical perspective, defining confidence limits for given rates of charring. A similar study was undertaken by Njankouo *et al.* (2004). However, in this instance, the charring rates of tropical hardwoods were studied. Further progress was made in the modelling of light timber frame assemblies by Clancy (2001) and Thomas (2002). The latter, in particular, provided new thermo-physical properties for plasterboard which were based on latent heat of evaporation energy calculations and are now widely used as a substitute for the properties previously developed by Sultan (1996). Similarly, Janssens (2004) developed the finite difference model CROW (Charring Rate of Wood) which considered heat and mass transfer in the prediction of depths of char in timber sections. The CROW model, due to its fundamental 1D approach adopting explicit timber and char properties, was believed to be applicable to alternative fire exposure conditions to the ISO 834 (ISO 1999) fire exposure. However, no formal validation was reported. Finally, Benichou (2004) built upon his earlier work (Benichou, *et al.* 2000 and 2001) and the studies of Sultan (1996) to develop a simple sectional analysis tool for assessing the fire resistance behaviour of floor joist assemblies. This was validated against experiments performed at NRC but only for standard fire exposure.

The modelling of timber floors in fire and the fundamental behaviour of heat transfer in light timber frames remains an area of interest for many researchers who continue to develop numerical codes designed for this purpose (Schnabl and Turk 2006, Craft, *et al.* 2008, Ang and Wang 2009 and Takeda 2009). However, from a practitioner's perspective, no practical tool exists for the design of timber structures exposed to fire. This is because existing models are complex in their interpretation of physical phenomena, whilst remaining limited in terms of the range of geometries that can be implemented and the fire exposure conditions adopted.

### 2.4.1 APPLIED RESEARCH

In addition to the more fundamental research mentioned, many recent applied studies have focussed upon developments for the pending update of Eurocode 5 part 1.2 (BSI 2004c). König (2006) investigated the applicability of thermal properties (in particular conductivity), presented in Annex B of Eurocode 5 part 1.2, when adopted in simulations investigating timber under non-standard fire exposure. König (2006) argued, and proved, that since the properties in the code were derived through calibration against data from standard fire tests the same properties could not be applied to natural fires or more simple representations, i.e. parametric fires. In recognition of this, he proposed that the conductivity values of the char layer (that is, those properties at temperatures greater than the pyrolysis temperature of timber) should be modified to improve the compatibility of modelling simulations with non-standard experimental results. The research did not lead to a relationship between the level of modification required and the severity of the natural fire. However, it did set out a framework for how simulations of temperature development in timber sections, exposed to natural fires, could be developed as a result of simple modifications to the conductivity properties of the char layer. In addition, introducing artificial fire loads in the cooling phase of fires was proposed in order to account for the additional temperatures seen in timber structures as a result of char oxidation.

In an analogous vein, Cachim and Franssen (2009) conducted comparative studies to establish the compatibility of the advanced calculation models contained in Annex B of EN 1995-1-2 with the charring rates proposed in the main body of the EN 1995-1-2. It was found that the charring rates included in EN 1995-1-2 were only applicable to timber moisture contents of 12% and an ambient wood density of 450 kg/m$^3$. As a result, they proposed modified charring rates that were dependent upon moisture content and softwood density. Also, in an extension to the thermal properties in Annex B, Cachim and Franssen (2009) proposed changes to the specific heat properties to make them applicable to variations in timber moisture content. At present, the specific heat properties in EN 1995-1-2 are only valid for timber with a moisture content of 12%.

Whilst the studies of König (2006), and Cachim and Franssen (2009) identified the limitations of the current version of EN 1995-1-2 (BSI 2004c) Frangi *et al.* (2010) and Just (2010) have been active in the development of new calculation methods aimed at superseding those currently in existence in the code. Frangi and colleagues (2010) identified that the scope of applicability of the "separating function" design equations in EN 1995-1-2 (Annex E) was limited. The separating function annex provides calculation procedures for the determination of the insulation and integrity fire resistance performance of a timber floor or wall. However the method, in its current form, was deduced from a limited number of fire tests on wall assemblies and linings. Therefore, only a limited number of types of timber structure are applicable. Recognising this, Frangi, *et al.* (2010) adapted and refined the existing method based on new experimental data, resulting in a modified method applicable to a range of materials more representative of current timber structures.

In a similar manner, Just (2010) recognised the limitations of the existing Annex C methods in EN 1995-1-2 (BSI 2004c). These are used for the calculation of the load bearing resistance of fire exposed timber assemblies with cavities that are completely filled with insulation. In its present form, the Eurocode allows the methods to be applied to insulations of either rock mineral wool or glass fibre type. However, Just (2010) established, through a range of experiments, that a fundamentally different behaviour is observed in fire exposed assemblies that are insulated with rock mineral wool compared to those insulated with glass fibre. As a result, modified calculation methods have been proposed for inclusion in EN 1995-1-2 which specifically address the performance of glass fibre insulated assemblies.

Finally, the breakdown and the fall off characteristics of gypsum plasterboard have been investigated by Just, *et al.* (2010) and Sultan (2010). Based on these studies, two contrasting methods for predicting the fall off time of different configurations of plasterboard in standard fire tests were proposed. The first, by Just, *et al.* (2010), is based on the use of a database of standard fire test observations to propose simple pessimistic relationships between fall off time, plasterboard grade and thickness. Different relationships are proposed for plasterboard when used to line walls or floors. In the second method, published by Sultan (2010), the use of a ‘critical temperature’ concept for the determination of plasterboard fall of time is proposed. Essentially, this suggests that a configuration of plasterboard will fail and begin to fall off once it reaches a certain ‘critical temperature’. The critical temperatures put forward in the research are empirical, determined from fire resistance tests. The former method by Just, *et al.* (2010) was developed for the purpose of providing new plasterboard failure times for implementation in the reduced cross section method presented in EN 1995-1-2 (BSI 2004c).

### 2.4.2 ENGINEERED TIMBER RESEARCH

The research discussed above was primarily conducted on timber technology that may be considered ‘traditional’. This includes solid section stud walls, joists floors and heavy weight construction. However, timber research has evolved as a result of innovations in the way in which timber buildings are now constructed and the use of engineered timber. The latter has already been discussed covering innovations such as engineered floor joists, SIPs and laminated timber. As the popularity of such products has increased, the need to conduct research into their fire performance has become a priority.

The fire performance of engineered floors in particular has been the subject of much research in the USA and Canada. In such countries it is not uncommon for light timber floors to remain unprotected (that is, not lined with plasterboard as would be standard in the UK). In response to a number of fire incidents in North America, Forintek (Richardson 2004), the Underwriters Laboratory (Izydorek, *et al.* 2008 and Backstrom, *et al.* 2010), NRC (Benichou, *et al.* 2010) and Tyco (Avila 2008) have performed independent research into the fire performance of unprotected engineered floors. All of the investigators concluded that unprotected engineered floors composed of steel web joists, timber composite I-joists or timber lattice girders are more likely to fail and collapse earlier in fires, relative to traditional solid section joist floors. All the studies indicated that in unprotected floors, failure was characterised by a very sudden increase in deflection before collapse. However, although failure times were noted for various forms of construction, the researchers did little to explain the possible failure mechanisms associated with these systems. The research, although interesting, was also of limited practical use to European researchers and designers as it would not be standard practice to allow such floors to remain unprotected. Further to this, the studies of Richardson (2004) and Izydorek, *et al.* (2008) were performed in furnaces heated in accordance with the standard fire curve and thus cannot be considered a true reflection of behaviour in real fires.

During this same period, Bregulla (2003) was one of the first researchers to study SIP behaviour in fires in any depth. The study investigated the fire performance of SIPs through various scale experiments ranging from cone calorimeter studies through to full scale standard fire furnace experiments. The performances of Pyrox, Sasmox and OSB faced SIPs with polyurethane (PUR), polystyrene (PS) and polyisocyanurate (PIR) insulation were also considered. However, although the study may have been considered novel at the time, today it is of limited application. This is because Bregulla (2003) largely focused upon the performance of unprotected OSB faced SIPs and SIPs faced with cement particle board. In the case of the former, the SIPs were predictably shown to ignite rapidly under fluxes typical of a compartment fire (35 to 50 $kW/m^2$). In the UK today, SIPs are almost exclusively OSB faced and would always be lined with at least 12.5 mm gypsum plasterboard.

In addition to the above investigations, further research has been undertaken in Europe in relation to the fire performance of cross laminated timber. The primary motives for the research were to develop a fundamental understanding of behaviour in fires through experiments (Frangi, *et al.* 2009) and to develop calculation methods for future revisions of EN 1995-1-2 (Schmid, *et al.* 2010). Interest in the area of cross laminated timber behaviour in fire has been stimulated as a result of the increased adoption of the technology in Europe, leading Frangi, *et al.* (2008 and 2009) to undertake numerous experiments. These experiments were largely focussed on temperature development and char layer formation. Using such tests and through additional experimentation Schmid, *et al.* (2010) developed a numerical sectional analysis tool capable of predicting the moment resistance of CLT elements in standard fires. The key finding of the study was related to inability of the current measures in EN 1995-1-2 to provide safe designs for CLT elements in fire. As a result, recommendations have been made regarding the zero strength layer concept when the reduced cross section method is applied to CLT.

Many of the developments made recently, in relation to both engineered timber and more generally for the purpose of updating EN 1995-1-2, have been published in a European best practice guide titled “Fire safety in timber buildings- Technical guideline for Europe” (Östman, *et al.* 2010). This guideline is intended as an interim guidance document before EN 1995-1-2 is updated and contains state of the art research relating to timber in fire. From a review of this document it is apparent that a number of knowledge gaps exist in relation to both engineered timber behaviour and also the fundamental performance of wood at elevated temperature. This is particularly the case where engineers wish to design timber structures exposed to realistic fire conditions.

## 2.5 NOVELTY OF THE ENGD RESEARCH

A review of the available literature indicates that research on timber in fire has not evolved much since the turn of the century. Since this time many new technologies have emerged and are now common in the UK and European market places. Interim guidance has been published concerning the fire performance of innovative timber systems (Östman, *et al. 2010*); however this is still not entirely comprehensive. New ‘timber products’, such as SIPs and Engineered Floor Joists, currently demonstrate adequate fire resistance via a standard fire test. However, in such a test, the behaviour of a single building component is observed and its ability to survive a prescribed artificial heating regime for a given period of time is measured. Therefore, the test yields little, if any, information about how the same component may behave when acting as a part of a structural assembly exposed to natural fires. To this end, the research conducted by the author aimed to demonstrate, through a series of full-scale fire tests, how innovative timber systems react when exposed to real fires. The experiments, which focused on new engineering technologies, such as structural insulated panel and gypsum protected engineered floor joist assemblies, are the first of their kind conducted worldwide. Therefore, the results and the studies based on these tests should, undoubtedly, make significant contributions to knowledge in the research area.

The performance based design and simulation of fire exposed timber structures is another area considered in this research. Available literature shows that, in spite of the significant research reported by SP Trätek at the turn of the century (König and Walleij 1999/2000), little if any other advances have been made. An important reason for this is the fact that timber thermal properties, including those set out in BS EN 1995-1-2, are still very much limited to standard fire exposure. This means that the temperatures of timber structures exposed to natural (or non standard) fires cannot be adequately determined, and that subsequent structural simulation cannot be performed using numerical tools and techniques such as FEA. These limitations, imposed by inadequate timber thermo-physical characteristics, have been recognised for some time (König 2006). However, little, if any, research has been undertaken to overcome such barriers. In this thesis simple but effective modifications to the thermal properties of softwood timber are proposed. The modified values should allow for the accurate determination of the temperature distribution within structural members exposed to natural (parametric) fires.

Further to the above, to demonstrate how such properties may be used in performance based design, the commercial finite element code TNO DIANA was adopted to model the behaviour of fire exposed timber structures. It has been identified through the literature review that timber is a complicated material. Section 1.3, which outlines the aspects of material behaviour unique to timber, discusses how complex through a review of material properties. From this it can be seen that timber is an orthotropic material whose tensile and compressive strength and stiffness degrades at differing rates upon heating. In addition, when heated, timber undergoes an irreversible change of state where solid timber becomes friable char. Both of these aspects make modelling timber with 'standard' commercial codes challenging. As a result, König and Walleij (2000) and Schmid, *et al.* (2010) have resorted to the development of simplistic cross sectional analysis tools for determining how timber behaves when exposed to fire. This is a perfectly suitable approach. However, its field of application is limited to single members in isolation and cannot be adopted to determine how timber members and systems interact at elevated temperature. To overcome this, the author has proposed a number of user supplied sub routines which can be implemented within commercial finite element computer codes to simulate timber behaviour in fire. As a result more complex systems, such as frames or sub-frames, can be simulated when exposed to fires, inclusive of the cooling phase.

Finally, it is apparent that many modern timber buildings are extremely light weight and as a result have little or no inherent fire resistance. This is apparent from both research undertaken by the author (which will be discussed in Section 4.1) and other research discussed in Section 2.4.2. As a result, the fire resistance of many modern timber systems (in particular light timber frame) is wholly reliant upon the fire performance of gypsum plasterboard protecting it. As a result, an understanding of the heat transfer aspects of plasterboard is critical for effective fire design. Many of the classic studies relating to the performance of plasterboards in fire are still referenced today (Mehaffey, *et al.* 1994, Sultan 1996 and Thomas 2002). However, plasterboard has evolved dramatically since the conclusion of these research projects. New formulations and admixtures are now apparent in their manufacturing. Minerals such as glass, vermiculite and bentonite are commonly introduced to enhance plasterboard’s fire resistance. As a result, many of the thermal properties given in reputable plasterboard studies (such as those highlighted previously) may not be appropriate for adoption in simulations of modern boards. Further to this, few studies have investigated the heat transfer characteristics of gypsum exposed to natural fires. In acknowledgement of the above, the author has investigated both of these aspects thoroughly through a combination of computational modelling, laboratory furnace tests and full scale natural fire experiments.

# 3 RESEARCH METHODOLOGY

The investigation of structures exposed to fires requires a diverse research methodology comprising laboratory experiments, large scale fire testing and numerical modelling. Laboratory experiments and numerical modelling are often a necessary element of research as full scale fire testing is extremely costly and rare. Only a few institutions in the world are equipped with the necessary facilities to undertake large scale fire experimentation of which BRE is one. To this end, the research project comprises a hybrid research methodology which adopts all of the methods outlined above. Each of these is discussed in more detail in the following sections.

## 3.1 LABORATORY TESTING OF TIMBER COMPOSITES

The investigation of the heat transfer behaviour of elements of construction requires both a heat source (i.e. a furnace) and extensive data logging tools, such as thermocouples and software. BRE’s structures laboratory at Garston contains a number of gas powered furnaces which can be used to investigate the fire performance of building products. These facilities were used to investigate the heat transfer behaviour of SIPs protected with gypsum plasterboard. The experimental approach comprised single SIPs rigidly fixed against the open door of a gas powered furnace (Figure 3.1). Such an approach allowed for both the heat transfer behaviour and combined heat and loading performance of SIPs to be investigated. In the latter case BRE’s 500 tonne compression machine was used alongside a semi-portable gas furnace (Figure 3.2). More information on the results can be found in Section 4.1.2.

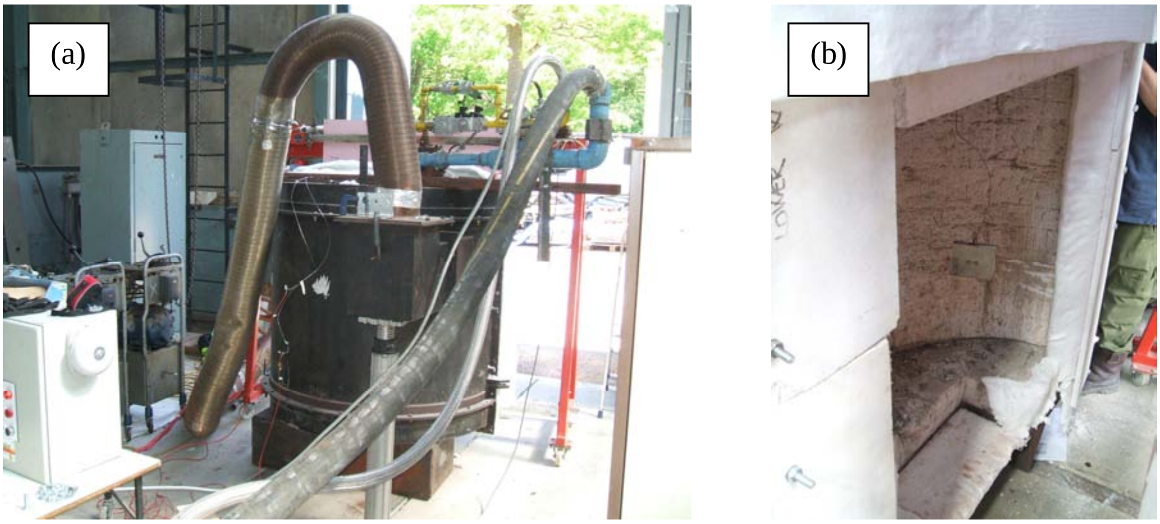



**Figure 3.1 (a) External view of barrel furnace (b) Open side of the furnace**

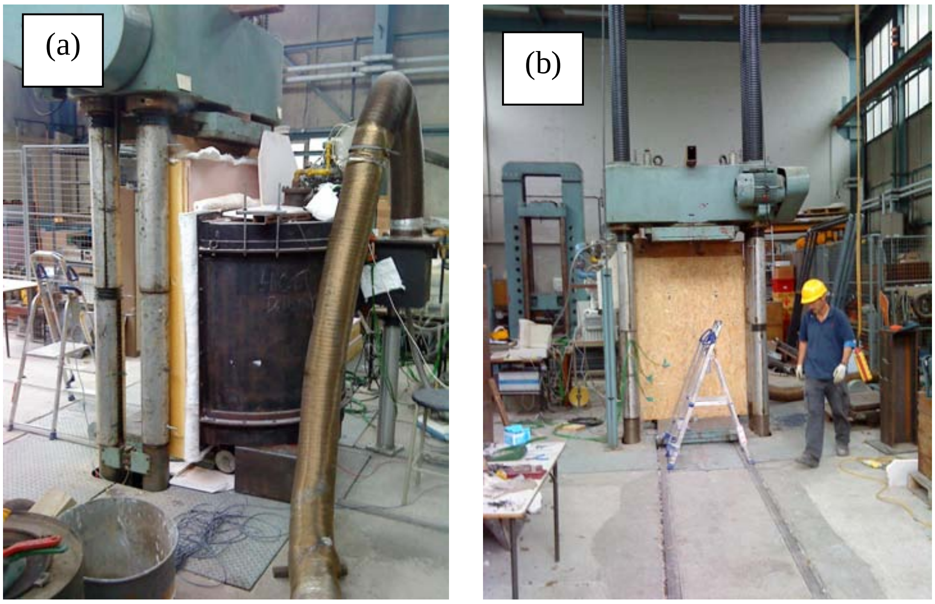



**Figure 3.2 (a) Isometric of 500 tonne compression rig and furnace (b) Rear elevation**

## 3.2 FULL SCALE FIRE EXPERIMENTS

The fire testing of full scale building systems is the most effective means of researching fire performance. Many complex behaviours are likely to be apparent when structural elements interact at elevated temperature. Such phenomena cannot be observed in laboratory tests on singular building components. In addition, unforeseen mechanisms for fire spread are often only apparent when entire buildings are exposed to fire. Clearly, large scale fire experimentation is the most credible link to real fire behaviour, both in terms of fire development/spread and structural performance. Often, especially in the case of fire investigation, only forensic based assessments of performance are possible from which it is difficult to derive why and how structures may have failed. Comparably, large scale fire experiments allow for the observation of structural and fire behaviour in real time.

BRE has a long history and a vast experience in undertaking full scale fire tests on buildings. Much of this stems from the iconic research conducted at Cardington in the 1990's. The author has had the opportunity to conduct seven full scale fire experiments during the research period which are discussed in Section 4.1. All of these experiments were conducted in BRE's North East fire facility where entire buildings or sub elements (single storeys) were exposed to natural fires formed using timber cribs. Illustrative images are shown in Figure 3.3.

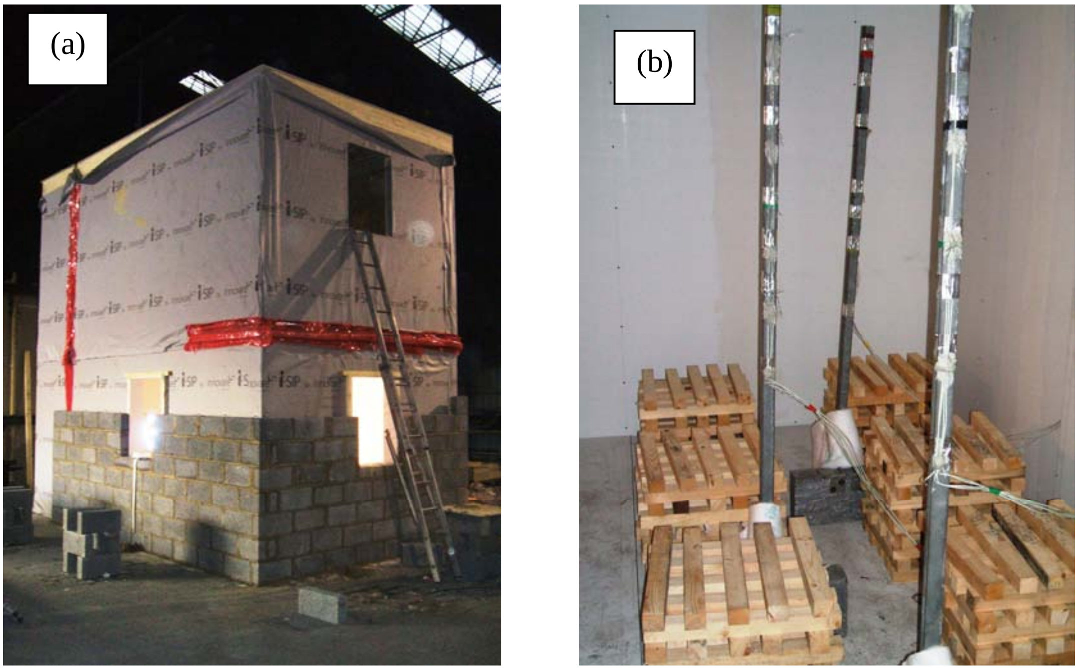



**Figure 3.3 (a) Two storey SIP fire test building (b) inside compartment with fire loading**

## 3.3 NUMERICAL MODELLING

Numerical modelling, be it thermal, mechanical or coupled is an invaluable tool for aiding understanding in relation to how structures behave at elevated temperature. Holistic fire experimentation is not only costly but also has the potential to cause significant damage. As a result, it is often only possible to measure a limited number of parameters such as displacements, temperatures and heat fluxes. The measurement of more interesting parameters, such as stress and strain, is often only possible using very expensive instrumentation or optical methods. As a result, numerical modelling provides an opportunity to observe such parameters through careful calibration and validation against experiments.

In support of both the laboratory and full scale fire experiments, extensive numerical modelling of the thermal, mechanical and coupled thermo-mechanical behaviours was conducted. The commercial finite element package DIANA, developed by TNO BV in the Netherlands (Manie 2010), was adopted for this purpose. TNO DIANA (Figure 3.4) is a robust Civil Engineering specific commercial software package which, through a combination of additional FORTRAN routines and the introduction of temperature dependent behaviours, can be used to simulate both the heat transfer behaviour and the mechanical behaviour of building elements and systems exposed to fire. The specific implementation of DIANA will be discussed in Sections 4.2 and 4.3.

Much of what is known about structures in fire has been derived using a similar or identical research methodology and is common amongst many classical studies and doctorates completed in this area. This research methodology is illustrated in Figure 3.5.

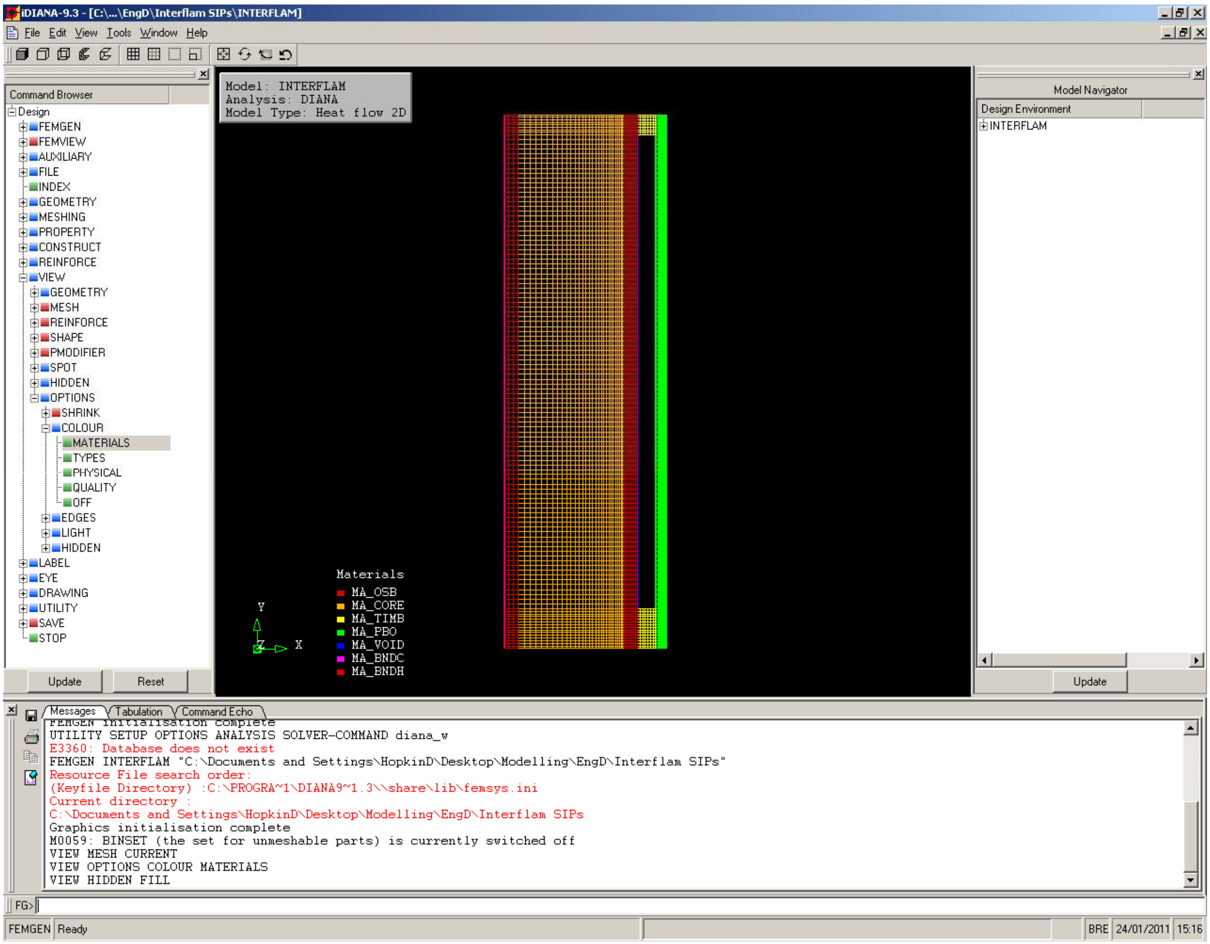



**Figure 3.4 TNO DIANA finite element software**

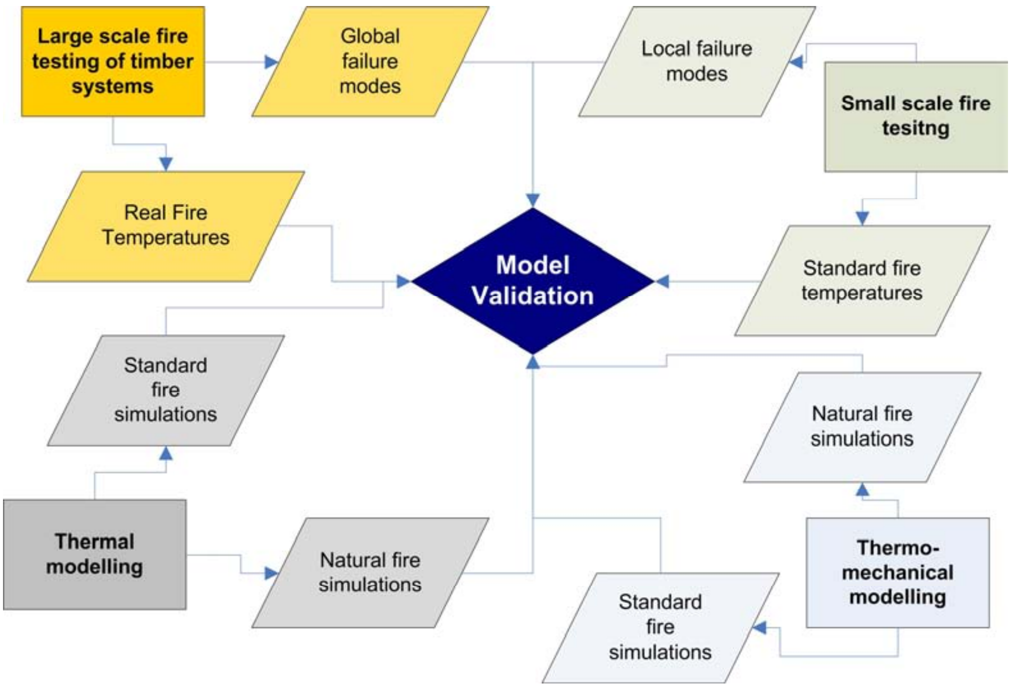



**Figure 3.5 Research road map**

# 4 ENGD RESEARCH

In this section of the thesis, original work undertaken by the author during the EngD programme is discussed. This is supplemented with publications which can be found in Appendices A to E. The work conducted comprised laboratory and natural fire experiments, coupled with heat transfer and thermo-mechanical modelling. This research methodology has been discussed previously in Section 3. The work is discussed in the sections that follow. Relevant publications written by the author and appended are highlighted as appropriate.

## 4.1 UNDERSTANDING FAILURE MODES AND BEHAVIOUR IN REAL FIRES

The research review discussed in Section 2 clearly showed that the fire performance of engineered timber products had not been subject to significant investigation. Where investigations had been conducted they have been largely limited to small scale standard fire tests. Such tests only tell part of the story in relation to failure modes and do not yield behavioural aspects that emerge as a result of system interactions.

Specific to engineered floor joist behaviour in fire, at the time of this EngD projects inception, only limited studies had been conducted in Canada (Richardson 2004) post millennia. These experiments exposed engineered floor joists of different types to standard fires without gypsum protection. However, the true performance of structures in fire can often only be evaluated via large scale natural fire tests, as noted previously. Such experiments have been conducted by the author on engineered floors and are discussed in Section 4.1.1.

Studies conducted in relation to Structural Insulated Panel (SIP) behaviour in fires were only reported by Bregulla (2003). These investigations, although novel at the time, are of limited value today as SIPs have changed significantly since the research was undertaken. Firstly, Bregulla (2003) researched SIPs formed with Sasmox and Pyrok facing boards. Secondly, almost all experiments conducted were on unprotected small scale samples. Today SIPs are almost exclusively OSB faced and protected by gypsum linings. As a result, further investigations of 'modern' SIPs exposed to fire have been conducted in this project. The results of both laboratory and natural fire experiments are discussed in Sections 4.1.2 and 4.1.3 respectively. The latter, similar to engineered floors, has resulted in a better understanding of systemic failure modes which arise when multiple building components interact.

### 4.1.1 NATURAL FIRE EXPERIMENTS ON ENGINEERED FLOOR SYSTEMS

The main aim of this part of the project was to investigate the performance of a protected timber floor system connected to load-bearing walls through proprietary connections. The systems were subject to a typical value of imposed load and a fire scenario that included direct flame impingement. In all cases the joists were loaded such that their utilisation (load ratio) was approximately 40% of ultimate capacity. The experimental programme involved testing three different floor systems, typically used in residential applications, exposed to a realistic fire scenario and realistic conditions of loads and restraints. Full details of this part of the research programme can be found in a paper that has been put forward for assessment in Appendix A. However, for completeness a summary of the findings of the experiments, and of the paper, is given below.

**Summary**

Three fire tests were conducted on different timber floor systems: (i) solid timber floor joists, (ii) I-Joist engineered floor beams with solid timber top and bottom flanges and an OSB web, and (iii) an engineered timber truss incorporating solid timber upper and lower chord members and a pressed steel web member. The latter two reflect the two most common types of engineered floor systems used in the UK and allow for direct comparison with a more traditional form of construction. Each joist type tested can be seen in Figure 4.1.

In each case, a system representing a separating/compartment floor was selected such as would be used to separate different occupancies within an apartment building. For this reason, the floors required 60 minutes fire resistance. To achieve this, guidance on the appropriate level of fire protection, was taken from manufacturer's information.


**Figure 4.1 Images of the joist systems tested (a) Solid joist (b) I-Joist (c) Steel truss web joist**

Based on the results of the test programme, it was concluded that engineered floors may be able to offer the same fire resistance as that of solid timber joist floors, provided that the engineered joists are properly protected from fire. This requires the use of adequate boarding, and ensuring that a good quality of installation is maintained during construction. However, when exposed directly to fire, some engineered joists may fail in a more rapid manner compared to that of solid timber joists. This conclusion was supported by the following observations:

- The performance of the engineered I-section joists showed that this type of floor may be capable of providing 60 minutes resistance to natural fire scenarios provided that two layers of 15 mm fire resistant plasterboard are used as recommended by the manufacturers. However, the need for more tests is recommended to assess the exact behaviour of such joists if exposed directly to fire due, for example, to failure of the lining boards.
- When exposed to fire directly, the behaviour of engineered truss joist floors resulted in a more rapid mechanism of failure. The test showed that under this condition this type of floor developed large deflections, and continued to deflect at a high rate over a short period of time leading to a sudden catastrophic failure of the floor system. This mode of failure was not observed in the solid timber joists test. The steel modules forming the web of the section were detached due to charring of the timber chords which caused the connecting plate to lose its bond. It was suggested that more tests are needed to determine whether the use of different type of connectors, the provision of thicker plasterboards or a combination of both may improve the performance of the floor.
- The chipboard flooring offered some contribution to the overall fire resistance of all of the floor systems tested by delaying the spread of fire if the ceiling void is breached. It also may offer additional structural resistance by acting as a stress skin should some of the joists become damaged.
- The joist hangers were shown to be capable of surviving 60 minutes exposure to a natural compartment fire with little or no damage observed.
- The deflection of the engineered truss joists was almost three times that of the solid timber joists after 60 minutes of fire exposure.

Post experiment images of all three types of joist tested can be seen in Figure 4.2.

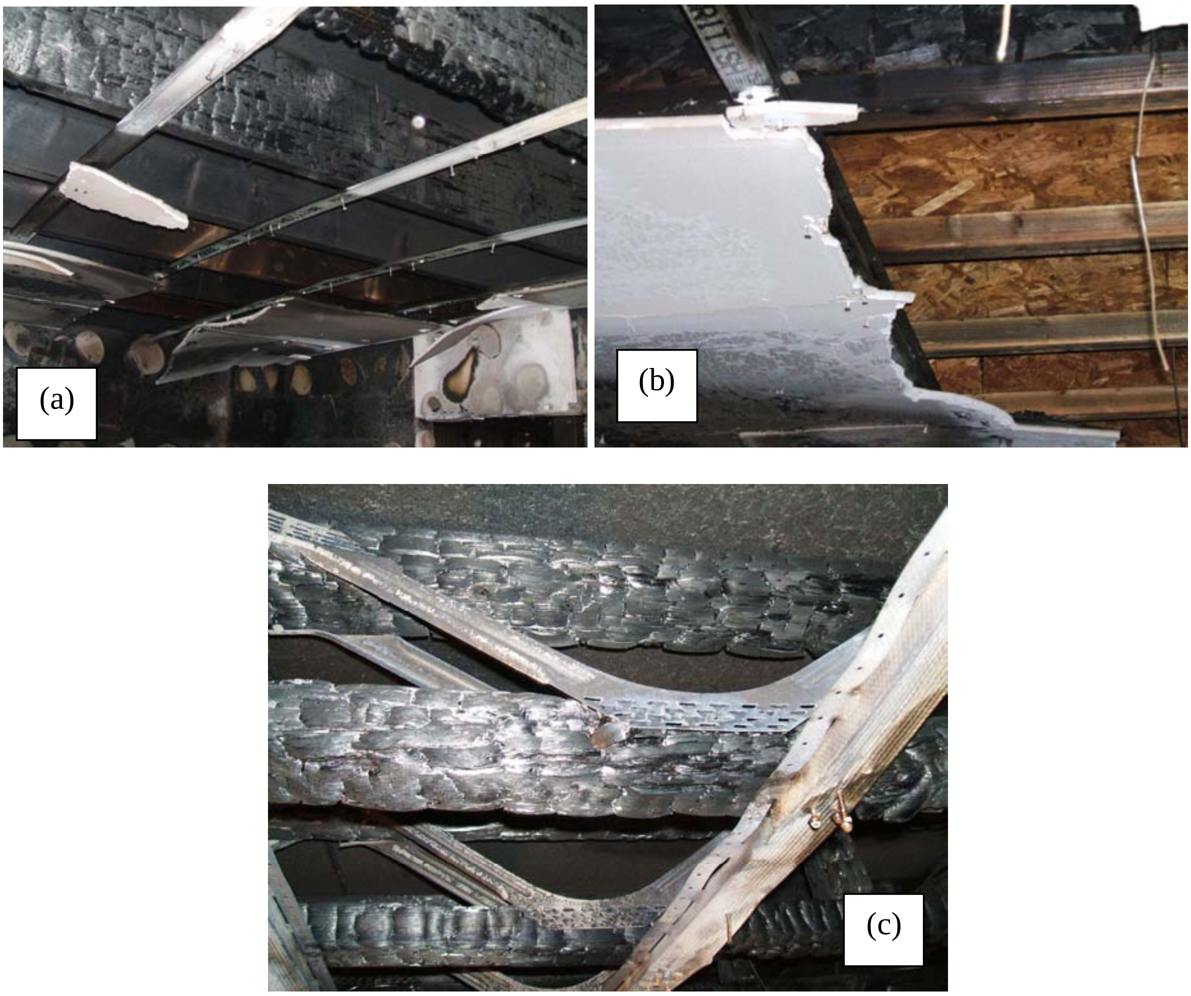



**Figure 4.2 Systems after the fire tests (a) Solid joist (b) I-Joist (c) Steel truss web joist**

### 4.1.2 LABORATORY STUDIES OF STRUCTURAL INSULATED PANELS

Much like the engineered floor joist systems discussed in Section 4.1.1 and Appendix A, little testing or research has been undertaken to investigate the fire performance of SIPs. This is apparent from the literature reviewed in Section 2. Therefore, with a clear knowledge gap and the growing adoption of SIPs in the UK, the author has undertaken research into the fire performance of SIPs. The first aspect of this is concerned with a laboratory study into the structural, heat transfer and combined heat and load behaviour of isolated panels.

In total, twenty four experiments were performed on single SIPs of overall dimensions 1200x1800x150 mm in BRE's structures laboratory. In all instances the panels had two 15 mm thick OSB skins. These experiments were divided into three distinct categories: testing panels under uni-axial compression, tests for measuring heat transfer, and tests which combined heat and uni-axial compressive loading. The full experimental programme is summarised in Table 4.1 with corresponding test references which will be used in the identification of test samples hereafter.

**Table 4.1 Summary of laboratory experiments**

| Test No. | Test | Test reference | PFP | Lining connection |
|---|---|---|---|---|
| 1-2 | ULF | PUR_L1/L2 | None | N/A |
| 3 | ULF | EPS_L1 | None | N/A |
| 4-6 | HT | PUR_301-303 | 15 mm PB (Type F) | Battens |
| 7-9 | HT | EPS_301-303 | 15 mm PB (Type F) | Battens |
| 10-12 | HT | PUR_601-603 | 30 mm PB (Type F) | Battens |
| 13-15 | HT | EPS_601-603 | 30 mm PB (Type F) | Battens |
| 16-18 | HL | PUR_301-303 HL | 15 mm PB (Type F) | Battens |
| 19-21 | HL | PUR_601-603 HL | 30 mm PB (Type F) | Battens |
| 22-23 | HL | EPS_301-302 HL | 15 mm PB (Type F) | Battens |
| 24 | HL | EPS_601HL | 30 mm PB (Type F) | Battens |

**PFP -** Passive fire protection; **ULF -** Ultimate load; **HT -** Heat transfer; **HL -** Heat and load; **PB -** Plasterboard; **EPS** (Expanded Polystyrene); **PUR** (Polyurethane).

**Type F** refers to fire resistant plasterboard as per BS EN 520:2004 (BSI 2004d).

### Uni-axial ambient compression tests

Three ambient temperature compression tests were performed on SIPs. Two of the panels had PUR cores and the other one had an EPS core. The experiments were conducted to (i) establish how the panels failed at ambient temperature due to a purely compressive load, and (ii) determine an appropriate level of loading which should be applied in the combined heat and loading tests.

All samples were placed in BRE's 500 tonne compression machine and were tested to failure. In both PUR panels, failure was initiated with a sudden brittle crack which propagated through the thickness and width of the OSB facings. Local to these areas, some micro-buckling and kinking was apparent in the OSB strands. In addition, some de-lamination occurred local to the crack site where the OSB and polymer insulation separated. This can be seen in Figure 4.3.


**Figure 4.3 Failure mode in an isolated SIP at ambient temperature**

The results of the three compression experiments performed on SIPs are summarised in Table 4.2 ($\delta x$ and $\delta z$ denote lateral and vertical deflection, respectively). It should be noted that the abnormally high level of loading achieved in the EPS panel is due to the presence of solid timber edge (splining) studs which framed the test sample. These are a common form of joining panels and can introduce high levels of redundancy into the structural system. They did however give a false indication of the panel's true ultimate load bearing capacity. As a result, only the PUR panels' ultimate loads were used to derive loading levels for subsequent furnace tests.

**Table 4.2 Summary of ambient axial load experiments**

| Reference | Ultimate load (kN) | Peak δx (mm) | Peak δz (mm) |
|---|---|---|---|
| EPS_L1 | 647 | 2.7 | 10.8 |
| PUR_L1 | 331 | 2.4 | 16.4 |
| PUR_L2 | 293 | 8.9 | 28.7 |

### Heat transfer experiments

A total of twelve furnace tests were performed on unloaded SIPs protected with gypsum (Type F) plasterboard, fixed via 25x50 mm vertical battens at 600 mm centres.

The experiments performed were simple and comprised a SIP pressed up against BRE's barrel furnace. All of the gaps were sealed using ceramic fibre blanket. The gas furnace was then ignited and was manually controlled to follow the ISO fire curve (ISO 1999) by monitoring a plate thermocouple inside the furnace. Furnace temperatures were logged using bead thermocouples. Temperatures within the SIP panel were monitored and logged at the centre line of the panel at three different heights corresponding to the third (X and Z) and mid-height (Y) levels. The placement of thermocouples through the panel depth is shown in Figure 4.4. At each height, thermocouples measured the temperatures at the back of plasterboard (A), back of front OSB skin (B), mid-core (C), back of rear OSB skin (D), and at the unexposed face (E).

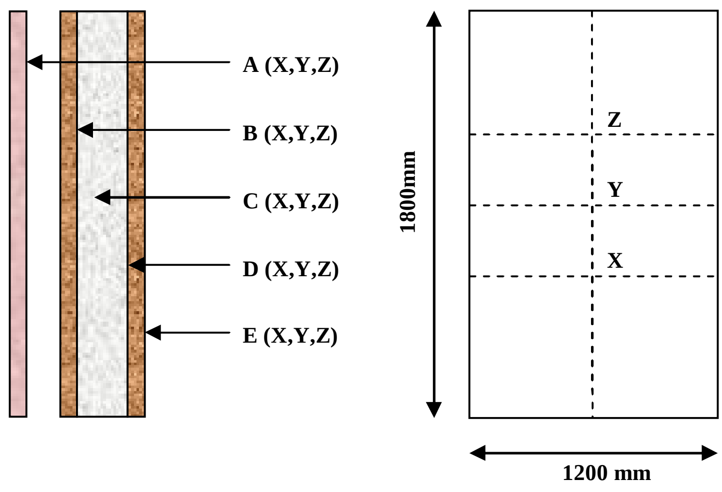



**Figure 4.4 Locations of thermocouples through the depth and height of panels**

Sample measurements of OSB and timber batten moisture content were measured using a moisture meter. The mean moisture content of the OSB veneers was 9.2% and the corresponding value for the timber battens was 11.7%, by weight.

Heat transfer results

In the first six tests of the heat transfer programme, PUR and EPS panels protected with 15mm (Type F) plasterboard were exposed to 30 minutes of the ISO (ISO 1999) fire curve using BRE's gas furnace. All samples survived the duration of the experiment without a need for premature termination. In addition, there was no indication of combustion below the plasterboard lining during the experiments. The temperatures at the back of the plasterboard (Location A) are shown in Figure 4.5. Mean temperatures for PUR and EPS panels are also shown. These are derived as the arithmetic average of all PUR and EPS data sets, respectively. Similarly the temperatures at the back of the OSB (Location B) are shown in Figure 4.6, again with averages for each core material as noted previously.

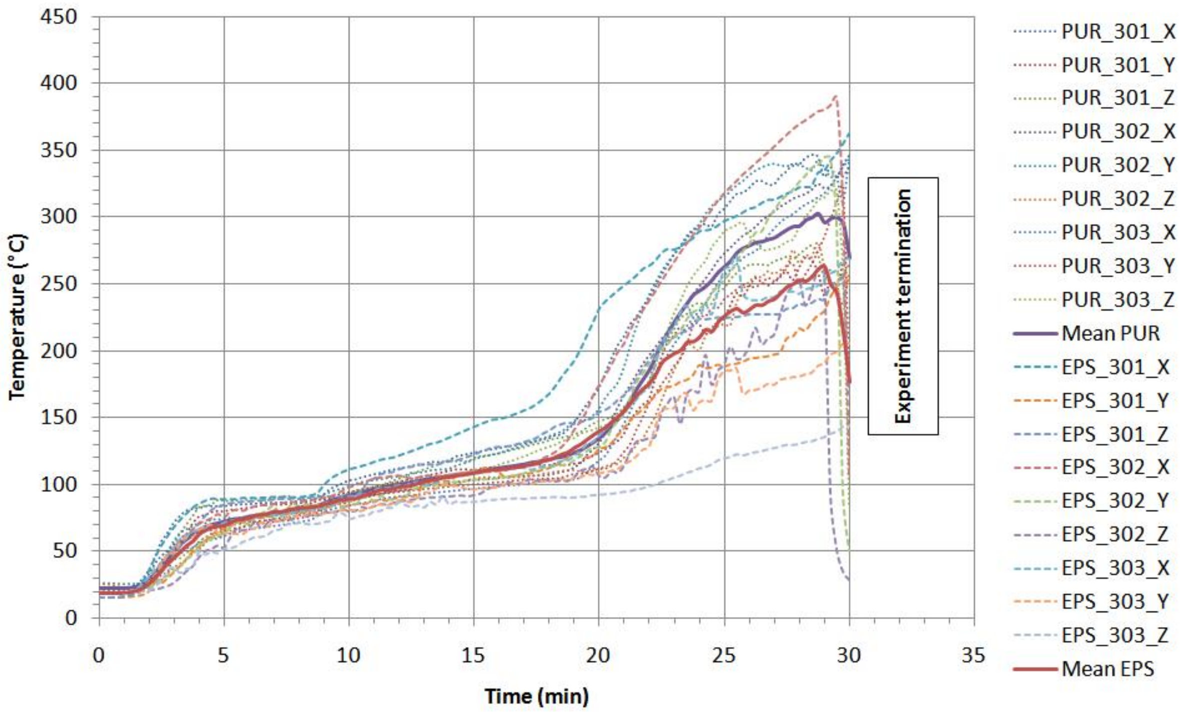



**Figure 4.5 Location A temperatures for all 30 minute experiments**

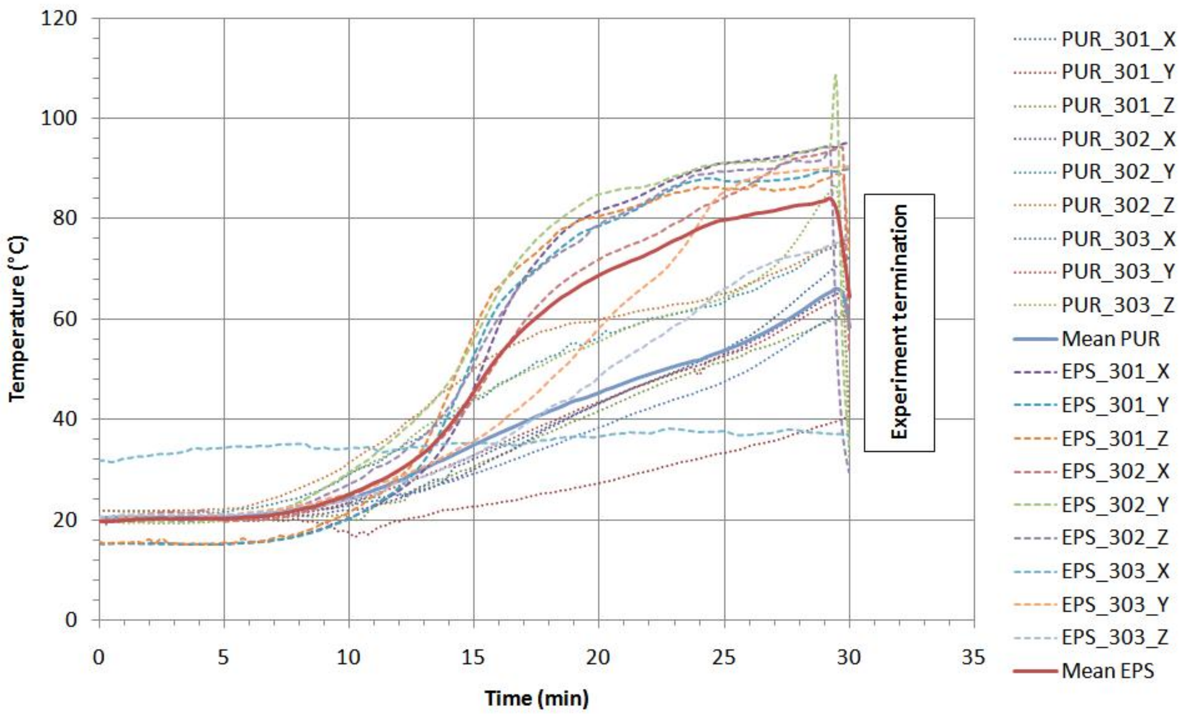



**Figure 4.6 Location B temperatures for all 30 minute experiments**

A further six tests were conducted on PUR and EPS panels protected with 30 mm of Type F gypsum plasterboard and exposed to 60 minutes of the ISO fire curve. In all instances, the panels survived the full duration of the experiment with no indication of combustion internally within the panel. Temperatures measured at the back of plasterboard (Location A) are shown in Figure 4.7. These reflect the temperatures behind two layers of 15 mm Type F plasterboard. An average temperature for each core material is also shown for completeness. The corresponding temperatures at the rear of the OSB are shown in Figure 4.8 with the arithmetic mean for both core material types.

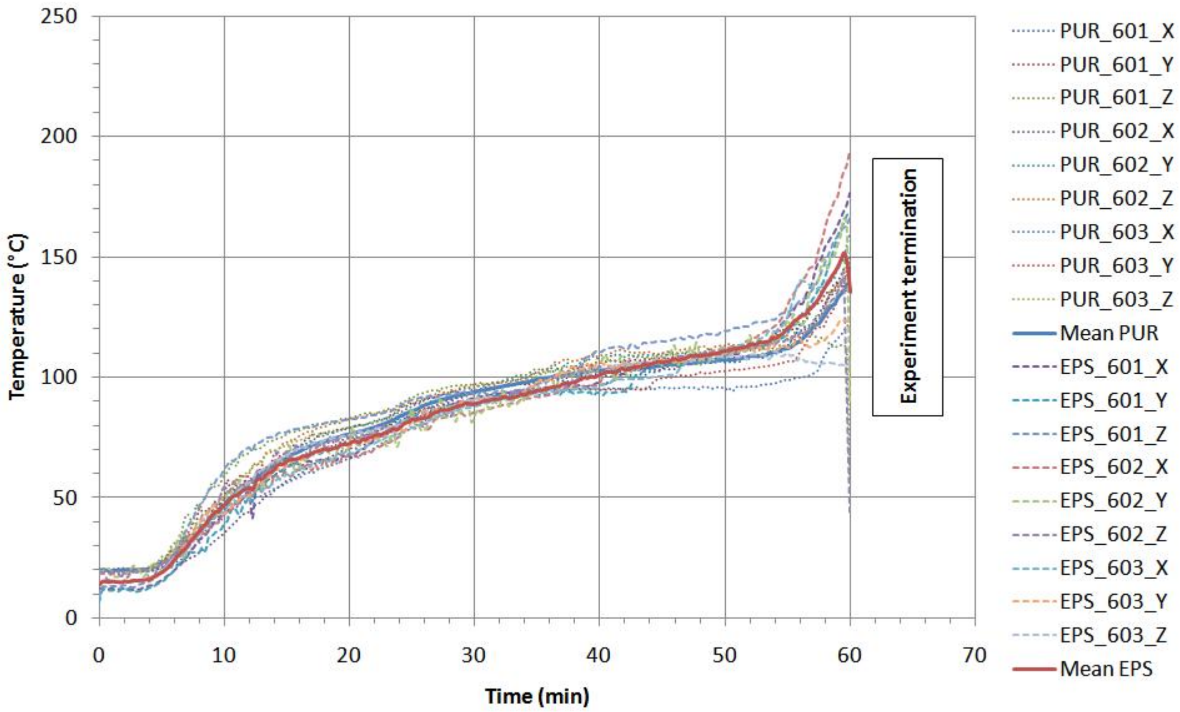



**Figure 4.7 Location A temperatures for all 60 minute experiments**

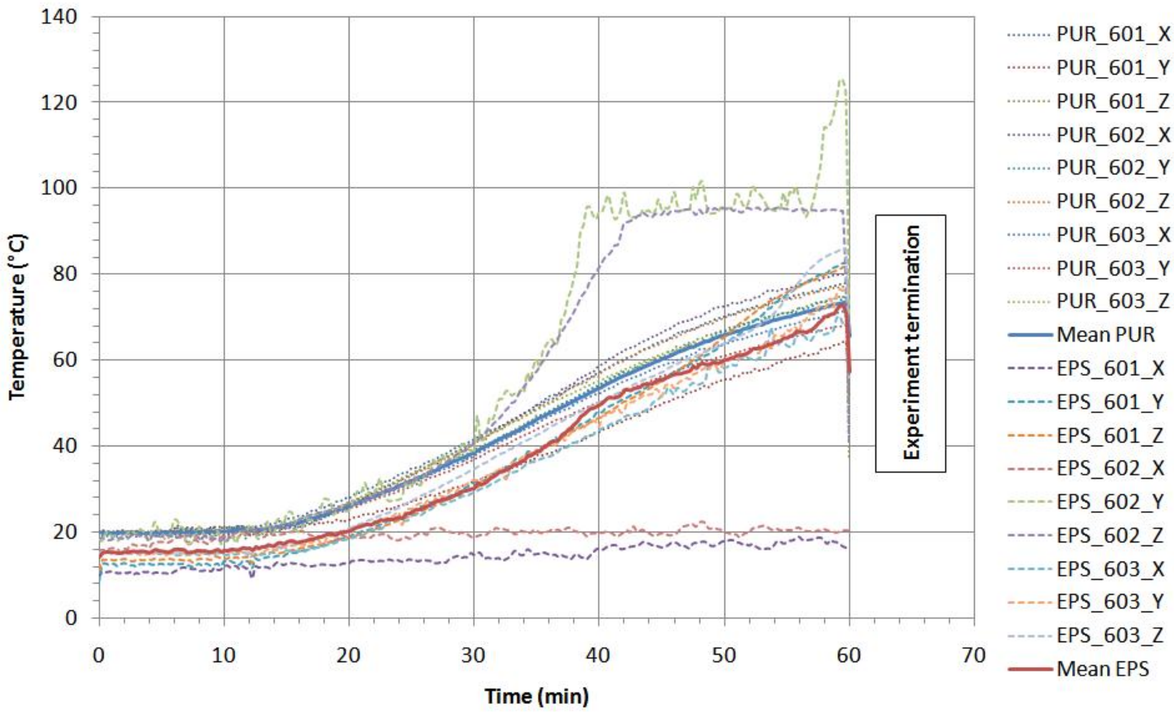


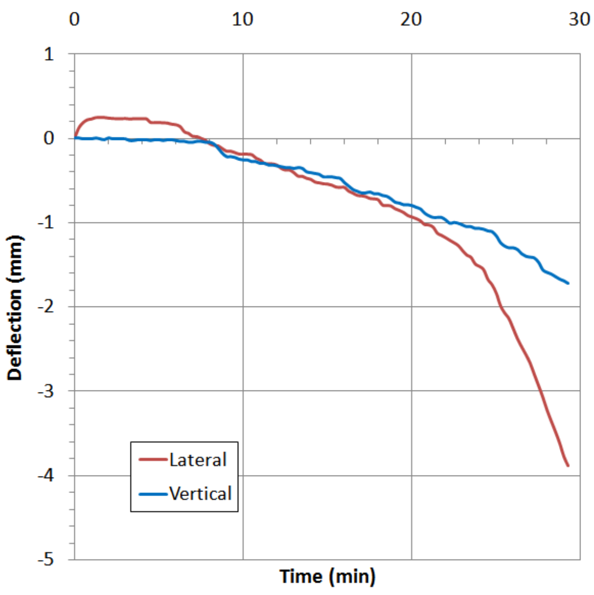

**Figure 4.8 Location B temperatures for all 60 minute experiments**

## Combined heat and load experiments

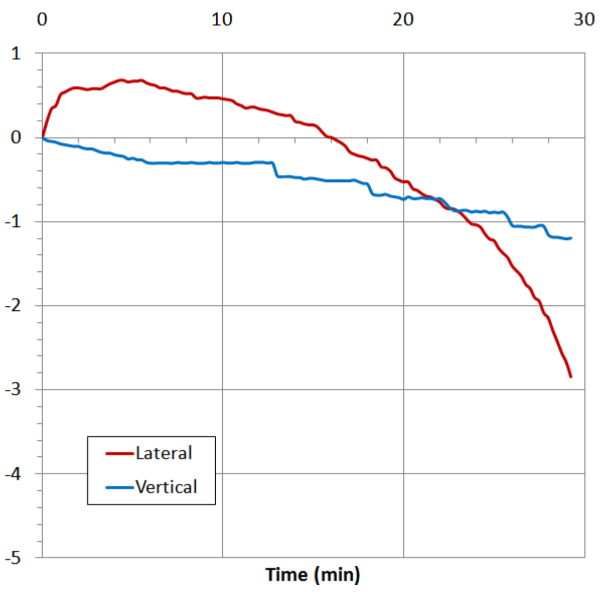
Nine combined heat and loading tests were completed as part of the laboratory programme. Each panel was first loaded in uni-axial compression to a load corresponding to 50% of the mean ultimate load achieved by the PUR test samples at ambient temperature. This equated to a target load of 130 kN or 108.3 kN/m. Once loaded, the furnace was ignited and one side of the panel was exposed to either 30 or 60 minutes of the ISO fire curve, depending on the plasterboard lining specification (15 or 30 mm Type F plasterboard). Similar to the heat transfer element of the programme, PUR and EPS panels were compared. Six PUR samples were studied each with either 15 mm or 30 mm of Type F plasterboard. Three EPS samples were tested, similarly with 15 mm or 30 mm of plasterboard lining. The instrumentation specification essentially merged the requirements of the heat transfer and ultimate loading elements of the research programme. The thermocouple placement and specification corresponded with that of Figure 4.4, whilst displacement transducers measured the mid-height lateral (out of plane) and rig cross head (vertical) deflections. The applied loading was logged using a load cell in the compression machine.

Results of the combined heat and load tests

The temperatures measured within the 30 minute loaded panels were very similar to those reported previously for unloaded specimens. Therefore, they will not be discussed further here. Sample mid-height lateral and vertical deflection are plotted versus mean furnace temperature for experiments PUR_303HL and EPS_302HL in Figure 4.9. The other experiments performed correlated well with these test results. In the graph, positive deflection denotes movements downwards and away from the furnace for vertical and lateral deflection respectively. The mean loading for experiments PUR_303HL and EPS 302_HL, throughout the 30 minute duration, was 127 kN and 126.5 kN, respectively, compared to a target load of 130 kN.



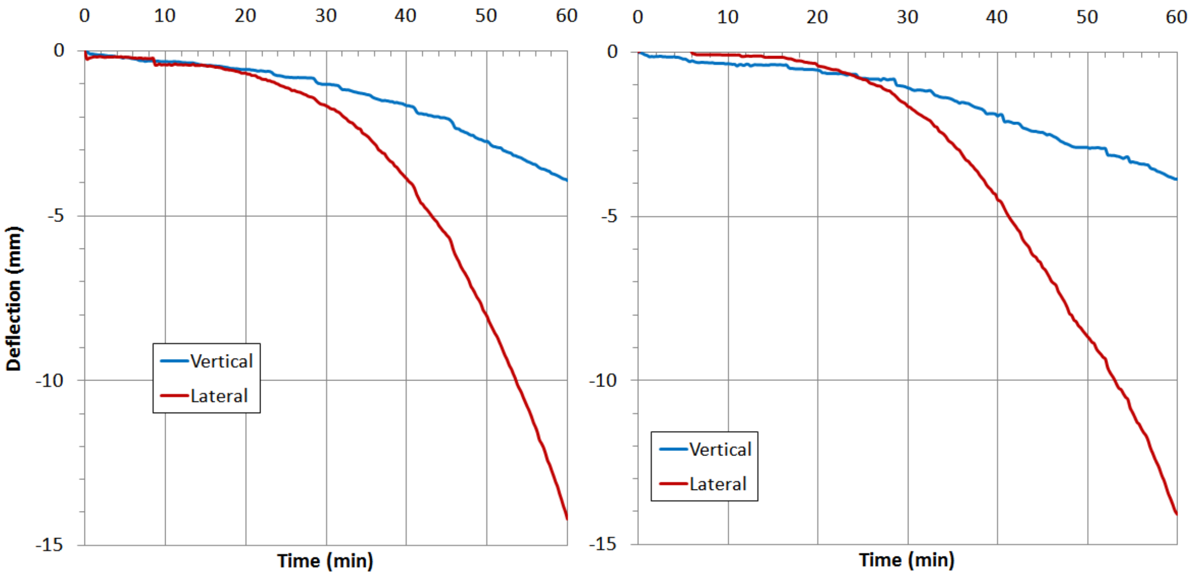

(a)




(b)

**Figure 4.9** **Time versus vertical and lateral deflection for (a) PUR_303HL (b) EPS_302HL**

Similar to the 30 minute tests, all temperatures measured within the 60 minute samples were in agreement with those measured previously on unloaded samples. For this reason the results are not discussed further here. The three tested PUR panels all survived 60 minutes exposure to the ISO 834 curve without loss of load-bearing capacity under mean loads of 125.5 kN, 124.2 kN and 124.7 kN, respectively. Experiment EPS_601HL was terminated after 34.5 minutes due to ignition of the insulation core. However, this was a failure that arose as a result of the test setup and cannot be considered as a true failure of the panel. Hot gases issuing from poorly sealed gaps around the furnace edges ignited the exposed insulation edges (which would not be present in a real building), bypassing the passive fire protection. Typical plots of the deflection versus mean furnace temperature are shown in Figure 4.10 for samples PUR_601HL and PUR_602HL. All experiments performed under 60 minute heat and loading conditions were generally in good agreement. However, due to the early termination of experiment EPS_601HL, no appreciable deformation developed due to the proportionally lower temperatures in the panel. The mean loading during the experiment was 82.5 kN. This was due to the almost instantaneous crushing of the timber spreader beam placed between the SIP and the compression machines loading platen. The damage to the spreader beam occurred on initial loading prior to the ignition of the furnace and as a result a consistent load of 82.5 kN was achieved with few fluctuations during the experiment duration (34.5 minutes).




(a) (b)

**Figure 4.10 Time versus vertical and lateral deflection for (a) PUR_601HL (b) PUR_602HL**

## Discussion

<u>Heat transfer experiments</u>

A number of temperature profiles have been determined for SIPs protected with different specifications of gypsum plasterboard. The study has indicated that, in all instances, the specification of Type F plasterboard of thickness 15 mm or 30 mm is sufficient to achieve the UK fire resistance periods of 30 and 60 minutes, respectively. The specification of 15 mm (Type F) plasterboard for 30 minute fire resistant applications has been shown to prevent the ignition of the insulation core (regardless of what material is used). In addition, it keeps the OSB temperatures sufficiently low to prevent structural integrity issues. Results from the 30 minute experiments showed that there is little or no appreciable difference in the temperature development through the depth of the panel for PUR or EPS insulation. Any difference noted is largely due to the migration of steam through the more permeable EPS insulation.

Where steam, due to the evaporation of water chemically bound in plasterboard, is allowed to permeate into the insulation core some local damage can occur in EPS panels due to the low glass temperature of polystyrene. This damage manifests itself as de-lamination between the core and 'fire-side' OSB veneer.

Although no combustion was apparent during the duration of any of the 30 minute heat transfer experiments, some flaming was noted once the samples were removed from the furnace. Due to the severe temperature gradient, indicated previously in Figures 4.5 and 4.6, the 'fire-side' OSB veneer reaches sufficiently high temperatures to ignite (in excess of 300°C). However, while protected by the plasterboard, there is insufficient oxygen for this to occur. On removal of the samples from the furnace, portions of the cracked gypsum plasterboard were removed. This resulted in instantaneous combustion of the underlying pre-heated OSB face, due to the sudden availability of air. This is shown in Figure 4.11.

**Figure 4.11 Combustion of the fire side SIP OSB veneer**

The specification of 30 mm of Type F plasterboard has been shown to be more than sufficient for 60 minute fire resistance applications. The temperatures at the rear of plasterboard have been shown to be generally less than 150°C. This results in temperatures at the back of OSB which do not exceed 100°C, regardless of insulation type. There appears to be no appreciable difference in the temperature profiles which develop in PUR or EPS insulated SIPs after 60 minutes furnace exposure. Again, any differences are largely due to steam migration through the more permeable EPS core. Steam migration due to moisture in the plasterboard lining entered the core as a result of penetrations drilled for the placement of thermocouples. No combustion was apparent in any 60 minute test samples either during or post experiment, after removal of the dry lining.

Combined heat and loading experiments

In all experiments there were no instances where load-bearing failures occurred. However, experiment EPS_601HL was terminated due to premature ignition of the polystyrene core. The loading levels imposed were far in excess of those typically allowable in SIP structures as these are limited by the levels achievable in the standard test procedure (typically 30 to 100 kN/m). Deflections in all panels were relatively small and were typically characterised by a gradual creep with time. Sizeable lateral deflections can develop in fire exposed panels (10 to 15 mm). However, these did not result in additional cracking in the plasterboard lining or any cracking in the OSB skins of the SIP. As a result, the temperatures observed in loaded specimens were essentially the same as those in the un-loaded experiments.

For SIPs adopted in 30 minute fire resistant applications (15mm Type F plasterboard), no clear difference was apparent in the load-bearing performance of PUR and EPS variants. Both developed approximately 3 mm lateral deflection and 1 to 2 mm vertical deflection. Due to the increased susceptibility of polystyrene to ignition, it has not been possible to adequately compare the fire resistance performance of such panels with those adopting PUR as an insulant for 60 minute applications.

**Summary**

The experimental work described above is the first of its kind in the UK. Its main aim was to investigate the fire resistance performance of PUR and EPS variants of SIPs with variations in plasterboard specification. The data collected are valuable to those wishing to develop numerical models aimed at simulating the fire performance of SIPs. The development of such models is subject to more discussion in Section 4.2. A number of important conclusions can be derived from the experimental programme:

- The specification of 15 or 30 mm Type F plasterboard has been shown to prevent damage to SIPs exposed to either 30 or 60 minutes ISO furnace exposure
- Due to the degree of redundancy inherent in panels, load levels (up to 108 kN/m) do not appear to play a significant role in the fire resistance of SIPs under furnace conditions
- The introduction of solid timber splining (timber studs used to connect panels) members vastly increases a SIPs load bearing capacity. The redundancy afforded by such studs in real buildings is likely to improve fire performance by allowing the redistribution of load should panels become damaged.

### 4.1.3 NATURAL FIRE EXPERIMENTS ON SIP BUILDINGS

Building upon the investigations of isolated SIPs exposed to standard fires (Section 4.1.2) the author, in collaboration with colleagues at BRE, investigated the holistic performance of SIP systems subject to fire. The purpose of this aspect of the research was to distinguish between behaviour that could be expected in fire resistance tests, compared to natural fire events.

To determine how SIP systems behaved in real fires, four experiments were completed in BRE's large-scale test facility. The experimental work aimed to simulate realistic fires in single (houses) and multi-occupancy (apartment block) dwellings and they were loaded appropriately using sand bags (see Figure 4.12). The test programme comprised two experiments on EPS SIP structures, one with a 30 minute (F2-EPS30) fire resistant plasterboard lining and one with a 60 minute (F1-EPS60) fire resistant plasterboard lining. In addition, two experiments were conducted on structures formed from SIPs with PUR cores, again, with two types of plasterboard linings (F4-PUR30 and F3-PUR60). A number of observations were made particularly in relation to the performance of the engineered I-Joist floor system and its levels of inherent robustness, and to the procedures of fire fighting for highly insulated frames. More information on the work undertaken can be found in the publication located in Appendix B.

**Figure 4.12 SIP experimental buildings (F1-EPS60)**

**Summary**

The test results and observations highlighted a number of important issues in relation to the inherent fire resistance of the structural system and the role of the Fire and Rescue Services in dealing with fires in SIP buildings:

- SIP systems are capable of achieving the functional requirements in Approved Document B to the UK Building Regulations (CLG 2007) in relation to B2 internal fire spread (linings) and B3 internal fire spread (structure).

- The apparent mode of failure of the system, i.e. panels and floors, is excessive deflection of the first floor caused by ignition and rapid combustion of the engineered floor joists. The rate of deflection increases very rapidly as the floor system approaches collapse. This behaviour is not influenced by the performance of the SIP system and would be the same for other panellised systems, traditionally built timber frame or joists supported on masonry walls. Indicative damage to the floor members can be seen in Figure 4.13.

**Figure 4.13 Damage to engineered floor systems (F2-EPS30)**

- There was no collapse of the floor in any of the tests despite the significant deflections (more than 200 mm or span/20). The chipboard flooring appears to have contributed to the stability of the floor at large deflections.

- There was no collapse of the wall panels in any of the tests. There was also no obvious deflection or deformation of the wall members.

- Consistent behaviour observed in laboratory (Section 4.1.2) and natural fire tests was apparent. Firstly, SIPs have little inherent fire resistance and the performance of the plasterboard governs behaviour. Secondly, isolated pockets of insulation damage could be noted although both the plasterboard and OSB remained intact. The damage is due to the issuing of hot gases and steam into the insulation space, which is sufficient to melt EPS but not PUR.

- At the end of the tests, the composite action assumed in design for SIPs can no longer be relied on due to either degradation of the inner layer of OSB and melting of the core (EPS) or degradation of the OSB and combustion of the core (PUR). As there was no collapse of the buildings, it is clear that an alternative load path was mobilised at the fire limit state. Load carrying capacity was most likely maintained through the solid timber ring beams at first floor level, through the presence of intermediate timber in the panels either at junctions between panels or around openings, and because of the presence of timber studs in the corners (Figure 4.14).

- The inclusion of service penetrations, such as electrical sockets, in the PUR tests made no appreciable difference to the performance of the panel or of the structure, in the fire scenarios studied.

**Figure 4.14 Damage to SIP walls (F4-PUR30)**

## 4.2 SIMULATING TEMPERATURE DEVELOPMENT IN TIMBER SYSTEMS

Laboratory fire testing and natural fire experiments can be used both in the development and validation of numerical models. Such numerical models are increasingly being used to design buildings to withstand fire. To this end fire experiments are an invaluable resource in the development of design concepts, tools and codes, which influence the way structural fire engineers design buildings.

Effective structural fire design not only requires an understanding of structural performance, but also heat transfer behaviour. Ultimately, the temperature reached by a structural member/system in fire governs its subsequent mechanical response. In this regard, timber is no different to any other structural framing material. Complexities relating to the thermal behaviour of timber do, however, make predicting the thermal response of timber systems more challenging. Firstly, timber is hydroscopic, as such it contains and absorbs free moisture which flows and evaporates upon heating. Secondly, the thermal properties of timber appear to be dependent upon the rates of heating and cooling. Finally, many new timber systems are 'composites'. In the UK, it is standard practice to protect timber structures with plasterboard because light timber frame is the preferred method of construction. To predict the response of a timber (or timber composite) system in fire it is necessary to understand the thermo-physical behaviour of the many constituents. Typically, this includes insulation (polymer, glass and mineral based), timber board products, softwood and gypsum. Investigations concerning the heat transfer behaviour of modern timber systems have been conducted by the author. The major findings are discussed in the sections that follow.

### 4.2.1 HEAT TRANSFER CHARACTERISTICS OF GYPSUM PLASTERBOARD AND SIPS

Gypsum plasterboards are the most widely used passive fire protection for timber structures, especially in the case of light timber frame construction. Understanding the complex thermo-physical behaviour of plasterboard at elevated temperature is vital in the performance based design of any structure adopting gypsum as passive fire protection. Numerous heat transfer studies have been conducted over the years where attempts have been made to simulate the fire performance of gypsum protected assemblies, subject to standard fire exposure. However, contradictory thermal properties for gypsum plasterboard are reported. As a result, it is unclear, from a practitioner's perspective, which properties are relevant for design purposes. In recognition of this, the author has undertaken a numerical study highlighting the consequences of adopting many of the differing property sets available in the literature. The main aim was to consider the validity of adopting parameters derived from calibrations against standard fire tests when used to investigate structures exposed to natural fires.

Therefore, the sensitivity of temperature development resulting from deviations from the assumptions that underpin such properties, and the consequences of adopting plasterboard properties derived from standard fire tests were considered. A full commentary of the work undertaken can be found in Appendix C.

**Summary of findings**

The study highlighted a number of important conclusions relating to the simulation of plasterboard assemblies exposed to both standard and natural fire conditions:

1. The widespread variation in properties available in the literature for gypsum plasterboard results in significantly different predictions of temperature development in protected SIPs. This is not surprising given the variation in methods of derivation and in the products investigated.

2. Based on equations proposed by Ang and Wang (2009), new indicative plasterboard thermal properties are proposed for standard fire exposure and the modelling of gypsum lined SIPs. The new properties distinguish between Type A (standard) and Type F (fire resistant plasterboard). They have been validated, and to some extent calibrated, against heat transfer experiments conducted at BRE Global (Section 4.1.2). Numerical results based on the modified properties were shown to give good agreement with the experiments conducted.

3. In general, properties proposed in the wider literature for gypsum board may be used in the simulation of protected timber assemblies under natural fire conditions. However, these properties can only be applied up to the point at which large gaps open in the plasterboard joints and the board ultimately fails. After this phase, heat transfer into floor cavities is largely driven by buoyant gases entering the space from the fire below.

4. Many of the properties for gypsum presented in the literature, including those calibrated by the author in this thesis, appear to be dependent on heating rate. As a result, their

use may lead to inaccuracies if applied in the simulation of temperature development under natural fire conditions. Properties which are most likely to be affected by the heating rate include moisture flow, ablation and cracking.

5. If modelling of gypsum protected assemblies exposed to natural fires is to be conducted, it is recommended that behaviour such as moisture flow and ablation are considered in a more explicit manner and not through ‘effective’ or ‘apparent’ thermal properties.

6. The inclusion of modified specific heat associated with mass transfer, as proposed by Ang and Wang (2009), and the effect of a third dehydration reaction in gypsum board, proposed by Thomas (2010), results in the underestimation of temperature development in SIPs and joist floors according to the experimental measurements taken and the simulations conducted. However, their applicability in stud wall assemblies has not been investigated.

### 4.2.2 MODIFIED CONDUCTIVITY MODEL FOR SOFTWOOD EXPOSED TO PARAMETRIC FIRES

Predicting the thermal response of gypsum is one aspect required to design timber structures exposed to fire. Beyond this, timber structure temperatures also need to be determined and a structural fire design performed. Recommendations for designing timber structures, as well as other structures made of concrete or steel, under fire conditions are given in the relevant parts of the fire Eurocodes. However, unlike other parts, the scope for using the part relevant to the fire design of timber structures (Eurocode 5 part 1.2) is quite limited. This is because, with the exception of a single annex, it is only applicable to structures subjected to standard fire exposure. The only exception to this is Annex A which gives guidance on the charring rates of initially unprotected timber members in parametric fires. However, in the UK the use of this Annex is prohibited as specified in the national annex to the Eurocode.

**Background to the modified conductivity model (MCM)**

The complex phenomena present in heated timber elements are difficult to model explicitly and, hence to date, ‘effective properties’ are often defined. This is similar to the approach taken for plasterboard, discussed previously in Section 4.2.1. Such properties implicitly account for the effects of complex behaviour, such as the flow of pyrolosis gases and water vapour, through calibration against known temperatures, in limited experimental configurations. König and Walleij (2000) have been instrumental in initiating such a process for timber. They calibrated ‘effective’ thermal properties for standard fire exposure conditions. These properties form the basis of the advanced calculation models contained in Annex B of EN 1995-1-2. However, additional studies by König (2006) proved (both experimentally and numerically) that these properties exhibited very conservative predictions of char depth when applied to non-standard (parametric) fire conditions, with heating rates in excess of those given by the standard fire curve. Similarly, the properties from the code were shown to result in non-conservative predictions of timber temperature and depth of char for heating rates lower than that of the standard-fire curve. As a result, EN 1995-1-2 explicitly states that the thermal properties present in Annex B should only be adopted for standard fire exposure and not for parametric fire exposure.

König (2006) previously proposed that consistency between parametric charring measurements in experiments and computational predictions, under standard-fire exposure, could be achieved via subtle modifications to the conductivity-temperature relationships proposed in Annex B of EN 1995-1-2. In addition, he noted that only those properties in excess of 350°C should be modified as they represent the ‘effective’ properties of the char layer. Phenomena in the char area, such as ‘reverse cooling pyrolosis flows’, cracking and ablation, appear to be influenced by heating rate. Although König (2006) made the observation that the thermal properties present in Annex B of EC5-1-2 were not appropriate for parametric fire applications and that better agreement could be seen through adaptation of the char layer conductivity, no follow on research has been conducted to quantify the necessary modification of the char layer conductivity.

In a paper written by the author as part of this EngD (refer to Appendix D), a modified conductivity model (MCM) for softwood timber based upon the principles outlined in König's research and upon EN 1995-1-2 specific heat modifications proposed by Cachim and Franssen (2009) is presented. The MCM was derived using numerical calibrations of a fire load ($q_{td}$) and heating rate ($\Gamma$) dependent modification factor ($k_{\lambda,mod}$) and the depths of char present in parametric design fires. In the latter case, the depth of char in such fires was determined using the Annex A approach of EN 1995-1-2.

The full derivation of the proposed model can be found in Appendix D. However, the resulting relations are shown in Table 4.3, Table 4.4 and Equation 4.1.

**Table 4.3 Summary of modified conductivity model**

| **Temperature** (°C) | **Conductivity** (W/m.K) |
|---|---|
| 20 | 0.12 |
| 200 | 0.15 |
| 350 | 0.07 |
| 500 | $0.09k_{\lambda,mod}$ |
| 800 | $0.35k_{\lambda,mod}$ |
| 1200 | $1.50k_{\lambda,mod}$ |

**Table 4.4 Specific heat after Cachim and Franssen (2009)**

| Temperature (°C) | Density ratio G | Moisture modified specific heat (J/kg.K) |
|---|---|---|
| 20 | 1+ω | (1210+4190ω) / G |
| 99 | 1+ω | (1480+4190ω) / G |
| 99 | 1+ω | (1480+114600ω) / G |
| 120 | 1 | (2120+95500 ω) / G |
| 120 | 1 | 2120 / G |
| 200 | 1 | 2000 / G |

$$k_{l,\mathrm{mod}} = k_{\Gamma,\mathrm{mod}} k_{qtd,\mathrm{mod}}$$ **[Equation 4.1]**

$$With \qquad k_{\Gamma,\mathrm{mod}} = 1.5\Gamma^{-0.48} \;,\; k_{qtd,\mathrm{mod}} = \sqrt{\frac{q_{td}}{210}} \;\; and \;\; \Gamma = \frac{(O/b)^2}{(0.04/1160)^2},$$

*Where:*

- *$\omega$ is the moisture content of timber (%)*
- *$O$ is an opening factor ($m^{0.5}$)*
- *$b$ is compartment thermal inertia ($J/m^2s^{0.5}K$)*

The above, when coupled with specific heat properties and appropriate densities, were found to give consistent transient depth of char predictions for the heating phase of a parametric fire, when compared to EN 1995-1-2 Annex A. However, from a structural engineering viewpoint, the definition of the depth of char in FEA simulations is insufficient to fully characterise the mechanical response of a member exposed to high temperatures. In timber, only those temperatures below 300°C are of concern. Above this threshold, the timber is charred and friable. As a result, the MCM must be able to place the char line correctly within a cross section, and accurately simulate temperature in the intact member. This allows the sectional response to be determined using strength, stiffness and temperature relations. Given the limited number (and limitations) of experiments conducted on timber exposed to parametric fires, the author was only able to investigate temperature development using the test data developed by König and Walleij (1999). The modelling conducted and the comparisons made are discussed in the section that follows.

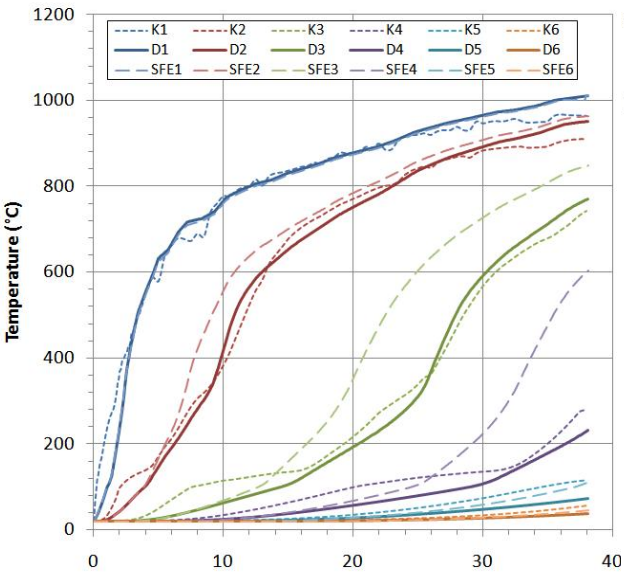
**Benchmarking against König and Walleij (1999) test data**

At the turn of the century König and Walleij (1999) reported six experiments on timber blocks exposed to parametric fires. These were labelled C1 to C6. The experiments exposed timber panels to one dimensional heat transfer via a gas powered furnace following parametric curves. From this it was first observed by König (2006) that the thermal properties present in Annex B of EN 1995-1-2 were inappropriate for use with non-standard fire exposure. The timber used in their experiments was a generic softwood with an estimated moisture content of 12% and a mean density of 420 to 430 kg/m$^3$. Although König and Walleij (1999) attempted to follow parametric curves, this was not entirely possible due to the furnace configuration. To model the experiments conducted by König and Walleij (1999) using the author's MCM (Appendix D) it is necessary to determine the parameters $q_{td}$ and $\Gamma$. Using gas temperatures measured by König and Walleij (1999), the author has fitted EN 1991-1-2 parametric curves to the measured gas temperature-time relationships via trial and error. The resulting key parameters are noted in Table 4.5. From observation of the test data it is apparent that experiments C1 to C3 follow the standard fire curve and then cool. However, experiments C4 to C6 follow a different accelerated heating regime. Given this, the author decided to attempt to simulate experiments C4 to C6 as they represent an obvious deviation from the standard fire curve. In addition, as the author's MCM was developed for the heating phase of parametric fires, only the heating phase of König and Walleij's (1999) experiments is considered at present.

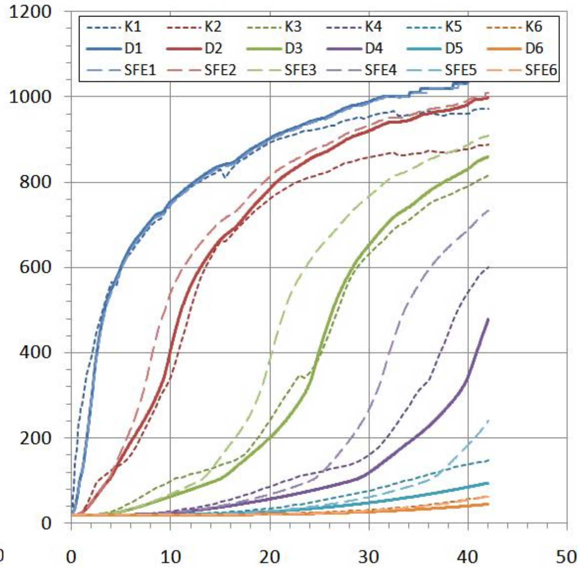
**Table 4.5 Fitted parametric curve parameters**

| | **C4** | **C5** | **C6** |
|---|---|---|---|
| $q_{td}$ (MJ/m$^2$) | 93.8 | 109.4 | 114.6 |
| Γ (-) | 2.7 | 3 | 4.5 |

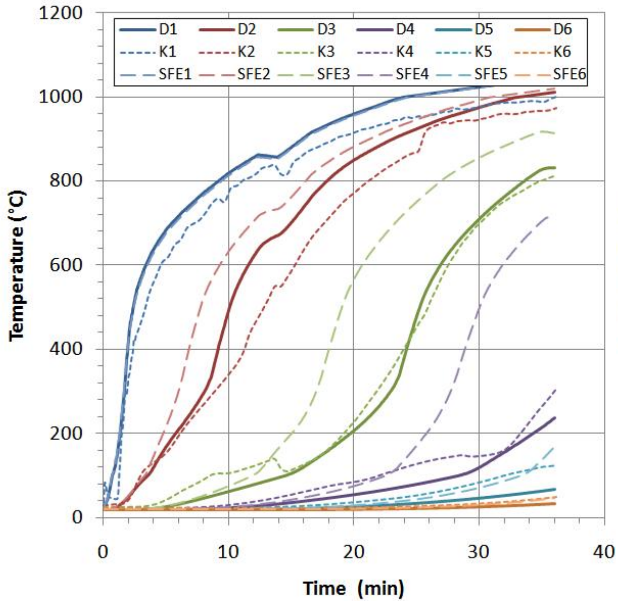
Using these parameters it is possible to determine the appropriate values of $k_{\lambda,mod}$ for each test (Equation 4.1) thus yielding modified conductivity properties. As the moisture content was estimated as 12% by König and Walleij (1999), the specific heat relationship from EN 1995-1-2 can be adopted without modification. Using the gas temperature measured in each experiment as a boundary condition, coupled with boundary coefficients of $\varepsilon = 0.7$ and $\alpha = 35$ W/m$^2$ K, the one dimensional heat flow was simulated using TNO DIANA (Manie 2010). In all instances first order quad elements, with dimensions of 0.5 mm, were adopted. Temperatures at depths of 0, 6, 18, 30, 42 and 54 mm, denoted 1 to 6 respectively, were measured by König and Walleij (1999). Therefore, temperatures for corresponding nodes are called from DIANA (Manie 2010). In addition, simulations were conducted with unmodified conductivity properties as per EN 1995-1-2 for comparison. Plots of the resulting temperature development are shown in Figure 4.15(a-c). Resulting depth of char, taken as the position of the 300°C isotherm, is also plotted against test data in Figure 4.15d.



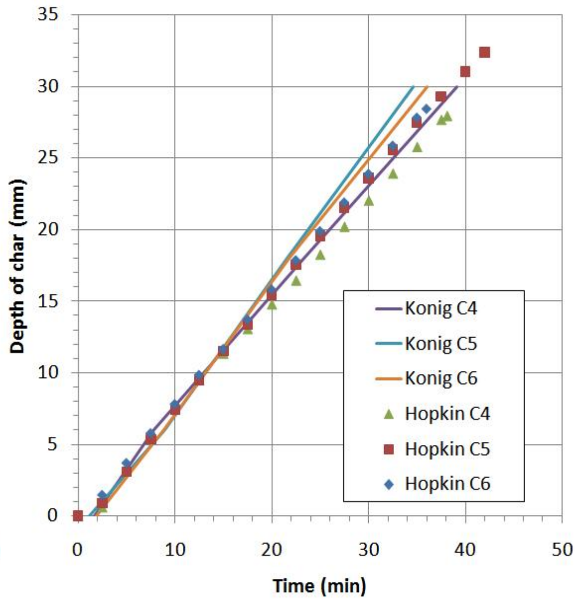

(a)




(b)




(c)




(d)

**Figure 4.15 (a-c) Temperature development, comparison of MCM (D), König (K) and EN 1995-1-2 (SFE) simulations/experiments; (d) Transient depth of char- simulation (Hopkin) versus experiment (König and Walleij 1999)**

It is apparent from the limited temperature validation conducted, that the MCM proposed by the author for softwood results in a vastly improved prediction of temperature development in timber members exposed to the heating phase of parametric fires (compared to EN 1995-1-2 Annex B properties). However, the experiments of König and Walleij (1999) are not well defined, and, as a result, stronger conclusions cannot be drawn without further benchmarking against more robust experiments.

**Extensions to cooling timber**

The modified conductivity model was derived by numerically calibrating timber char conductivity to heating rate by positioning the 300°C isotherm (or char line) so that the method yielded the same charring depth as set out in Annex A of EN 1995-1-2. More information on this process can be found in Appendix D. This calibration was conducted so that the depth of char after a period of $t_0$ minutes was consistent when calculated using both FEA simulations and Annex A. The period $t_0$ is defined as the 'constant charring phase'. It describes a linear relationship between depth of char and time. During this period, char of a thickness $\beta t_0$ develops. However, after this period and during cooling, according to EN 1995-1-2, a further char layer with thickness $\beta t_0$ develops, giving a total depth of char of $2\beta t_0$. In this instance the term $\beta$ is the parametric charring rate in mm/min. Given that the MCM was developed for the heating phase of parametric fires (i.e. up to $t_0$), its applicability in the cooling phase of fire development is uncertain. To verify its applicability further benchmarking was conducted against Annex A of EN 1995-1-2 by performing simulations with the proposed conductivity changes and a fully defined parametric fire (inclusive of cooling). An example prediction of charring depth is shown in Figure 4.16. The graph shows transient depth of char development determined using Annex A of EN 1995-1-2 and the developed MCM. In the latter case the 300°C isotherm has been taken as the notional char line. In this instance a $t_0$ value of 30 minutes has been adopted corresponding to $\Gamma$ = 12.15 and $q_{td}$ = 210 MJ/m$^2$.

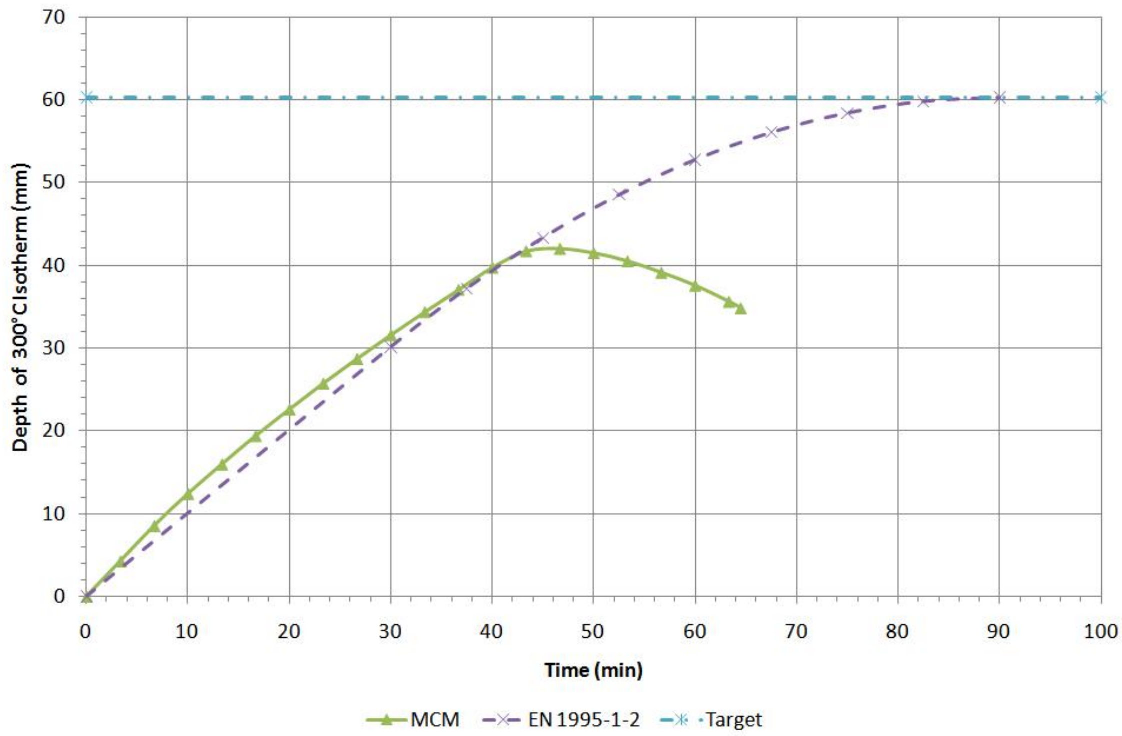



**Figure 4.16 Comparison of FEA, MCM and EN 1995-1-2 calculation of depth of char**

Since charring is a dominant phenomenon, and transient effects and thermal expansion of timber appear to have little bearing on behaviour in fires, it becomes less important to accurately simulate temperature and char development as a function of time. By definition, performance-based design is a process whereby a structure is designed to survive the entire duration of a fire, and, in crude terms, the resulting building has infinite fire resistance. It follows, that to design a timber member for such an event, it is only necessary to determine the maximum depth of char (at the end of cooling) and the maximum temperature apparent in any undamaged residual timber. This process is semi-independent of time.

Further numerical calibrations performed by the author show that, via a slight modification to the fire load dependent term ($k_{qtd,mod}$) in the MCM, the total depth of char can be determined accurately using FEA simulation. The calculated char depth is inclusive of the additional char that develops during cooling. The modified term is given by Equation 4.2.

$$k_{qtd,\text{mod}} = \sqrt{\frac{4.q_{td}}{210}} \,. \qquad \textbf{[Equation 4.2]}$$

This simple modification yields the following relationships between depth of char and time for different parametric fire exposures, see Figure 4.17. In all instances $q_{td} = 210$ MJ/m$^2$:

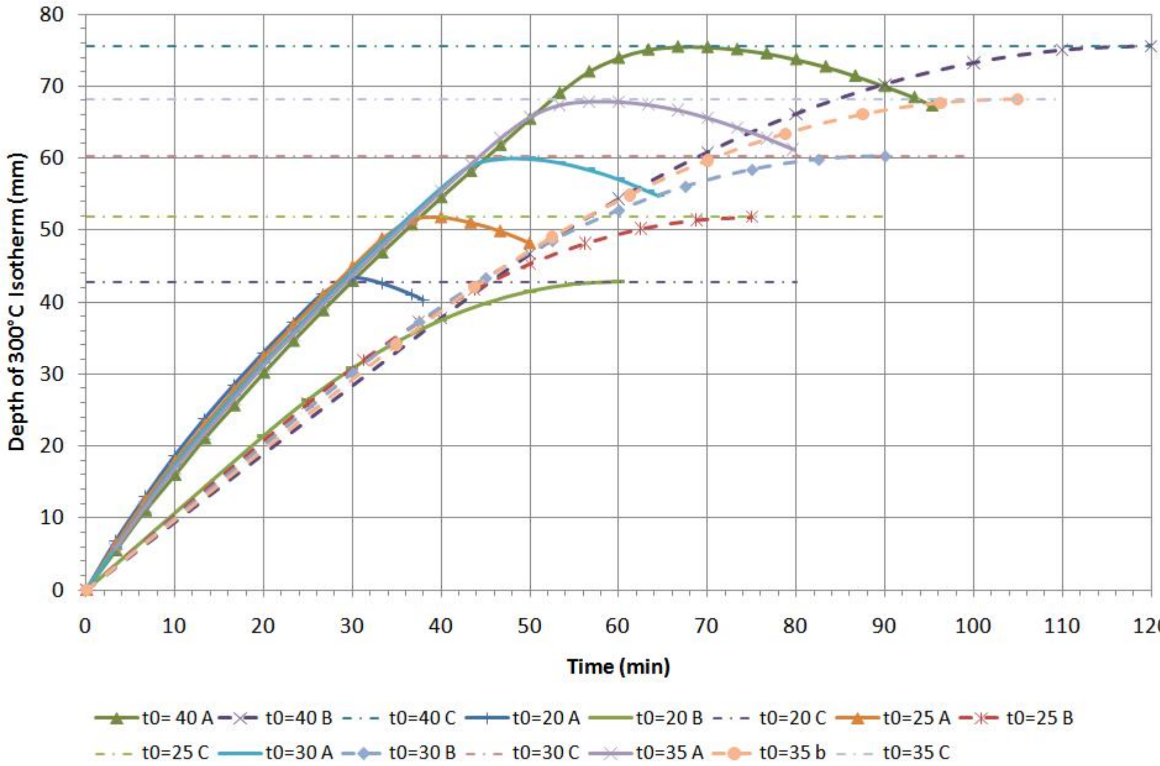



**Figure 4.17 Position of 300°C isotherm using modified $k_{qtd,mod}$ (A) and EN 1995-1-2 Annex A (B). Target depth of char shown as (C).**

Figure 4.17 shows that in all instances the maximum depth of char determined via simulation is consistent with the Eurocode approach. As a result, although in transient terms the depth of char is inconsistent, the residual cross section determined in both cases at the end of the fire is identical. From a scientific viewpoint the proposed method does not accurately simulate the physical complexities that occur in timber on cooling. However, this is also the case for the many empirical methods contained in EN 1995-1-2. To gauge the applicability of such an approach, determining the charring depths alone is not sufficient. It must also be shown that ultimate temperature development in uncharred timber is compatible with that apparent in reality. To verify this, further benchmarking should be conducted against the test data of König and Walleij (1999) or other available experimental data.

**Summary of findings**

A modified conductivity model for timber has been derived. The model is based on numerical calibrations between parametric depth of char and char layer conductivity. The full derivation of the model is outlined in a supporting paper in Appendix D. It was found that, with modified conductivity properties, the depth of char in a section (or position of the 300°C isotherm) can be located with relative accuracy during the heating phase of a parametric fire. In addition, through benchmarking against experimental data provided by SP Trätek, it has been found that the proposed conductivity modifications also result in vastly improved predictions of temperature development in timber members exposed to non-standard fires.

Further benchmarking of depth of char predictions using the modified properties and that of the empirical charring method of Annex A indicates that the proposed adaptations still do not adequately simulate char formation and temperature development in the cooling phase of a parametric fire. This is likely to be due to char oxidation, which results in additional 'fire load', thus increasing the temperature of a timber member beyond that of the cooling surrounding gas temperature. Such a conclusion was supported by the findings of König and Walleij (1999).

The simulation of temperature development in cooling timber members is a complex and difficult task. König (2006) suggests that different thermal properties should be adopted in the heating and cooling phases of fire development. From a design perspective, this is an awkward approach. As a result, the author has proposed a pragmatic engineered solution which, in theory but subject to further verification, should allow the use of computational techniques for the design of timber buildings exposed to parametric fires. This approach should be further investigated and additional benchmarking conducted against any available test data.

## 4.3 ENABLING DESIGN THROUGH COMPUTATIONAL MODELLING

Computational modelling has become an invaluable tool for academics and consultants alike in the research and design of structures for fire. However, the modelling of fire exposed timber structures is still not widely undertaken for design purposes due to a number of complexities in material behaviour and also the limitations of EN 1995-1-2. The complexities relating to timber and associated material behaviour at elevated temperature has been introduced in Section 1.3.

The outstanding technical barrier to the ‘performance based design’ of timber structures is the absence of thermal properties for non-standard fires. Developments made by the author in recognition of this have been discussed in Section 4.2.2. The next step is to couple both the thermal and mechanical aspects of fire performance.

The modelling of fire exposed timber structures using generic FEA codes like ABAQUS and DIANA is not a simple task. As a result, some adaptation is required in recognition of the complex mechanical behaviours that exist at elevated temperature. The development of numerical tools capable of predicting timber response in non-standard fires is discussed in the sections that follow. Predictably, a number of additional complexities have been encountered en route.

### 4.3.1 IMPLEMENTATION OF THE MCM IN COUPLED THERMO-MECHANICAL UNI-AXIAL CASES

The developed MCM (Section 4.2.2) is intended to be adopted as a performance based design tool used to determine thermal response in non-standard fires for subsequent mechanical analyses (thermo-mechanical analyses). The simplest example of thermo-mechanical behaviour of a structural element at elevated temperature is that of a uni-axially loaded member such as a strut or tie, subject to simple heating (e.g. 1D). This case has been used as the starting point for simulations tasked with predicting failure times of uni-axial timber members in natural fires. Verification of the MCM's ability to predict temperatures and subsequently failure times, as part of a coupled thermo-mechanical analysis, subject to non standard fires, has been performed via a number of numerical simulations. The results are benchmarked against EN 1995-1-2's reduced cross section method. This element of research is summarised below for completeness but is discussed in further detail in a paper found in Appendix E.

**Summary**

The MCM has been adopted in coupled thermo-mechanical analyses of plane strain uni-axial timber members subject to one dimensional heat transfer. The timber members are designed to fail at different times using the reduced cross section method contained in EN 1995-1-2. Comparisons of (FEA) simulated and calculated (EN 1995-1-2) failure times have been conducted for short tension and compression members subject to either standard or parametric fire exposure. The MCM, when adopted in coupled thermo-mechanical analyses, is shown to yield comparable 'failure times' as the EN 1995-1-2 reduced cross section method. For parametric fires the 'reduced cross section' was determined using Annex A charring rates. The research identifies that current provision for the 'zero strength' layer in EN 1995-1-2, when adopted for tension members, may be un-conservative under both non-standard and standard fires. More information can be found in Appendix E.

### 4.3.2 ADAPTATION OF TNO DIANA FOR MODELLING TIMBER AT HIGH TEMPERATURE

The adoption of generic FEA codes to simulate uni-axial behaviour is fairly simple as only a tensile or compressive stress state exists at any one time. As such it is possible to simply define unique relationships between temperature, strength and stiffness. This was the approach adopted in the investigation summarised in Section 4.3.1. The use of more general finite element packages, such as DIANA and ABAQUS, for modelling full timber structures exposed to fire has yet to become common due to a number of complexities relating to the behaviour of timber. For example, it is brittle and fractures in tension, while being more ductile and plastic when subject to compression.

In addition, with increasing temperature, the degradation of timber's constitutive behaviour in tension is different from that in compression. As a result, timber's Modulus of Elasticity (MOE) depends on its state of stress. Therefore, a single MOE-temperature relationship cannot be defined, unlike in the uni-axial cases mentioned previously in Section 4.3.1.

Finally, upon heating timber undergoes a phase change whereby wood becomes friable char. Upon cooling, char still has little or no strength or stiffness. Therefore, it is not appropriate to specify timber strength on the basis of temperature alone when cooling is to be considered. Knowledge of the full temperature history is required. Most commercial FE programs do not incorporate many of the above characteristics in a direct way. Therefore, it is necessary to adapt such codes to accommodate these behaviours. An approach for doing this is described in the remainder of this section. Implementation of the approach in the FEA software TNO DIANA (Manie 2010) via FORTRAN user-supplied subroutines (USS) is also described.

**USRYOU - A subroutine for determining Modulus of Elasticity (MOE) of timber**

DIANA offers a number of subroutine options for customising the analyses performed. One such subroutine is the USRYOU option. This allows users to return MOE based upon a number of inputs, including integration point strain and temperature. The author has developed a USRYOU USS for determining the MOE of timber exposed to both heating and cooling regimes.

Firstly, integration point and element numbers are called from the program along with temperature at the given integration point. Temperature history of elements is recorded via a common block, which determines whether the temperature of the timber (a) exceeds the charring temperature of 300°C, (b) exceeds the moisture evaporation temperature of 100°C, or (c) is below 100°C, i.e. timber neither charred nor suffered moisture evaporation. Based on this, the temperature history common block is updated incrementally, which allows state history to be recorded via a state variable (SV). The latter allows for a number of hypotheses to be investigated, which will be discussed below.

Using the recorded temperature history the stress state may be investigated. The strains in the local element coordinates are called from DIANA. The dominant integration point strain is determined, which is then used to evaluate MOE appropriate for the temperature and the strain state. For example, if $\varepsilon_{xx}$ is found to be the largest element strain and the strain is negative (compressive), the MOE is returned, based upon EN 1995-1-2 (BSI 2004c) compression (-ve) reduction factors ($K_{EC,mod,\theta,i}$) and element temperature. The converse case would be adopted if $\varepsilon_{xx}$ was found to be positive (+ve). In this instance reduction factors are defined as $K_{ET,mod,\theta,i}$. This process is shown diagrammatically in Figure 4.18.

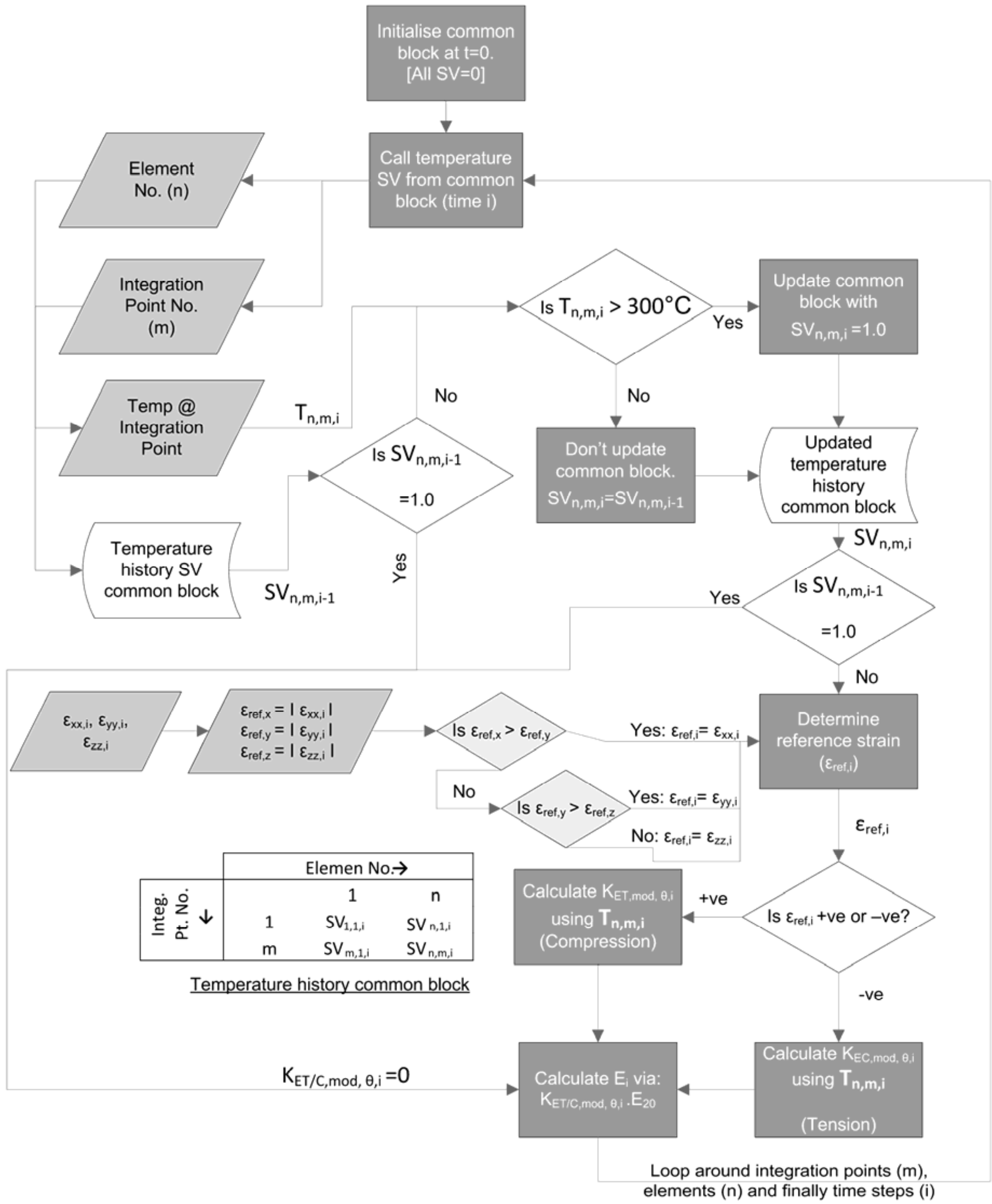



**Figure 4.18** **Flow chart for USRYOU routine**

**USRCST/TST - A subroutine for determining compressive/ tensile strength of timber**

Two more USSs, namely USRCST and USRTST, are available in DIANA for determining the tensile and compressive strength, respectively. These routines in particular are for implementation with Total Strain Based constitutive models, which are discussed in a later section (Section 4.3.3). The routines utilise the temperature history common block initialised in the above USRYOU routine to calculate tensile and compressive strength using EN 1995-1-2 reduction factors. The element number and integration point number are used to reference allocated memory slots, where temperature state variables are stored. Compressive and tensile strengths (limiting stresses) are passed to DIANA for implementation in the adopted Total Strain Based Constitutive model. This process is shown in a flow diagram in Figure 4.19.

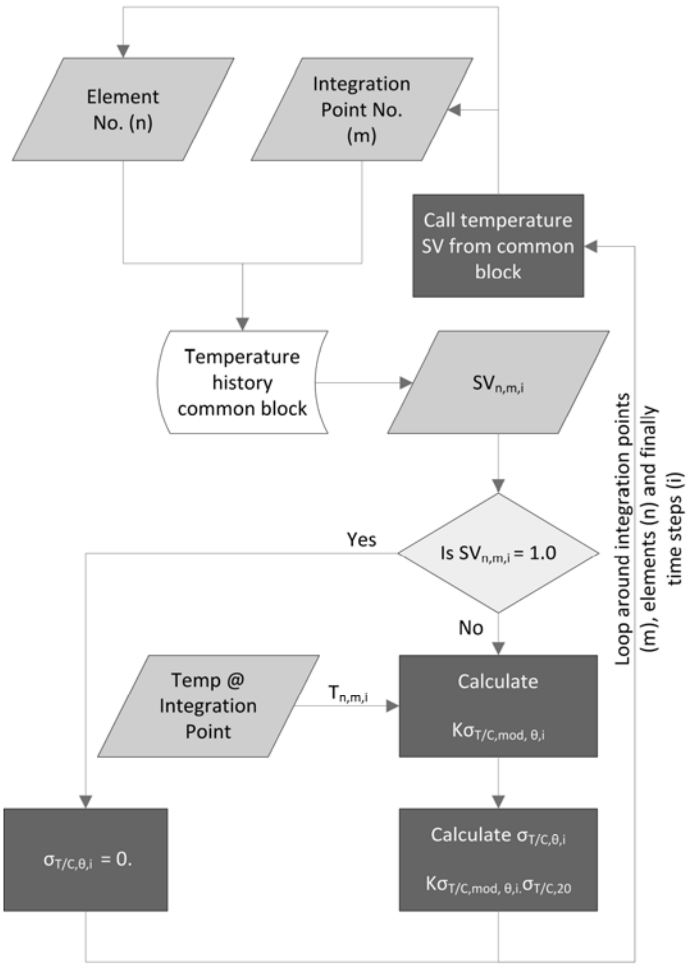



**Figure 4.19** **Flow chart for USRCST/TST routines**

**The use of temperature state variables in the cooling phase**

When the temperature reaches a certain limit within a timber section its moisture content will be completely lost through evaporation. As the section's temperature increases, charring of certain parts may also occur. The charred timber will not contribute to the strength and stiffness of the section. Clearly, this damage is irreversible. However, little is known about the strength and stiffness of the remaining un-charred part of the section and how the section behaves during cooling down. To study this, the concept of temperature history state variables (SVs) has been introduced. This allows for a number of hypotheses to be investigated.

The first hypothesis is based on the assumption that, during cooling down, undamaged timber recovers none of its strength or stiffness. This would be the case if temperature changes during the cooling phase are ignored. In this case, the maximum temperature reached during heating up governs the behaviour during cooling.

In the second hypothesis it could be argued that moisture lost during the heating phase cannot be regained during cooling down. This means that strength and stiffness of timber whose temperature, upon heating, did not exceed 100°C will be fully recovered to that appropriate to its temperature during cooling down. However, timber heated beyond 100°C, but not charred, may recover its strength and stiffness but only up to the maximum value applicable to dry moisture-free timber. The latter condition implies that reduction factors corresponding to 100°C should remain applicable even when timber temperature drops below this value.

A third and final hypothesis is that, while cooling down, all non-charred timber will recover the entire strength and stiffness appropriate to its temperature. This implies that loss of moisture during heating has had no effect on the recovered properties. Clearly, this approach may be implausible as it suggests the return of moisture into the timber during cooling.

Although timber is a hydroscopic material the return of free moisture would take time much in excess of the decay phase of most fires. The implications for these three hypotheses can be observed through a number of simple numerical tests on single quad elements or simply supported beams. This testing process is discussed in the following section.

**Subroutine testing and implementation**

The testing of the USSs was conducted at a number of different scales. Firstly, trials of the three hypotheses were conducted using single first-order quad elements uniformly heated and cooled down. These were subject either to a compressive or tensile strain. In these trials, only the USRYOU subroutine is implemented so that non-linear elastic solutions can be sought without either cracking or plasticity. Resulting strain-temperature plots are shown in Figure 4.20, for all three hypotheses. A constant load was applied throughout. In such trials it is not possible to indicate permanent charring damage as this would result in numerical instability. Thus, the maximum applied temperature was 210°C. Tensile and compressive loads of identical magnitude were applied to allow for the difference in MOE degradation with temperature for different strain states to be checked. However, only one set of results (compressive) is shown as the other indicated the same pattern.

The second element of USS testing is concerned with the behaviour of simply supported beams subject to a temperature gradient. Such a beam was modelled in DIANA using a number of first-order 2D beam elements. Temperatures were specified at 11 integration points through the cross section of beam elements. Integration point distribution was according to a Simpson integration scheme. The adopted temperature profiles are shown in Figure 4.21. The legend indicates fire from below with 11 integration points numbered from the top down. The applied temperatures are fictitious ones and serve only to demonstrate implementation of the USS. The modelled beam is 4 m in length and has a 100 mm x 250 mm cross-section. The beam is subject to a nominal load of 5 kN/m. The development of deflection upon heating, followed by cooling can be seen in Figure 4.22 In this case, the beam temperature developed beyond 300°C. Therefore, permanent deformation due to charring was apparent, including the case with full un-charred timber strength recovery (hypothesis 3).

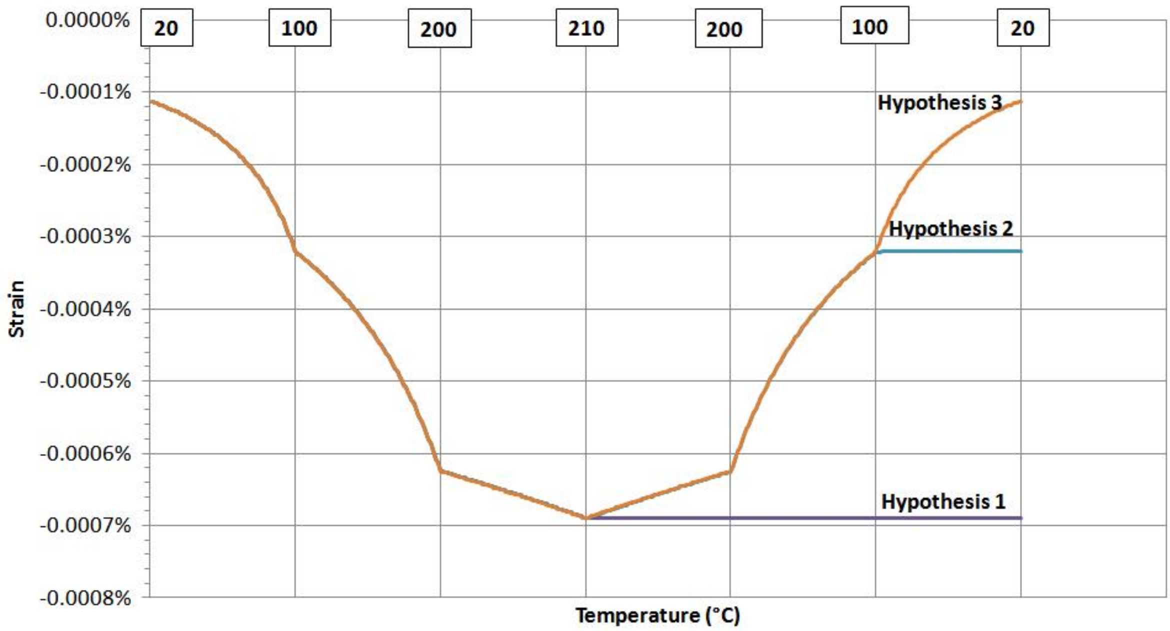



**Figure 4.20 Single-element implementation of USRYOU subroutine: temperature-strain plots for a constant nominal load**

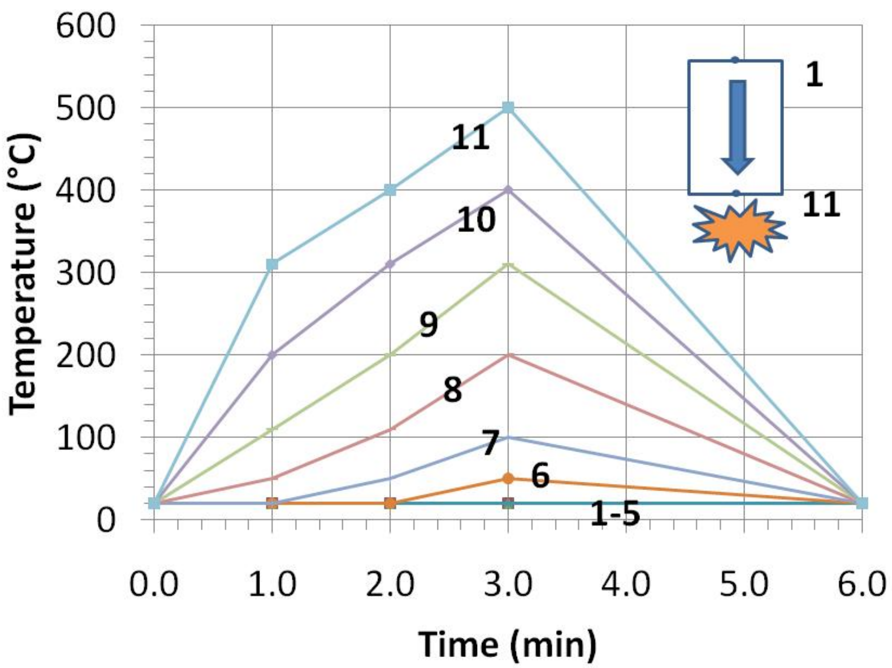



**Figure 4.21 Beam implementation of USRYOU and USRCST/TST subroutines: Temperature profiles at integration points**

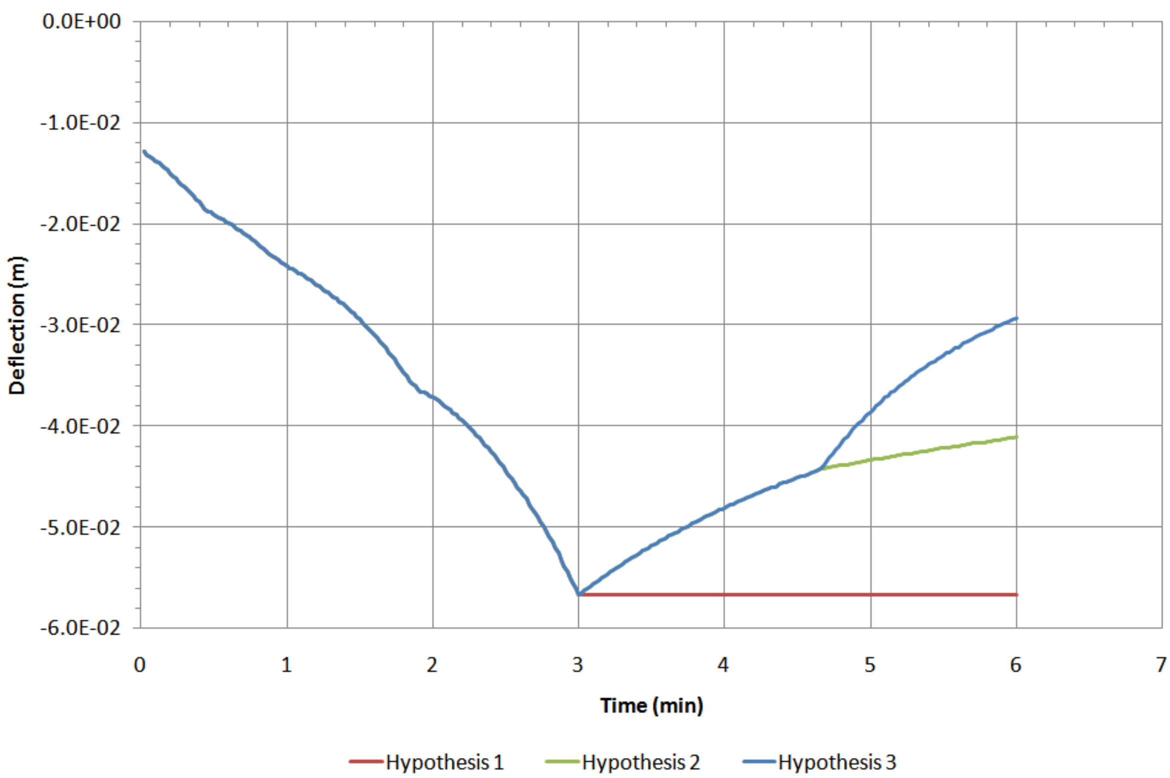



**Figure 4.22 Beam implementation of USRYOU and USRCST/TST subroutines: Deflection–time plots**

**Discussion**

A number of relatively simple modifications to a commercial finite element code have been developed. The modifications allow for timber in fire to be modelled in potentially much more complex scenarios than previously considered. Until recently, the thermo-mechanical modelling of timber using commercial codes has not been widely undertaken. Where attempts have been made it has been done with simple ad-hoc codes, having limited fields of application, or simple uni-axial cases (Fragiacomo, *et al.* 2010b). If more advanced simulations of timber are to be conducted then either specialist codes need to be developed or commercial codes adapted.

The adaptation of a commercial code, such as DIANA, by means of FORTRAN user-supplied subroutines is desirable for a number of reasons. Firstly, with simple modifications to the stress relations, outlined above in the USSs presented, any number of element variations, from beam, through shell, to block elements, can be considered depending upon the problem encountered. Secondly, the powerful robust solvers, which are heavily tested in commercial codes, can be adopted with only the aspects of material behaviour which need to be appropriately represented by the USS, such as MOE, tensile and compressive strength with increasing (and decreasing) temperature.

In relation to the strength and stiffness recovery of timber upon cooling, experimental evidence suggests that the first two hypotheses may be more realistic (refer to Appendix A and B). In the many experiments conducted by BRE on timber structures over the last decade, including those presented in this thesis, there appears to be little evidence to suggest any strength or stiffness recovery in timber structures exposed to fire, upon cooling.

### 4.3.3 THE IMPACT OF TIMBER FRACTURE ENERGY AT ELEVATED TEMPERATURE

The developments outlined in Section 4.3.2 describe only small aspects of the constitutive behaviour of timber at elevated temperature. Parameters such as MOE, tensile and compressive strength alone are often insufficient to characterise a material's behaviour. The USSs developed can however be coupled with more complex aspects of behaviour such as fracture mechanics, to more completely describe the constitutive behaviour of timber in fire.

The developed user routine USRTST describes the tensile strength of timber only and not the post fracture softening branch. To date, timber in fire researchers have assumed perfectly brittle behaviour for wood in tension (Bazan 1980, König and Walleij 2000, Buchanan 1990/2001 and Fragiacomo, *et al.* 2010b). This implies instantaneous dissipation of all strain energy upon cracking. However, this is an important conservative assumption, which is not desirable in numerical modelling as it causes numerical instability. The sudden and complete dissipation of strain energy at a single integration point in an FEA model is often sufficient to cause non-convergence. To this end the tension softening behaviour of wood in fire is an important consideration as behaviour at elevated temperature is already highly non-linear, without further instability arising as a result of poorly defined crack constitutive relations.

The fracture energy of timber ($G_f$), more specifically softwoods, is an area well researched at ambient temperature. Many textbooks give fracture energies for different cracking modes, which are shown to be highly dependent upon density (Thelandersson and Larsen 2003). Larson and Gustafsson (1990/91) give one such correlation for notched timber members subject to bending, where:

$$G_f = 1.07\,r - 162 \qquad \textbf{[Equation 4.3]}$$

Where $G_f$ *is* fracture energy (Nm/m$^2$) and $\rho$ *is* density in kg/m$^3$.

For typical softwood this gives fracture energies ranging from 160 to 480 $Nm/m^2$ for mixed mode cracking. However, little, if anything, is known about the fracture energy of timber or other brittle materials at elevated temperature.

The numerical mechanical modelling of timber at elevated temperature is rare. To date, simplified models are often adopted using spreadsheets and sectional analysis tools (König and Walleij 2000, Schmid, *et al.* 2010). As a result of such an approach, it is not necessary to define fracture energy as timber in tension can be treated as a perfectly brittle material. As the models are so simple in their interpretation of the structural mechanics, numerical instability does not arise as a result of this assumption.  However, such an approach cannot be adopted in more general FEA computations as this may lead to numerical instability. To this end, tension softening regimes are often defined to describe the post fracture aspect of a materials constitutive relation (See Figure 4.23). The definition of such behaviour requires either knowledge of fracture energy (i.e. the integral of the deformation stress curve) or ultimate crack strain, i.e. the strain at which all crack stress has vanished. Neither codes and standards, nor academic research give an insight into the effect of increasing temperature on fracture energy or ultimate crack strain of timber. As a result, it is often necessary to assume values, which can have a very large influence upon deformation behaviour and upon the ultimate load carrying capacity derived using numerical techniques, such as FEA.

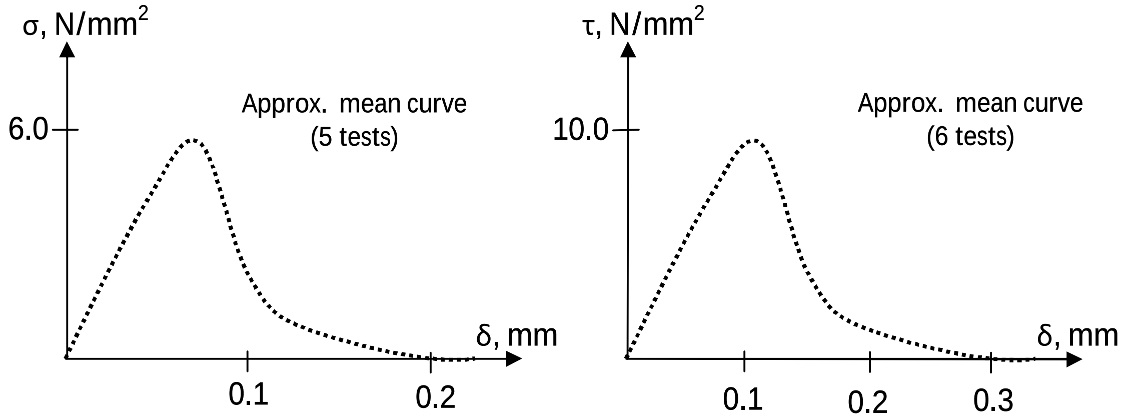



**Figure 4.23 Indicative fracture behaviour of timber in (a) tension and (b) shear (displacement vs. stress) after Thelanderson and Larsen (2003)**

In the DIANA FEA package it is possible to define a number of tension-softening relationships based upon fracture energy, crack bandwidth and/or ultimate crack strain. To investigate the impact of these parameters, a parametric study was designed to study the behaviour of simply supported beams, loaded to different utilisation levels, under standard fire exposure. To undertake the study, the developments outlined in Section 4.3.2 have been adopted. The concept of total strain-based cracking as implemented in DIANA is introduced briefly in the following section. The design of the parametric study is discussed in further detail in a later section.

### Total strain-based cracking

The DIANA constitutive model based on total strain is developed according to the Modified Compression Field Theory, originally proposed by Vecchio and Collins (1986). The three-dimensional extension to this theory is published by Selby and Vecchio (1993). A constitutive model based on total strain describes the stress as a function of the total strain. This concept is known as hypo-elasticity, when the loading and unloading behaviour is along the same stress-strain path. In the current implementation in DIANA, the behaviour in loading and unloading is modelled differently with secant unloading (Manie 2010).

The most commonly used approach is the coaxial stress-strain concept, in which the stress-strain relationships are evaluated in the principal directions of the strain vector. This approach, referred to as the 'Rotating crack model', has been applied to the constitutive modelling of reinforced concrete for a long period (Manie 2010).

The rotating crack concept is not an ideal representation of how cracks form in real materials. A more accepted approach is that of the fixed stress-strain concept, in which the stress-strain relationships are evaluated in a fixed coordinate system that is frozen upon cracking. Both methods are easily described in the same framework, where the crack directions (nst) are either fixed or continuously rotating with the principal directions of the strain vector (Manie 2010).

The basic concept of total strain crack models is that the stress is evaluated in the directions given by those of the crack. The strain vector $\varepsilon_{xyz}$ in the element coordinate system (xyz) is updated with the strain increment $\Delta\varepsilon_{xyz}$ according to Equation 4.4.

$$(t + \Delta t_{i+1})e_{XYZ} = te_{XYZ} + (t + \Delta t_{i+1})\Delta e_{XYZ}$$ **[Equation 4.4]**

This is transformed to the strain vector in the crack directions with the strain transformation matrix Z giving:

$$t + \Delta t_{i+1} e_{nst} = Z(t + \Delta t_{i+1} \Delta e_{XYZ})$$ **[Equation 4.5]**

The strain transformation matrix Z is either fixed upon first cracking or depends on the current strain vector (rotating crack model). The total strain crack models, be it the fixed or rotating crack model, are appealing as they are numerically very stable when compared to smeared strain decomposed alternatives. In such cases the total strain is decomposed into elastic and crack components, i.e.:

$$e = e_E + e_{cr} \qquad \textbf{[Equation 4.6]}$$

This decomposition of the strain allows for combining the decomposed crack model with, for instance, plastic behaviour, in a transparent manner (Manie 2010). The sub-decomposition of the crack strain $\varepsilon_{cr}$ gives the possibility of modelling a number of cracks that simultaneously occur. However, simplistically, in a total strain based formulation, the compressive (ductile) and tensile (brittle) characteristics can be idealised within a single material model describing both aspects of physical behaviour. The total strain based concept is not new and has been adopted in concrete structures for some time. The constitutive equations given for concrete in EN 1992-1-2 (BSI 2004a) are total strain based models where transient behaviour such as load induced thermal strain is included implicitly in the stress-strain curve.

**Parametric study design and modelling approach**

In DIANA, the tension softening relations of a material can be described either via fracture energy or ultimate crack strain. Both of these parameters can be specified as a function of temperature. DIANA offers linear, exponential or Hordyk tension-softening regimes, which describe the stress-strain relations of an open crack (see Figure 4.24). More information on the tension softening regimes can be found in Manie (2010).

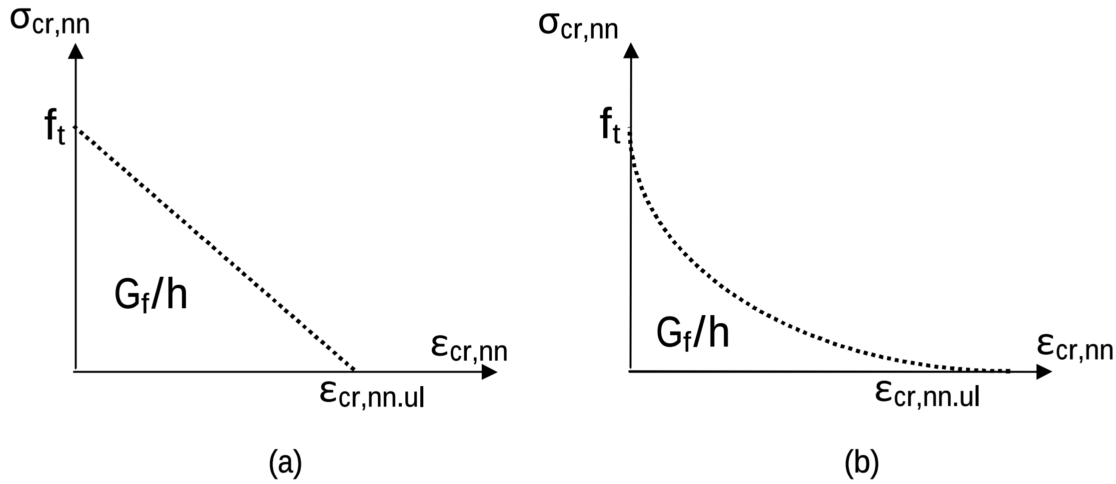



**Figure 4.24 Tension-softening relationships available in TNO DIANA (Manie 2010): Linear (a) and Hordyk (b)**

In the parametric study conducted, both of the regimes in Figure 4.24 were adopted to investigate the apparent failure time of a simple timber beam exposed to fire (ISO 834) from below and subject to varying degrees of load level (via a mid-span point load). A simple bi-linear model describes the plasticity behaviour of timber in compression as part of a total strain-based crack model incorporating the above. The beam was modelled as continuum using second-order quad plane-stress elements.

The analysis was conducted as a staggered thermo-mechanical model whereby second-order structural elements are converted to first-order flow elements. Thermal and boundary properties were as per EN 1995-1-2 and EN 1991-1-2, respectively (BSI 2004c/2002). Grade C30 timber is assumed throughout with a characteristic density of 300 kg/m$^3$. Tensile strength was derived according to Thunnel (1941) assuming 80% fractile strength. The Modulus of Elasticity (MOE) as a function of temperature was determined using the previously outlined USRYOU subroutine (Section 4.3.2).

Timber beams 150 mm deep and 2 m long were subject to different utilisation ratios of 25, 50, 75 and 90%. The required loads to achieve such utilisation levels were derived using the reduced cross section method set out in EN 1995-1-2 for standard fire exposure. Target ‘failure times’ were also derived using this method. Where a “mixed” fracture energy is referenced, this implies an increasing fracture energy with temperatures as per Figure 4.25.

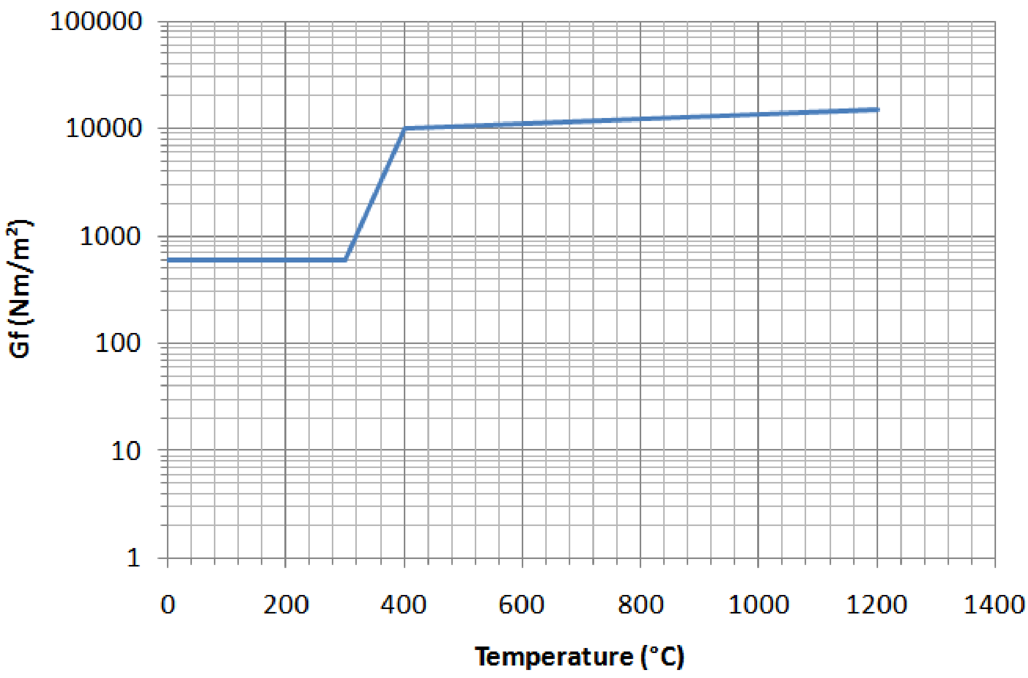



**Figure 4.25 Mixed fracture energy adopted in simulations**

The mixed fracture energy concept is introduced as a potential solution to numerical instability. Large strains can develop in the char zone of a beam, which contributes little to the mechanical resistance yet may govern the termination time of a simulation, should the total strain at the extreme char fibres exceed that of the ultimate crack strain. The application of a single large fracture energy for all temperatures (i.e. 5000 Nm/m$^2$ for all temperatures) may over-predict the load-carrying capacity of a timber beam and, as such, it is important to maintain realistic fracture energy values for un-charred timber. However, the application of a large value of fracture energy for high temperatures (i.e. >300°C) ensures that the softening branch of the material remains defined even when strains are large. This ensures that the crack strain is never greater than the ultimate value described by the softening relation, thus ensuring numerical stability. The parametric study conducted is summarised in Table 4.6.

**Table 4.6 Parametric study summary**

<table>
<tr><th>Group No.</th><th>Utilisation (%)</th><th>Fracture energy (Nm/m²)</th><th>Tension Softening</th><th>Target failure time (min)</th></tr>
<tr><td>1 (A-E)</td><td rowspan="2">25</td><td rowspan="8">600 (A), 1000 (B), 2000 (C), 5000 (D), Mixed (E)</td><td>Linear</td><td rowspan="2">66 (3960 s)</td></tr>
<tr><td>2 (A-E)</td><td>Hordyk</td></tr>
<tr><td>3 (A-E)</td><td rowspan="2">50</td><td>Linear</td><td rowspan="2">34 (2040 s)</td></tr>
<tr><td>4 (A-E)</td><td>Hordyk</td></tr>
<tr><td>5 (A-E)</td><td rowspan="2">75</td><td>Linear</td><td rowspan="2">13.5 (810 s)</td></tr>
<tr><td>6 (A-E)</td><td>Hordyk</td></tr>
<tr><td>7 (A-E)</td><td rowspan="2">90</td><td>Linear</td><td rowspan="2">5 (300 s)</td></tr>
<tr><td>8 (A-E)</td><td>Hordyk</td></tr>
</table>

Simulation failure is crudely taken as the last converged step. It is recognised that such a termination can be brought about due to numerical instability and not a physical failure. However, where fractures develop without alternative means of load redistribution, it is highly likely that failure is due to a violation of the stress-strain relationship for the material and thus can be considered as a ‘true failure’. This is particularly the case for instances where large fracture energy values, and thus large ultimate crack strains, are specified for the char layer, i.e. the mixed case, and where the structure is determinate.

### Results

Without supporting experimental data, the author has chosen to measure the relative impact of fracture energy on ‘failure’ time by comparing simulation termination times with predicted failure times, using the reduced cross-section method of EN 1995-1-2. Results are divided by tension-softening regime and as such plots of apparent simulation failure time and EN 1995-1-2 derived failure time are shown for Linear and Hordyk tension-softening regimes in Figures 4.26 and 4.27, respectively.

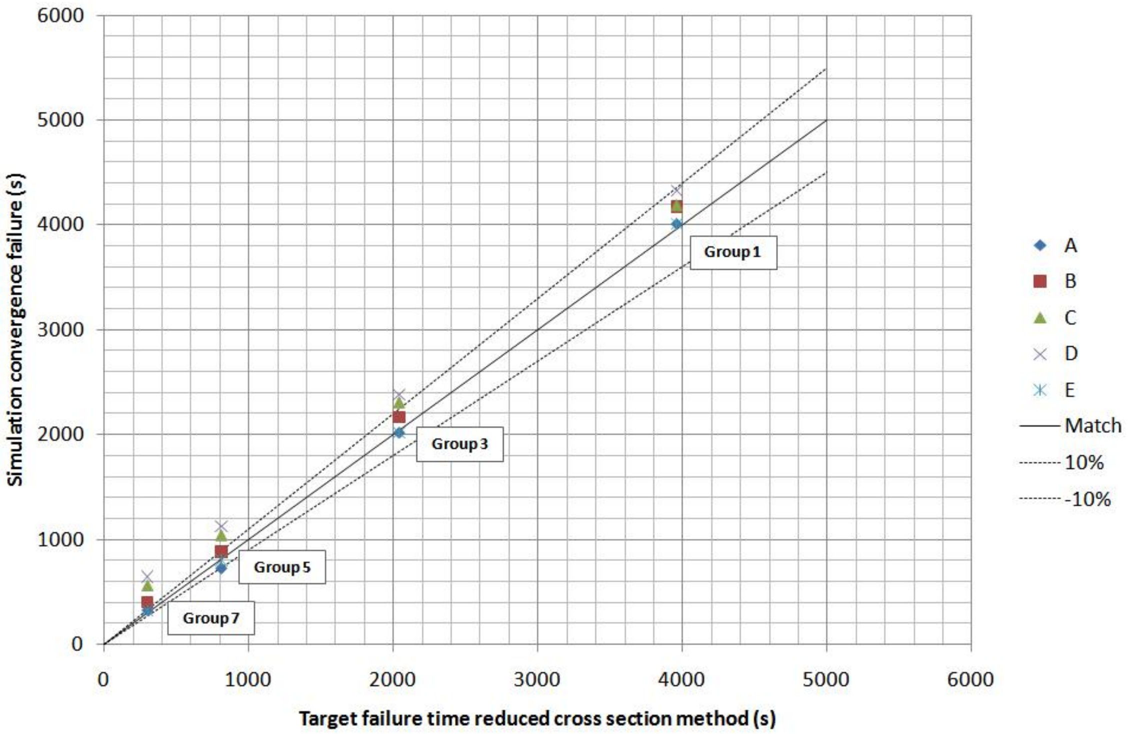



**Figure 4.26** Simulation termination time versus predicted failure time from EN 1995-1-2 (Linear tension softening)

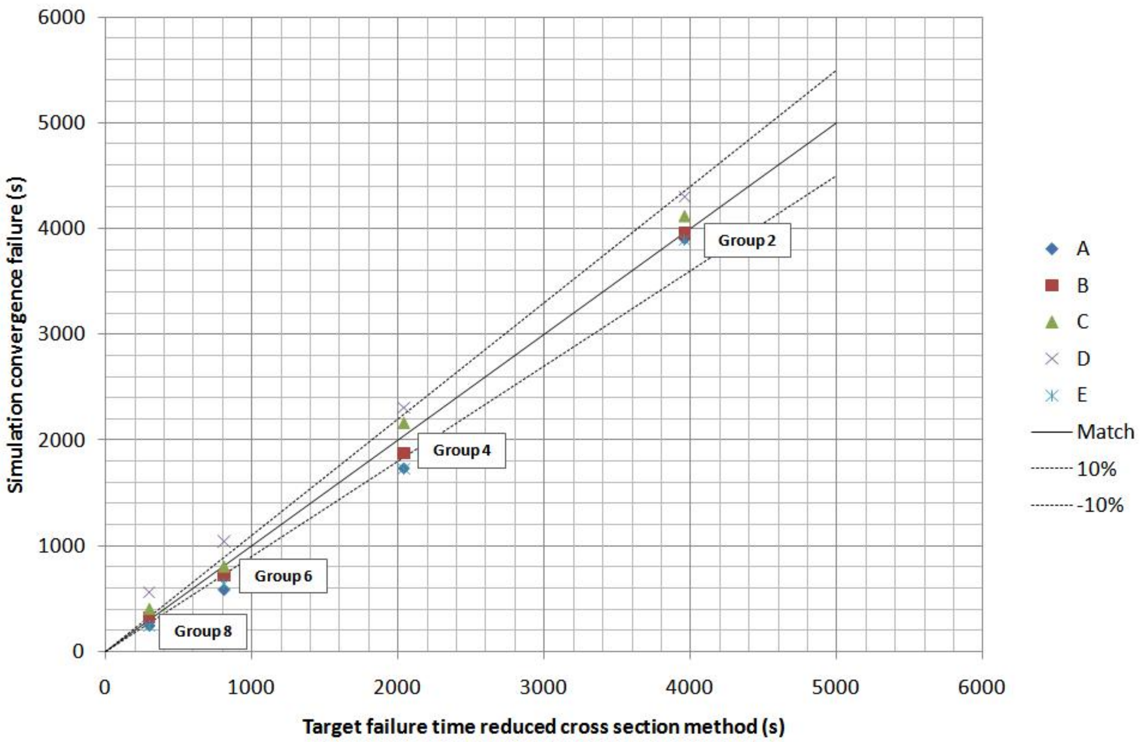



**Figure 4.27** Simulation termination time versus predicted failure time from EN 1995-1-2 (Hordyk tension softening)

**Discussion**

Figures 4.26 and 4.27 demonstrate that the assumed fracture energy has an important influence on the simulation termination time when a timber beam is exposed to fire from below and is subject to different levels of load. The larger the fracture energy, the more ductile a structural member behaves as crack stress is dissipated over a much larger crack strain.

In numerical simulations, the incorrect input of fracture energy can result in overall reductions in tensile strength as the values specified should be sufficient for the full tension-softening regime to be defined. In DIANA, the limiting tensile strength is dependent upon the tension softening regime, fracture energy, MOE and crack bandwidth (h). Where small crack bandwidths and fracture energies are introduced, reductions in tensile strength can occur, which impact heavily upon apparent 'failure time'. This behaviour was found to be more critical when Hordyk tension softening is adopted over Linear.

It is apparent that neither EN 1995-1-2 (BSI 2004c) or the literature give guidance on the magnitude of fracture energy that should be adopted when simulating timber at high temperature. For the purposes of modelling timber beams exposed to fire, it has been found that linear tension softening is adequate. A mixed fracture-energy approach (i.e. increasing $G_f$ with temperature) can ensure that numerical instability does not develop in the char zone, where strains are high, whilst also giving realistic strength characteristics and brittleness behaviour in the undamaged residual cross section.

## 4.4 SUMMARY

The previous sections present a number of developments and studies conducted by the author during the Engineering Doctorate research period. In instances where supporting publications are included for assessment (Appendices A through E), only brief overviews of the work undertaken are included. More detailed information can be found by referencing the relevant papers. It can be seen that a significant number of numerical studies have emerged from the experiments performed. The experiments and modelling attempts undertaken highlighted a number of limitations both in terms of physical understanding of timber and material properties available for modelling modern timber structures when exposed to fire. From this, new proposals for the thermal properties of softwood in natural fires have been developed, plasterboard behaviour in natural fires studied and computational developments made for implementation in commercial FEA codes.

# 5 FINDINGS AND IMPLICATIONS

The research conducted has been diverse, although under the common theme of the fire performance of engineered timber products and systems. Findings and conclusions can be drawn under three common themes, namely:

- Understanding failure modes and behaviour in real fires
- Simulating temperature development in timber systems
- Enabling fire design through computational modelling

In the following sections, each of the themes is fully discussed. The implication of the findings for BRE Global and, and more general, to the wider industry is also presented. Limitations of the current research and recommendations for further development are also highlighted.

## 5.1 THE KEY FINDINGS OF THE RESEARCH

The key findings of the research by theme are discussed below.

### 5.1.1 UNDERSTANDING FAILURE MODES AND BEHAVIOUR IN REAL FIRES

Engineered timber products, such as engineered floor joists and SIPs, have emerged as a result of trends towards leaner construction, faster erection times and more energy efficient buildings. All of these drivers fall under the overarching theme of sustainable buildings. The experimental element of this research has highlighted a number of important factors which demonstrate correlations between sustainability and fire safety.

Engineered floor joists are structurally as stiff as traditional sawn joists. They require significantly less material and, as a result, are much lighter. It has long been understood that redundancy, in terms of under utilised material, in a structure is beneficial for fire performance. This often allows for a degree of ‘inherent’ fire resistance, without the need for further protection. However, the fire experimentation conducted on typical engineered floor joists highlighted some areas of concern arising as a result of efficiencies made in section size (Section 4.1.1). It became clear that, due to the efficiencies made in cross sectional size, neither timber I-Joists nor truss web joists offer any appreciable inherent fire resistance. As a result, the fire ‘resistance’ of such members is wholly dependent on the fire protection offered by gypsum plasterboard. The fire performance of plasterboard, and hence its ability to protect from fire, is highly sensitive to the quality of workmanship upon installation. This may cause the failure of engineered floor systems, even when protected, to be unpredictable. In addition, as such joists have little inherent fire resistance, their failure in fires can be brittle and sudden. Indications from this research project suggest that both timber I-Joists and steel web joists can fail in a sudden catastrophic manner after plasterboard failure. In contrast, solid section joist floors tend to fail more slowly with a gradual creep in deflection. The failure behaviour of engineered floors has consequences for the fire and rescue service entering a building.

SIPs offer excellent thermal efficiency and speedy build times. This is particularly true when adopted as either a non-structural infill panels or as a substitute for structural walls and columns. A SIP, which is made of different materials, functions as a composite panel. As a result, the strength of the panel is much greater than the sum of strengths of its individual parts. The composite action between the insulation core and the facing boards of the panel is extremely important for a SIPs structural integrity. When SIPs are tested in isolation, their fire performance is of concern (Section 4.1.2). As the temperature within the core rises, de-bonding may occur between the temperature sensitive insulation and facing boards. This could lead to a loss of composite action, and ultimately buckling of the facing boards when adopted as compression members. This process may be delayed by the introduction of gypsum plasterboard which may keep the core cool for the desired fire resistance period. Much like engineered floor joists, SIPs appear to have little, if any, inherent fire resistance.

When SIPs were fire tested as part of an entire assembly, i.e. a two storey house (Section 4.1.3), the results were surprising. SIPs demonstrated a level of inherent fire resistance which was mobilised through a number of load redistribution paths. Firstly, unlike engineered floors, a large volume of redundant timber is present in SIP buildings. This is in the form of corner 'cripple' studs and framing members around windows and doors. Predictably, once the protective plasterboards of SIPs fail, a vast amount of damage in the panels occurs. This results in a complete loss of composite action which, in isolated members, would initiate failure. However, when acting as a part of a system, gravity loads were redistributed through ring beams, splining members, cripple studs and timber members forming façade openings. This prevented the collapse of any of the buildings tested. In relation to fire spread, the same detailing contributing to load re-distribution was found to be beneficial in preventing significant fire spread through insulated cavities. Where fire broke into the insulation cores, damage and fire spread was limited to isolated areas 'boxed in' by solid timber splining and framing members.

### 5.1.2 SIMULATING TEMPERATURE DEVELOPMENT IN TIMBER SYSTEMS

The importance of plasterboard in the fire performance of modern timber structures is apparent from the experiments conducted (discussed in Section 4.1). In many cases, the fire resistance of modern timber systems equates to the point at which plasterboard fails. Therefore, the prediction of plasterboard behaviour is vital in the design of timber buildings for fire resistance.

Review of the literature highlighted a plethora of inconsistent thermal properties for gypsum plasterboard. The impact of adopting the various available properties was highlighted in the work undertaken by the author (Section 4.2.1). None of the datasets, when adopted in non-linear thermal models of SIPs, appeared to give good correlations with temperatures measured in the experiments conducted by the author. As a result, new plasterboard properties were proposed for the efficient modelling of highly insulated timber assemblies. This element of work highlighted an important consideration for modelling gypsum protected structures. It is apparent that the properties proposed by the author and those in the literature are situation specific. Many properties, including the ones proposed by the author, are derived on the basis of ‘numerical calibrations’. This essentially means properties are calibrated to give the same temperatures determined via experimentation. As a result, when such properties are applied to problems different to those used for calibration, inaccuracies may arise. Further to this, due to variability in plasterboard formulations from different manufacturers, it is not possible to define characteristic properties which are likely to be applicable to all gypsum variants. This is likely to be the reason why plasterboard thermo-physical properties are yet to be included in EN 1995-1-2.

The simulation of plasterboard systems exposed to natural fires has proved challenging. This is due, partly, to the calibration process described above. Modelling the thermal response of timber, plasterboard or combinations of the two is common in the fire engineering research community. In doing so, effective properties that implicitly include physical phenomena difficult to model explicitly are usually adopted. Examples of such phenomena are moisture flow, gas movement, ablation and fracture. However, as calibration is conducted relative to standard fire tests (i.e. a prescribed well defined gas temperature regime) many, if not all, of the gypsum plasterboard properties quoted in the literature are heating rate and situation dependent. This was found to be the case when simulations of timber floors subject to natural fires were conducted (Section 4.2.1).

The concept of heating rate objectivity also extends to softwood. Annex B of EN 1995-1-2 gives thermo-physical properties appropriate for modelling wood exposed to the standard fire curve. The adoption of such properties in natural fires is known to exhibit inaccuracies for the reasons outlined above for plasterboard. In resolution to this, the author proposed simple modifications to the conductivity model outlined in EN 1995-1-2 for softwood. This model was shown to give excellent predictions of char depth when compared with the approach of EN 1995-1-2 Annex A, and with experiments conducted by König and Walleij (1999). Extending the model to include the cooling phase proved to be troublesome. As a result, a pragmatic FEA solution was proposed which allowed for the determination of residual reduced cross section using the outlined MCM. Predictions of depth of char using the author's MCM and EN 1995-1-2 Annex A have shown to yield comparable results. This is promising as, to date, the use of FEA as a design tool is limited to timber structures exposed to the standard fire curve only.

### 5.1.3 ENABLING DESIGN THROUGH COMPUTATIONAL MODELLING

The structural fire engineering performance based design of steel and concrete structures has been undertaken for some time and involves the design of structures to withstand a fire proportionate to the risks foreseen and the ventilation available. This process has benefits as the factor of safety for a fire exposed structure is quantifiable and efficiencies are often achievable, as regulatory fire resistance requirements can be overly onerous. To benefit from such a process, buildings are designed to withstand a natural fire or simple representation thereof. Limitations in EN 1995-1-2 mean that, to date, this is not achievable. As such, many of the beneficial aspects of whole building behaviour, which are widely acknowledged, cannot be realised for timber buildings. For large section timber this often means excessively large cross sections to achieve a given fire resistance requirement. The first barrier preventing the performance based design of timber structures is the determination of temperatures and depth of char in natural fires. The author has proposed a resolution to this in this thesis (Section 4.2.2). The second barrier is the analysis of complex timber buildings. To date, modelling the structural behaviour of timber is limited to using adhoc sectional analysis tools, or the use of commercial FEA codes for simple uni-axial problems (Fragiacomo, *et al.* 2010b). This is because timber is a complex material with properties dependent upon temperature, temperature history and stress state.

In recognition of the above, a number of FORTRAN subroutines (USSs) were developed which enabled commercial FEA codes, in this instance TNO DIANA, to be modified such that large timber structures exposed to fire can be modelled (Section 4.3.2). The routines developed are at present theoretical. However, the content presented in this thesis gives an outline integrated procedure for modelling large-scale timber assemblies exposed to both standard and natural fires.

The previously proposed MCM can be adopted with the developed USSs , and with a number of other complex aspects of material behaviour, to perform nonlinear thermo-mechanical analyses. This has been demonstrated through a further study using a series of FEA models (Section 4.3.3). In this study, the implementation of the constitutive behaviour of timber using relatively simple Total Strain Based models, the use of appropriate fracture energy models, and the use of the proposed USSs were all considered. More studies are needed for further development of these approaches before its use as design tools. However, initial findings appear to give consistent results with many empirical methods for isolated structural members present in EN 1995-1-2.

## 5.2 CONTRIBUTION TO KNOWLEDGE

The natural fire experimental work discussed in this thesis (Section 4.1) is the first of its kind internationally. A number of US and Canadian studies have investigated engineered floor behaviour. However, these were either furnace experiments (subject to the standard fire curve) or were conducted on unprotected ceilings (i.e. no gypsum boarding), such as would be used in North American basements. All the experiments conducted as part of this EngD included engineered timber floor systems of one form or another. The results obtained allowed for authoritative scientific conclusions to be made regarding the failure modes of engineered floor joists in fire. The results confirmed that engineered floors can fail in a more catastrophic manner than traditional joist floors. These findings are in line with experiences seen overseas, where fire and rescue service personnel have been injured as a result of engineered floor collapses.

The studies conducted on two storey SIP buildings are also unique. The project highlighted possible systemic failure modes for entire prefabricated timber buildings. The knowledge and measurements gathered have been used, and will be invaluable in the future, for the development and verification of numerical models. The ultimate aim of such models should be to assist in the performance based design of timber and timber composite structures.

Little, if any, progress has been made regarding the thermal response of both timber and plasterboard in fires. Flagship studies, such as those conducted by Mehaffey, et al (1994), Sultan (1996), König and Walleij (2000) and Thomas (2002) have not been significantly advanced. Attempts have been made in recent times, however these have largely remained focussed upon standard fire exposure (Frangi 2001 and Fragiacomo, *et al.* 2010a). In this regard the research presented is innovative as it has been successful in advancing the above studies, taking them from standard fire exposure to parametric and natural design fires.

Clearly, the latter represents a more realistic situation which, when utilised, should greatly improve the understanding of the real behaviour of structures.

Many of the important developments made in relation to the adaptation of commercial FEA codes for modelling timber in fire (Section 4.3.2) are general. Therefore, they should be valuable to other researchers. As a result, detailed studies of the behaviour of timber structures of various complexity exposed to fire, can be performed. The theory applied is fundamental. However, much like most timber advances made in the last 25 years, the developments are pragmatic and yield conservative approximations of timber behaviour in fire. This should lead to better understanding of the true behaviour of a wider class of problems that were not possible to consider in the past.

Finally, to date, the use of fracture energy concepts (Section 4.3.3) for wood at elevated temperature has not been researched in any detail. However, this area was explored by the author as an alternative to simple perfectly brittle assumptions. An assumption of ideally brittle timber behaviour usually leads to highly unstable numerical models. Clearly, this is an effect that most practitioners would like to avoid. For this reason, pragmatic solutions have been proposed which yield comparable results to EN 1995-1-2 and, in the meantime, can be adopted in studies of more complex timber buildings.

## 5.3 IMPLICATIONS / IMPACT ON THE SPONSOR

BRE Global has benefited from the research in a number of ways. Firstly, prior to completion of this study, the advanced analysis (FEA) of structures exposed to fire was not an available capability. However, as a result of this work BRE Global is now able to offer sophisticated FEA solutions as a service to clients. Clearly, this will help to reinforce its position as the UK’s leading fire research centre. These solutions can be further developed to support not only the research and consultancy business, but also the testing and assessment area. In the latter case assessments supporting costly fire resistance tests can be conducted using FEA tools.

The research conducted herein is innovative and is driven by market trends/demands in the construction industry. The fire performance of timber buildings is politically relevant as the market share continues to grow, new technologies emerge and an increasing number of fire incidents occur. As a result BRE Global is leading innovative research in this area. The fundamental work undertaken in this doctorate will form a foundation for subsequent research funding applications and proposals via organisations such as the BRE Trust and the NHBC Foundation. Both are charitable organisations who offer research funding in all areas of the built environment.

## 5.4 IMPLICATIONS / IMPACT ON WIDER INDUSTRY

Ultimately, the studies and the test data reported herein have the potential to influence government fire guidance, in particular recommendations in Approved Document B (ADB) to the Building Regulations. ADB provides guidance on ways to meet the functional requirements of the Building Regulations with regard to fire safety. The experiments reported herein were commissioned by CLG as part of the ADB supporting research programme. In addition, the results of the experiments have been published in UK SIPs Association (UKSIPSA) and National House Building Council (NHBC) guidance. Both of these documents will inform and influence how engineered timber is used in the UK. The experiments have helped dispel a number of misconceptions related to SIP construction. This should allow for a wider use of SIPs in mass produced buildings, allowing the relevant industries to flourish.

Findings in relation to the engineered floor joist studies have been disseminated to the Fire and Rescue Service through the parallel CLG project Stakeholder Group. As a result, fire fighters are now more aware of how engineered floors behave, and may collapse, in fire. This should lead to the development of safer and more efficient fire fighting techniques.

The presented numerical modelling, and in particular the proposed MCM and its scope of application, have the potential to further improve the methods contained in EN 1995-1-2. However, this requires additional experimentation, validation and development of analytical procedures.

It is anticipated that the developed USS will be adopted by consulting engineers and by BRE Global for the performance based design of timber structures.

Much of the work contained herein will be published through the IHS/BRE press who are the BRE Groups publishing organisation. As a result the findings will be disseminated to the wider public and to the industry. This is in line with the company's ethos. In addition, the work conducted herein has been published in trade magazines and journals, as well as scientific publications, to ensure that both practitioners and researchers are aware of the developments.

## 5.5 RECOMMENDATIONS FOR FURTHER RESEARCH

The conducted research has highlighted a number of further questions to be answered, and areas for further research. These are summarised below:

1. Failure modes for engineered floors have been identified through full scale fire experiments. Based on this knowledge, it is desirable to derive 'design' methods which can be

used to determine the fire resistance of engineered floors. Ultimately, these methods should be simplified enough for use by practitioners.

2. The thermal response of SIPs exposed to standard fires has been successfully simulated using heat transfer models. However, this should be extended to cover response to natural fire exposure. This requires more studies and experimental work to be conducted relating to how gypsum plasterboard behaves in natural fires. In addition, the thermo-mechanical response of SIPs exposed to fires is yet to be evaluated. Given data collected from the performed small and large scale tests, it should be possible to couple thermal and mechanical behaviour to successfully simulate fire resistance tests and real fire conditions.

3. Where performance based design for timber is to be considered for gypsum protected systems, plasterboard fall off time needs to be determined due to exposure to natural fires. The impact of heating rate on failure time should be observed to determine possible relationships between energy exposure and fall off time.

4. The developed MCM should be further validated for the heating phase of natural fires through the performance of additional experiments. Transient experimentally measured temperature profiles in timber members exposed to real fires could be used to provide validation data.

5. Standard fire tests on large section timber beams should be used to validate the developed USRYOU, USRCST and USRTST subroutines.

6. The fracture energy of timber at different temperatures should be observed using simple steady state uni-axial experiments.

## 5.6 CRITICAL EVALUATION OF THE RESEARCH

The research undertaken is valuable to a number of parties, including BRE Global, the wider academic research community and industry. However, it has limitations. More extensive instrumentation could have been used to monitor the experiments, and to collect more data. This is particularly true in the case of the large scale fire experiments. Where simulations of such tests have been conducted, compartment temperatures have been estimated based upon measured ceiling temperatures. This is not ideal for two reasons. Firstly, temperature gradients through the height of the compartment were not measured. This means that any simulations conducted may need to assume a uniform temperature, for walls, corresponding to that of the ceiling. Secondly, the used bead thermocouples represent the ‘convective’ gas temperature and not that due to radiation. Plate thermometers could have been used in parallel alignment to surfaces to determine the radiation incident on the surfaces of the building. This would allow for more accurate simulations to be conducted. In addition, measurements of strain would also have benefited any subsequent modelling activities.

The reported numerical developments also have its limitations. The MCM has been developed on the basis of charring depths proposed in Annex A of EN 1995-1-2. This means that the validity of the MCM depends upon the accuracy of the codified equations. Ideally, the MCM would have been developed in isolation, using experiments conducted by the author. This would ensure that the model developed was independent of EN 1995-1-2 and also was validated against well defined experiments, with detailed temperature measurements. This was not possible due to time and budget constraints. However, the MCM has been partially validated using test data provided by SP Trätek. This element of the research would certainly benefit from further experiments. In addition, the adaptation of the author’s MCM for cooling is an engineered approach. The complex physical phenomena, present in cooling timber, are not adequately addressed and as such the method does not advance physical understanding. The approach provides a simplistic practical solution to a very complex problem.

Conclusions relating to the importance and impact of fracture energy are made on the basis of comparisons with EN 1995-1-2. Therefore, since supporting experiments do not exist, the validity of the related conclusions depend upon the accuracy of the results of simple analytical tools, such as the reduced cross section method.

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# APPENDIX A LARGE SCALE NATURAL FIRE TESTS ON PROTECTED ENGINEERED TIMBER FLOOR SYSTEMS

**Full Reference**

LENNON, T., HOPKIN, D.J., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. Large scale natural fire tests on protected engineered timber floor systems. *Fire safety journal,* **45**(3), pp 168–182.

**Abstract**

As part of an ongoing research project to investigate the performance in fire of specific types of Innovative Construction Products and Techniques (ICPT), the BRE have carried out large-scale fire tests to determine the response of different floor systems to a realistic fire scenario. The principal objective was to determine the mode of failure of different floor systems to provide information to key stakeholders (particularly the Fire and Rescue Service), which can be taken into account in the dynamic risk assessments that underpin fire fighting operations. This paper presents the results and observations from those fire tests for three floor systems: (i) solid timber floor joists, (ii) I-section floor beams with solid timber top and bottom flanges and an Oriented Strand Board (OSB) web, and (iii) a timber truss incorporating solid timber upper and lower chord members and a pressed steel web member. These reflect the two most common types of engineered floor systems used in the UK and allow for direct comparison with a more "traditional" form of construction.

**Keywords**

Fire testing, compartment fires, floor joists, structural fire engineering, timber

## 1. Introduction

Timber is a widely used construction material for structural framing. In residential and low rise commercial applications light timber frames are commonly used. In traditional construction methods, the walls and partitions of a light timber frame are constructed from sawn timber studs. The floors are formed from plywood or particle board sheeting fixed to solid timber joists. However, new build properties are increasingly utilising ‘engineered floor joists’, such as I-section joists with an OSB web and truss joists with steel web sections, as an alternative to solid timber members. In general, the engineered floors are lighter and may be used to cover longer spans compared to solid joists. In addition, due to their relatively light weight, they are regarded as being more ‘buildable’ as they can be handled manually by fewer people on site. This drives down the build time thus reducing construction costs.

Within the fire engineering community concerns have been raised regarding the fire performance of some innovative engineered timber floor joists. The general perception is that such systems may not perform as well as solid timber members when exposed to natural fires [1]. In addition, there is a concern that they may fail in a sudden catastrophic manner with little warning [2]. These concerns are currently being addressed in a fire testing programme commissioned and funded by the Department for Communities and Local Government (CLG) as part of on-going research under the more general theme of the fire performance of innovative construction products and techniques.

In early 2009 BRE, with funding from CLG, started a programme of large scale fire tests on timber floors exposed to natural fires. The main purpose of the programme was to establish the general behaviour of engineered floor systems in fires and to determine if such joists fail in a more catastrophic manner than traditional solid timber floor joists. Hence, a traditional floor of the latter type was fire tested to provide the control data required for a comparative study. In addition, two more engineered floor systems commonly used in the UK were also fire tested. One system comprised I-section floor beams made of solid timber flanges and an OSB web. The joists of the second floor system were trusses whose upper and lower chords are made of solid timber, and all web members were made of pressed steel, with the exception of two mid-span timber down stands. The methodology adopted and the results and observations from these natural fire tests are the subject of this paper.

## 2. Test methodology

### 2.1 Compartment design and member specification

The three tests were performed at BRE's North East test facility in a single storey compartment formed from concrete blocks. The compartment was designed to dimensionally reflect a typical domestic dwelling with an associated design loading appropriate for this purpose. The compartment had internal dimensions of 4 m by 3 m, with the joists spanning in the long direction. The floor-to-ceiling height of the compartment was 2.4 m. The compartment had two ventilation openings in one of the short and one of the long sides, both measuring 0.75 m x 1.00 m. The compartment is illustrated in Figure A.1. Based on this layout, the size and spacing of the engineered floor joists of each type of floor was determined by the corresponding floor manufacturer. The resulting member sizes (with centres at 400 mm) are summarised in Table A.1.

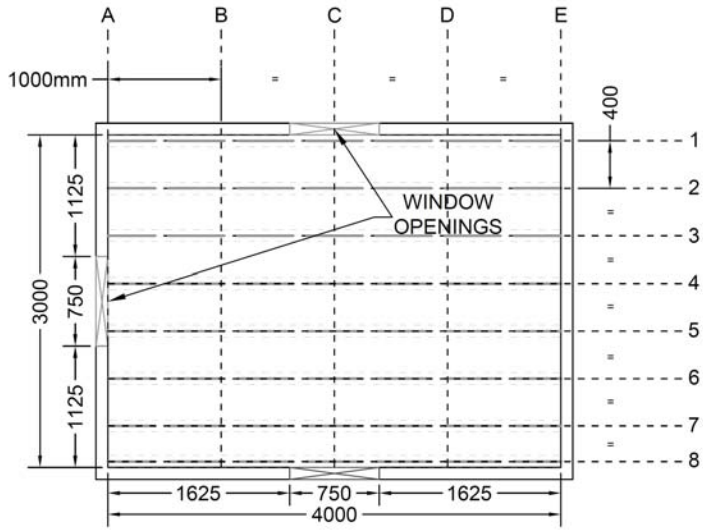


**Figure A.1a. experimental compartment: plan**

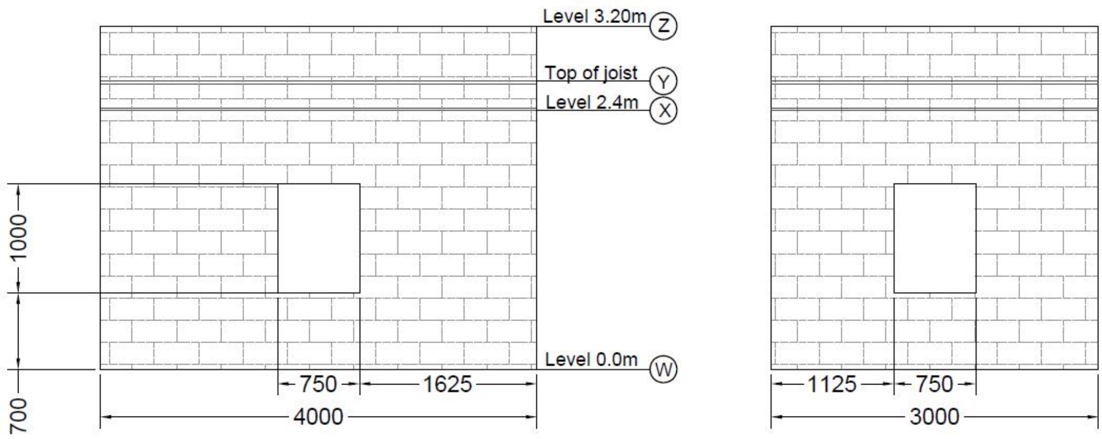


**Figure A.1b. experimental compartment: elevation**

The joists were connected to the supporting masonry walls using common joist hangers. The hangers were manufactured from cold formed thin steel sheet (1mm thick). These were embedded into the mortar between block work courses.

**Table A.1. Joist details**

| | Overall dimensions (mm) | Flange dimensions (mm) | Web | Web to flange fixing |
|---|---|---|---|---|
| Solid timber joist | 45 × 220 | N/A | N/A | N/A |
| Engineered I-section joist | 45 × 220 | 45 × 45 | 9 mm OSB (structural grade) | Phenol-formaldehyde adhesive |
| Engineered truss joist | 72 × 220 | 72 × 45 | Cold formed steel pressed web (1mm gauge) | Mechanical nailing plates with 7mm protruding teeth |

Photographs showing the block work compartment and joist construction for the solid timber joist floor are shown in Figure A.2. An image indicating the type of joist hanger used is shown in Figure A.3. The dimension W is dependant on the size of joist used.

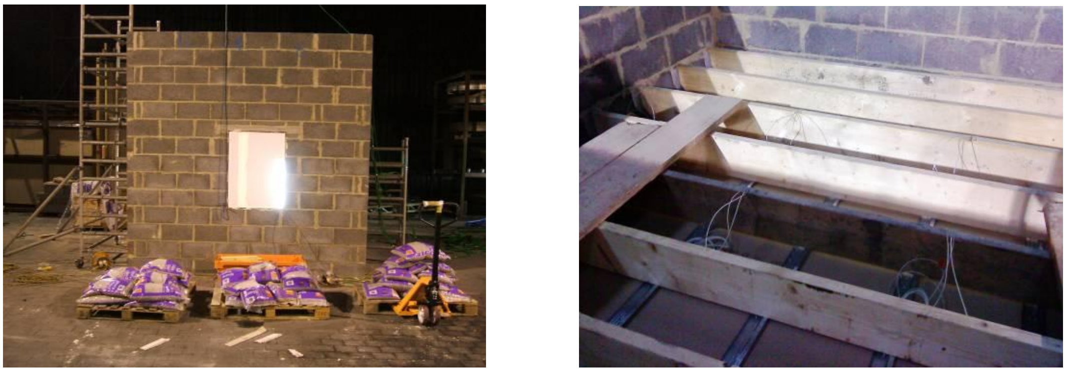

**Figure A.2. Test compartment (a) and joist construction (b)**

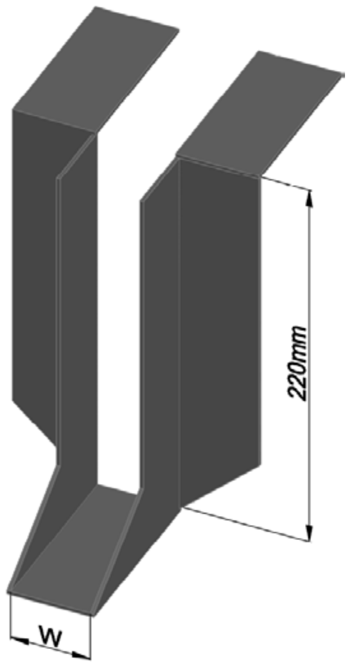


**Figure A.3. Joist hanger detail**

## 2.2 Passive fire protection

For each joist type a passive fire protection (PFP) system suitable for 60 minutes fire resistance, in accordance with the UK building regulations [3], was adopted based on manufacturer's guidance and the results of fire resistance tests. The PFP adopted in the instance of the solid timber joists was chosen based on the guidance contained in the British Gypsum "White book" [4]. For the solid timber floor joists and steel truss web joists, 25mm of type F plasterboard was specified for the floor to achieve a regulatory rating of 60 minutes fire resistance. Comparatively 30mm of type F plasterboard was specified by the manufacturers of the engineered I joists.

In all instances the internal lining was fixed by an experienced contractor. Firstly 45mm wide resilient bars were fixed at 400mm centres spanning perpendicular to the floor joists using 38mm screws. A layer of either 12.5mm or 15mm fireline plasterboard (depending on the joist type) was then fixed to the resilient bars using 32mm drywall screws at 400mm centres. A second layer of either 12.5mm or 15mm fireline board was fixed through the inner layer of the board to the resilient bars using 44mm dry wall screws at 230mm centres. The outer layer of board was staggered such that no joint in either layer was in the same position. All joints in the outer layer were filled with a generic ready-mix jointing cement.

The resulting through-depth floor construction comprised 25 mm or 30 mm of fireline plasterboard (depending on the system tested) fixed to the underside of the floor joist via resilient bars. In all instances the fixing arrangements were similar. The flooring above the compartment was formed using 22 mm of ‘tongue and groove’ chipboard fixed to the top of the joists via 38mm screws at 400mm centres. No form of insulation was placed in the floor void. A typical section through the depth of the floor is shown in Figure A.4.

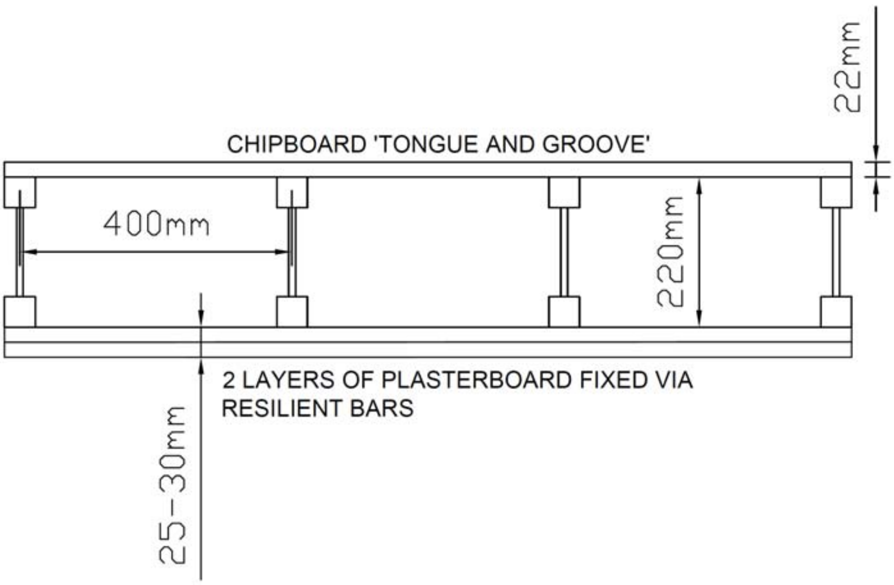


**Figure A.4. Typical section through floor (dimensions in mm)**

### 2.3 Fire design scenario

The purpose of the tests was to evaluate the response of the floor systems to a “realistic” fire scenario such as may occur in a room within a modern apartment building. The dimensions and ventilation condition within the room of origin were consistent with a fire within the living area of an apartment building where the door is closed. They are also consistent with previous research into the performance of timber frame structures in fire [5]. To provide a comparison with the results from standard fire tests and to evaluate the performance of the passive fire protection the fire load density was calculated such that the floor joists would be exposed to a natural fire of a severity equivalent to 60 minutes of exposure in the standard fire test. The figure adopted of 450MJ/m² is lower than codified design values but is in line with published average fire load densities and consistent with previous research into the performance of medium rise timber frame buildings in fire [5, 6]. The equivalent severity was calculated using the time equivalence method of BSEN 1991-1-2 [7] taking into account the number and size of openings and floor area. In addition the predicted compartment time-temperature response was calculated according to the parametric approach detailed in annex A of BS EN 1991-1-2 [7].The design fire load was provided by twelve cribs each with 25 kg of solid timber, giving a total fire load of 5400 MJ, for each test.

### 2.4 Instrumentation

Each test was instrumented in an identical manner. Type K thermocouples were placed in a number of positions to measure the following parameters:

- the compartment atmosphere (fire) temperature;
- the temperature of the bottom flange or, in the instance of the solid joist, the temperature at ¼ depth;

- the temperature of the top flange or, in the instance of the solid joist, the temperature at ¾ depth;
- the temperature of the web or, in the instance of the solid joist, the temperature at ½ depth;
- the temperature of the joist hangers at ¼, ½ and ¾ depths.

Three joists were instrumented in each test with thermocouples at the ends, mid span and quarter points of the members. In total 60 temperature instruments were used in each test. An image of an instrumented (steel web) joist is shown in Figure A.5.


**Figure A.5. Joist instrumented with thermocouples (shown with arrows)**

Additionally, nine linear voltage displacement transducers (LVDT) were placed on three members at the quarter and mid-points of the members. One end of the transducer was fixed to the chipboard sheathing whilst the other was fixed to a reference frame, which spanned across the compartment walls and provided a fixed datum, against which displacement could be measured.

An instrumentation plan for all three tests is shown in Figure A.6. Figure A.6a shows the placement of thermocouples, with depth for each of the locations indicated in Figure A.6b.

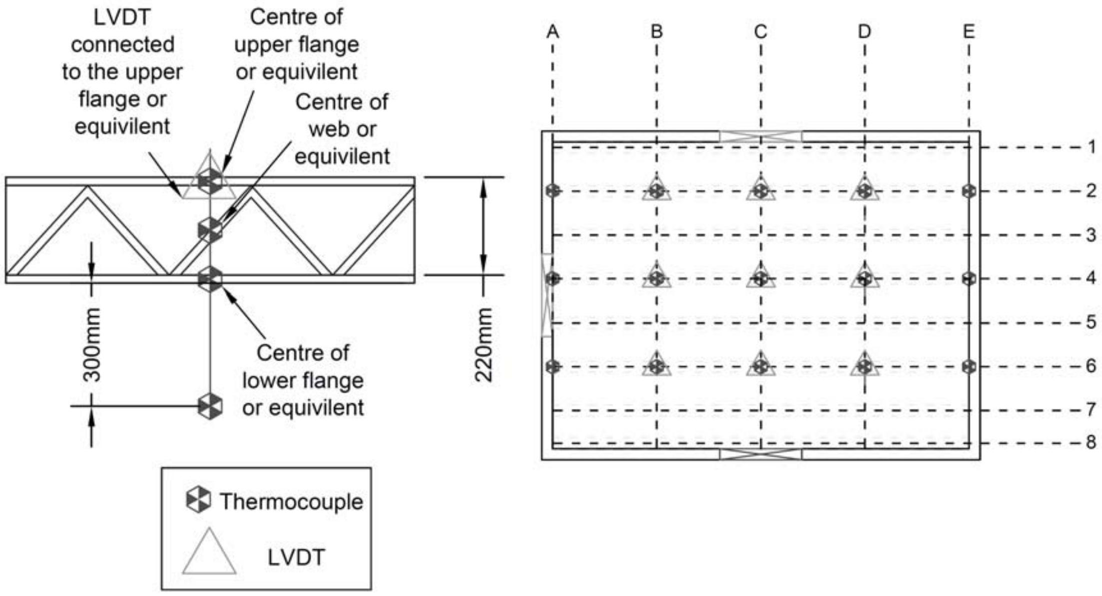


**Figure A.6. Instrumentation plan: (a) placement of thermocouples; (b) plan of compartment**

### 2.5 Floor Loading

The floor system was designed for an imposed load of 1.5 kN/m², in addition to its self-weight, which is representative of a typical load in a residential building as per BS EN 1991-1-1 for category A buildings [8]. At the fire limit state the structure will typically be designed for a combination of 50% of the imposed load and 100% of the dead load [7, 9]. This load combination is consistent with the guidance in the UK National Annex to BS EN 1991-1-2 [10]. Therefore, a uniformly distributed load of 0.75 kN/m² was applied to the floor system during testing using 36 sandbags (each 25 kg). An image of the upper side of the floor prior to testing is shown in Figure A.7.


**Figure A.7. Floor plate with loading**

The floor deflections due to the application of the imposed load were measured by taking readings prior and post application of the load. The difference between these two values was taken as the deflection to due loading. The maximum recorded deflection was just over 3.5 mm.

<u>3. Test procedure</u>

All three tests were completed in February and March of 2009 at BRE's North East test facility. The first test was performed on traditional solid timber floor joists, the second test was on OSB web joists and the final test on steel truss web joists concluded the experimental programme. The timber cribs were constructed in the compartment using 50x50x500mm timber battens each weighing approximately 0.5kg. Each level of the crib comprised 5 battens divided evenly over 10 levels giving an overall plan dimension of 500x500x500mm. These were arranged in the compartment as shown in Figure A.8. In all instances the fire was initiated by lighting paraffin soaked fibre strips, which connected the three rows of cribs. Approximately 1.75 litres of paraffin was divided evenly between the 6 fibre strips and 12 cribs. Each strip was ignited in sequence and the fire was allowed to develop naturally until either the fire load was consumed or a decision to terminate the test was made. For completeness the sequence of ignition of the fibre strips is also shown in Figure A.8. At the time of ignition, in all tests, the timber fire load had an average moisture content of 12%. Readings of all instruments were taken at 15 sec intervals from ignition until termination of the test.

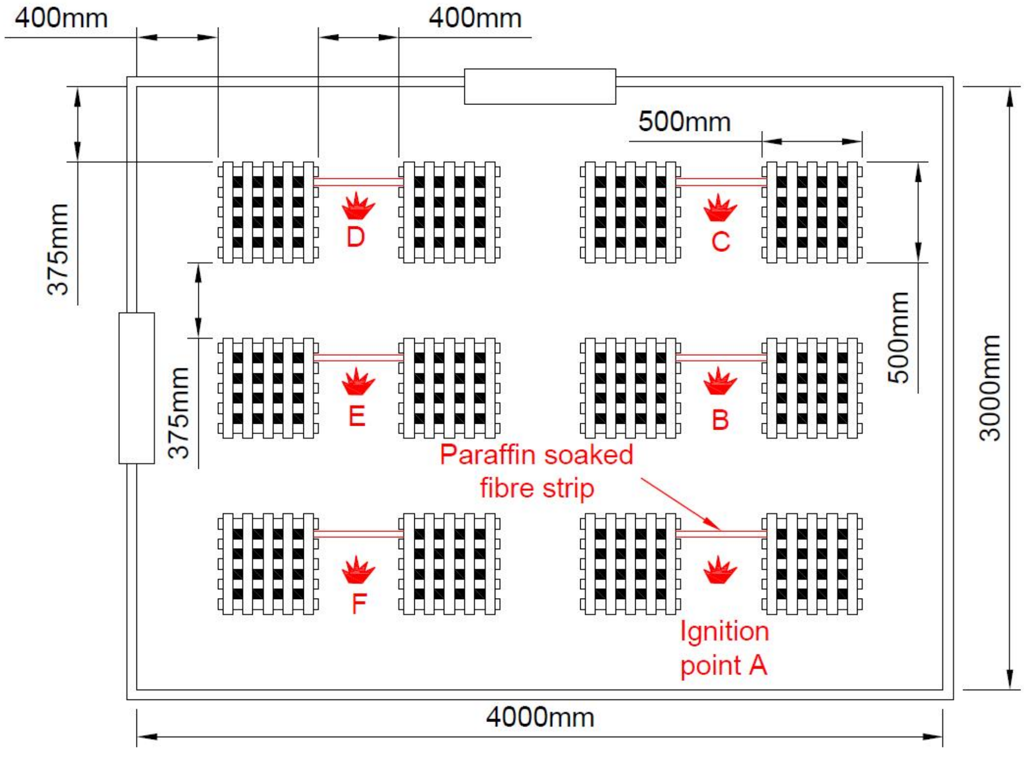


**Figure A.8. Fire load arrangement plan**

4. Results

## 4.1 Test 1 - Solid timber floor joists

The key observation and measurements noted for the solid timber floor test are summarised in Table A.2 alongside those taken for the experiments on the two engineered joist alternatives. Once ignited the fire was allowed to develop and burn naturally. At approximately 55 minutes into the test a decision was made to terminate on the basis of a loss of integrity in the compartment and the likelihood of damage to the instrumentation above the compartment. At this point extensive flaming was seen in the ceiling space, with flames impinging directly on the underside of the chipboard surface. Figure A.9(a) shows the timber cribs shortly after ignition. The fire resulted in a linear increase in temperature until flashover occurred within the compartment, after approximately 18 minutes. The fully developed fire is shown in Figure A.9(b).

**Figure A.9. (a) Cribs burning pre-flashover; (b) Fully developed fire**

**Table A.2. Key observations from all tests**

| Test | Test term. time (min) | Peak gas temp (°C) | Time of peak gas temp (min) | Peak joist temp (°C) | Time of peak joist temp (min) | Peak deflection (mm) | Peak rate of deflection (mm/min) | Breach of PFP (min) | Nature of breach |
|---|---|---|---|---|---|---|---|---|---|
| Solid floor joist | 56 | 1084 | 44 | 773 | 52 | 31 | 1.0 | 30 | Fall off of ceiling boards |
| OSB web I joist | N/A | 1034 | 42.5 | 312 | 60 | 10 | 0.2 | 50 | Minor gapping at board joints |
| Steel truss web joist | 56.5 | 1036 | 41 | 756 | 56 | 92 | 6.4 | 40 | Fall off of ceiling boards |

Figure A.10 shows the measured compartment temperatures for the 15 locations identified in Figure A.5. The fire severity, as indicated by the area under the curve, is equivalent to 60 minutes exposure to the standard curve. The parametric design curve for the fire provides a reasonably accurate prediction of peak temperature and overall duration. It can be seen that some temperature points are in excess of the parametric prediction. This is due to the approximate nature of the parametric post flash-over model. Although the cooling phase was curtailed due to fire brigade intervention, at this stage the peak compartment temperatures had been attained and the fire was in the early stages of the cooling phase.

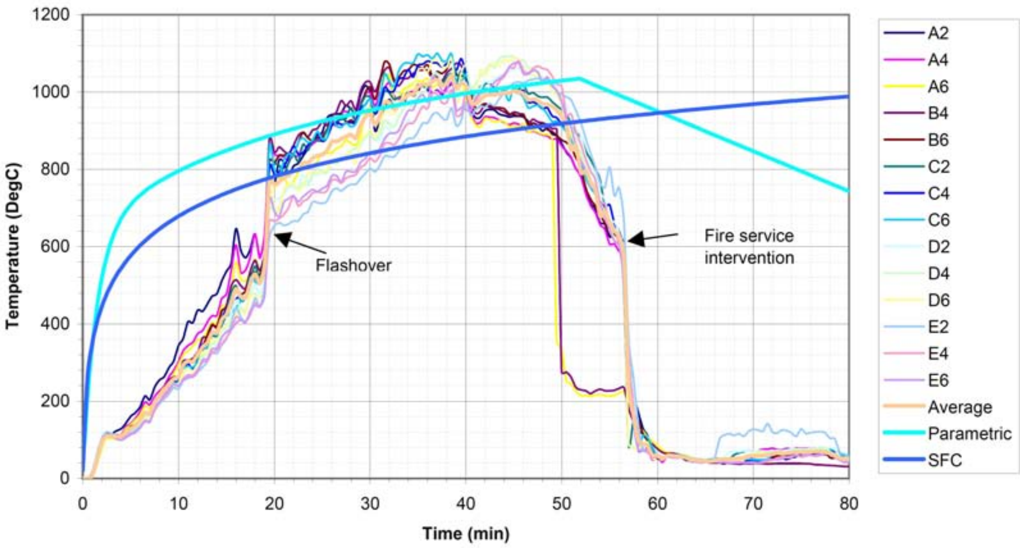


**Figure A.10. Compartment temperatures against design parametric fire and standard fire curve (SFC)**

During the steady-state phase of fire development (approximately between 30 to 50 minutes), with compartment temperatures around 1000ºC, a number of the wall boards fell away from the masonry substrate. During this phase, localised areas of the exposed layer of ceiling board fell into the compartment with gaps opening onto the upper layer, allowing hot gases and sporadic flaming into the floor space between the joints. Eventually, the final layer of plasterboard fell away locally, allowing the fire to spread into the ceiling space, as shown in Figure A.11.

**Figure A.11. Fall away of plasterboard layers on ceiling**

The extent of the damage to specific joists following localised failure of the ceiling linings can be assessed with reference to both temperature readings and visual observation. Thermocouple readings indicate temperatures on the surface of the joists in excess of 800ºC which is well above the notional ignition temperature of the material. Peak average joist temperatures by measurement location are summarised in Table A.3 for all three tested floor systems. From visual observation the most significant damage occurred in the North East corner of the compartment in the area between the two window openings. The temperatures measured at point B2, which is local to this area, are shown in Figure A.12. The results are confirmed by visual observation. Figure A.13 indicates the depth of char on joist 2 located near the window opening.

**Table A.3. Peak temperature of joists during test duration**

| | Peak average joist temperature by measurement location (°C) | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A2 | A4 | A6 | B2 | B4 | B6 | C2 | C4 | C6 | D2 | D4 | D6 | E2 | E4 | E6 |
| Solid floor joist | 773 | 406 | 343 | 734 | 235 | 313 | 473 | 292 | 292 | 273 | 387 | 240 | 264 | 214 | 272 |
| OSB web I joist | 50 | 173 | 124 | 125 | 172 | 123 | 105 | 313 | 153 | 140 | 173 | 107 | 86 | 91 | 63 |
| Steel truss web joist | 285 | 367 | 381 | 382 | 559 | 575 | 376 | 756 | 460 | 242 | 417 | 418 | 168 | 243 | 196 |

It is apparent from these figures that the joists near the window openings were severely exposed to the naked flames. For example, joist 2 at locations B and C had the largest deflections, signifying the onset of failure. The location of these large deflection readings corresponds to the location where the plasterboard fall off occurred and where the joists ignited.

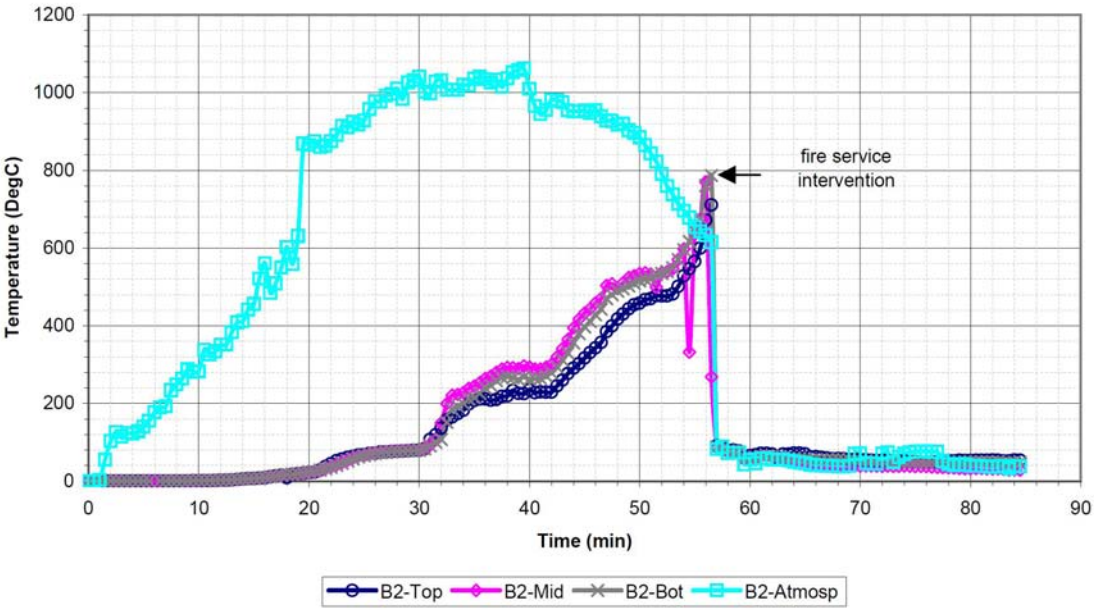


**Figure A.12. Joist temperatures, location B2 in Test 1**

**Figure A.13. Depth of char on joist 2**

Prior to termination of the test on 56 minutes the peak rate of deflection of joist 2 was 1.0 mm/min, which occurred within the period of 32 to 58 minutes of fire exposure. To illustrate the relationship between the temperature of the joist and its deflection, these parameters have been plotted with respect to time on a single graph for locations B2 and C2. This is shown in Figure A.14. It is clear that the onset of vertical deflection corresponds with the high average temperatures measured on the joists at these locations. An average temperature of the joist per location has been used as very little variation with respect to depth in the floor void at each measurement location was observed. This is calculated as the sum of the temperature measurements at ¼, ½ and ¾ depths divided by three.

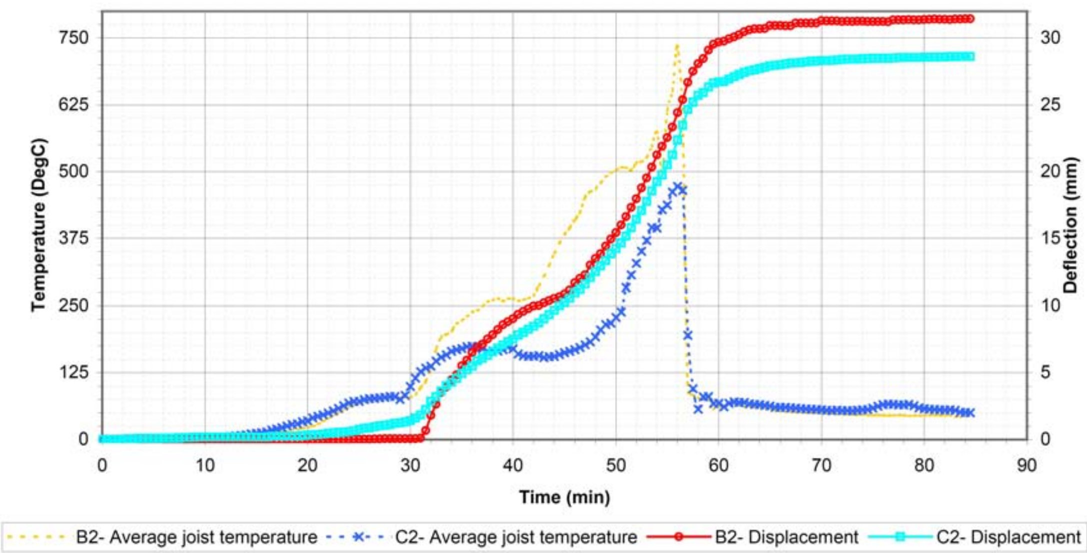


**Figure A.14. Floor deflection and joist temperature for locations B2 and C2, Test 1**

## 4.2 Test 2 – Engineered I Section floor joists with OSB webs

Test two was completed two weeks after the initial test on solid floor joists in the same block-work compartment. The joist hangars from the first test were reused as they were not damaged and were correctly sized for the I-sections. It is important to stress that, in this instance, the specified internal lining for 60 minutes fire resistance was 30 mm of fireline compared to 25 mm for the solid joist case. The fire was allowed to burn naturally for 70 minutes. At this point the fire brigade were asked to accelerate the cooling process so safe access to the compartment could be established for further inspection of the floor system. The change in compartment temperature with time is illustrated in Figure A.15, which also shows a comparison with the design parametric fire curve and the standard fire curve.

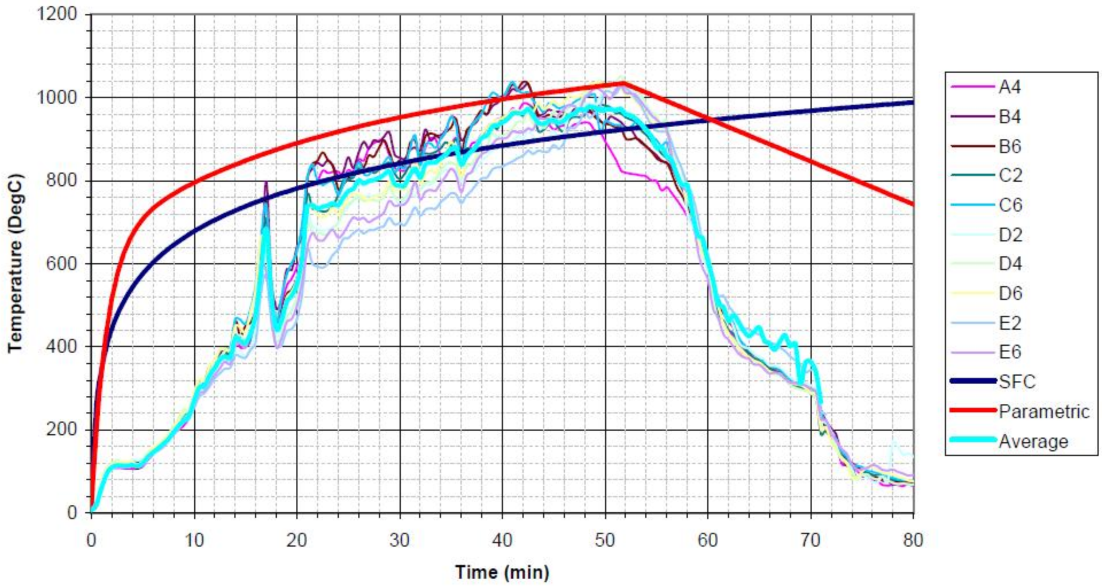


**Figure A.15. Compartment temperatures against parametric and standard fire curves**

Problems with cross-draught led to fluctuations in atmosphere temperature within the test compartment. This was overcome by restricting the ventilation to the laboratory. Again, the post flashover parametric model shows good correlation with the overall peak fire temperature and with the time to peak temperature.

The overall fire development was very similar to that of the previous test, and flashover occurred within a similar time frame. However, in this test the plasterboard appeared to survive for longer. After approximately 49 minutes from ignition, temperatures within the compartment reached their maximum values. At this point, the first observed loss of the exposed layer of ceiling board occurred. However, the inner layer of boards remained relatively intact and continued to protect the floor joists until the flames were no longer in contact with the ceiling (Figure A.16).

**Figure A.16. Fall off of first layer of plasterboard in Test 2**

The measured average joist temperatures were largely below 175ºC and remained so for the duration of the test (Figure A.17). Therefore, unlike Test 1, it was not necessary to extinguish the fire to prevent fire spread through the floor. It should be noted that there is a small peak in temperature at location C4 where the temperature reached 300°C (Figure A.17). This is likely to be due to a small opening in the plasterboard, allowing hot gases to enter the ceiling void. However, examination of the joists at the end of test showed no evidence of burning in that area and only a minor smoke stain was observed.

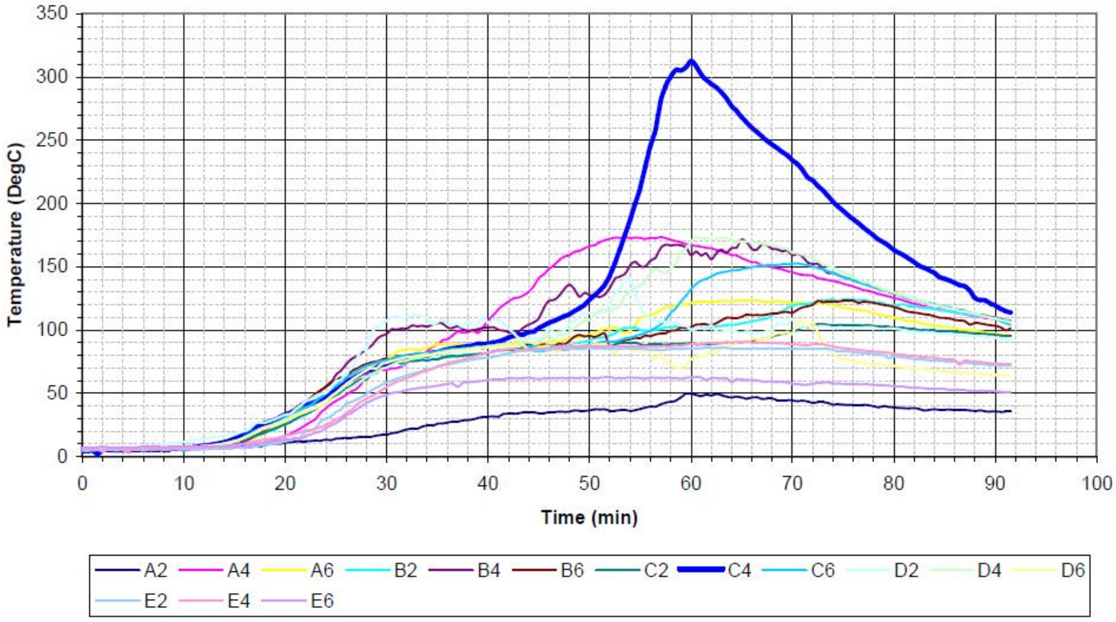


**Figure A.17. Average joist temperatures in Test 2**

Due to the proportionally low temperatures of the joists compared to Test 1, the floor deflected very little at constant rate of approximately 0.2 mm/min. (Figure A.18).

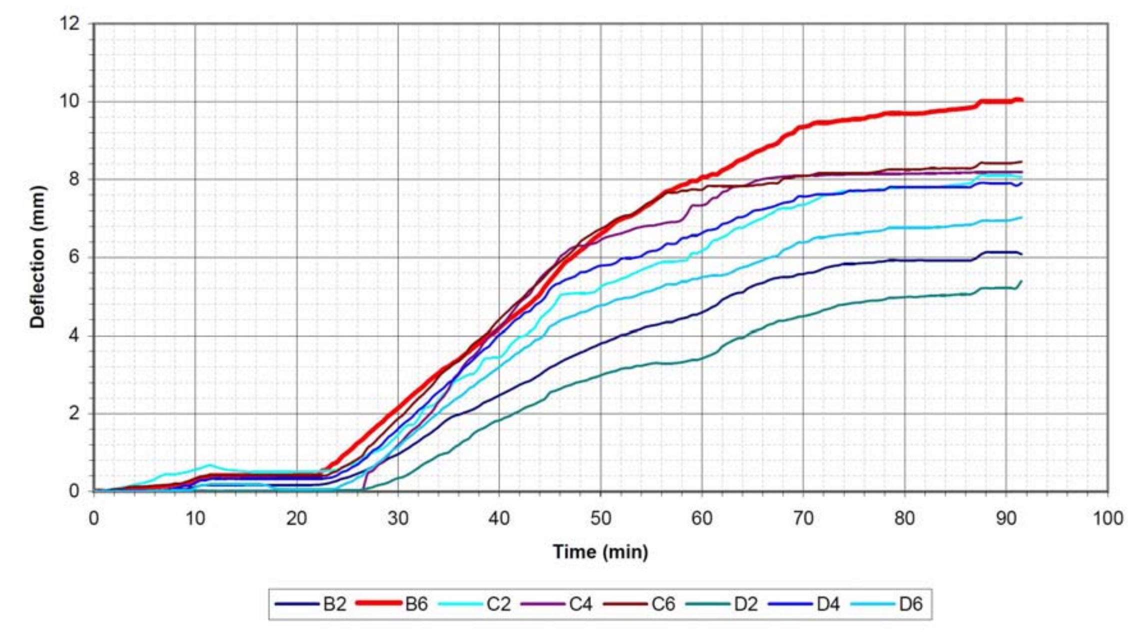


**Figure A.18. Floor deflections at various locations, Test 2**

### 4.3 Test 3 – Engineered truss joists with a steel web

In this instance truss joists with steel webs were tested in the same block-work compartment used in the previous tests. The joists had a larger thickness than that of the previously tested floors. Therefore, new joist hangers were used to accommodate the larger sections. The engineered truss joists were protected with 2 x 12.5 mm of fireline plasterboard to provide for 60 minutes fire resistance. The timber cribs were ignited and the fire was allowed to develop naturally. However, the test had to be terminated after 56 minutes due to significant floor deflections and concerns for its stability. At this point the fire brigade were asked to intervene and extinguish the fire. The compartment temperatures measured during the test are plotted in Figure A.19. The fire development, in terms of peak duration and intensity, is almost identical to that of the earlier tests. In addition, the measured temperatures show good correlation with the compartments design parametric fire curve. However, the cooling phase of the fire has been truncated due to fire brigade intervention, as was the case for Test 1.

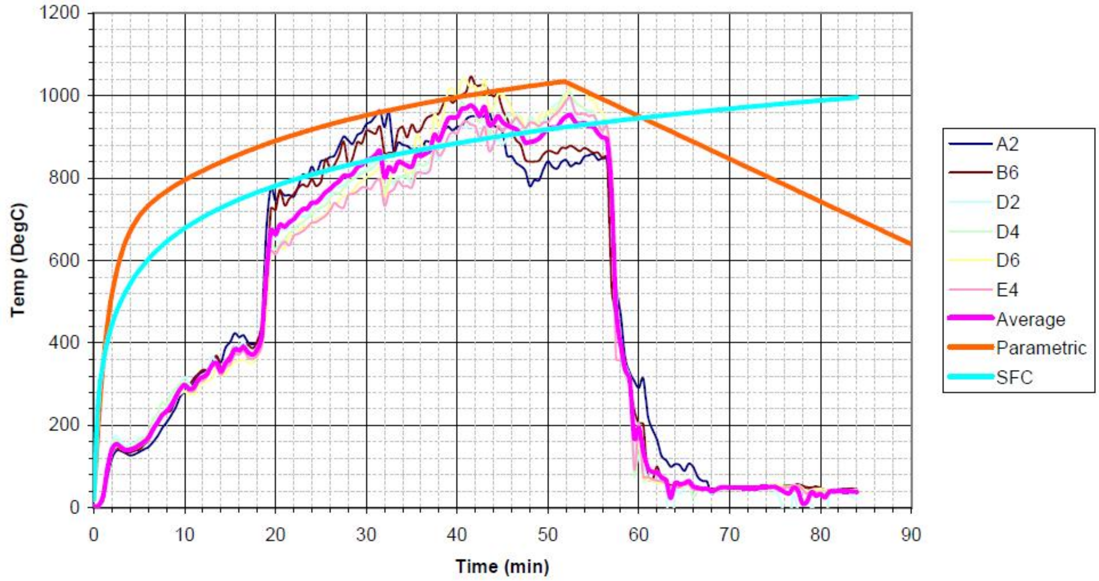


**Figure A.19. Compartment temperatures, Test 3**

Flaming within the ceiling void was extremely severe. This was due to localised plasterboard failures, which allowed flames to spread into the ceiling and ignite the floor joists. Evidence of this is shown in Figure A.20. Once the plasterboard lining had been breached, large deflections local to this area developed and caused further plasterboard breakaway. The evolution of average joist temperatures with time is shown in Figure A.21.

**Figure A.20. Evidence of plasterboard break away on ceiling, Test 3**

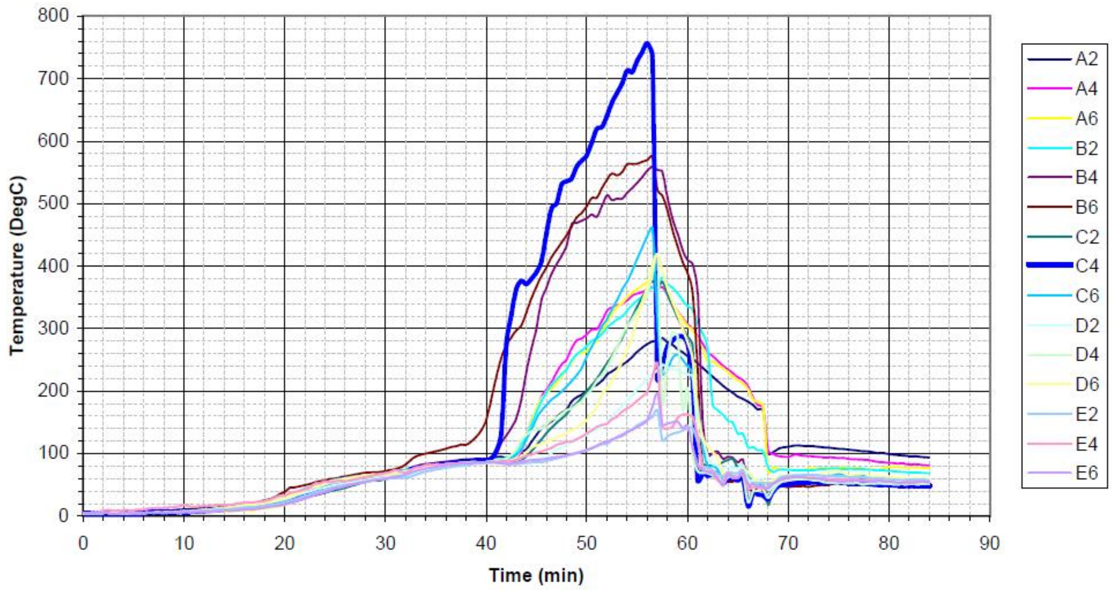


**Figure A.21. Average joist temperatures in Test 3**

In most instances the joist temperatures measured are in excess of the notional ignition temperature of solid timber indicating combustion of the joists. Figure A.22 shows the floor deflections with respect to time. The onset of failure occurs at 46 minutes, correlating with the rapid rise in temperature identified in Figure A.21. This appears to be the time at which large portions of the internal ceiling lining were observed to fall away. The maximum rate of deflection over the period of 46 to 60 minutes of fire exposure was approximated as 6.4 mm/min.

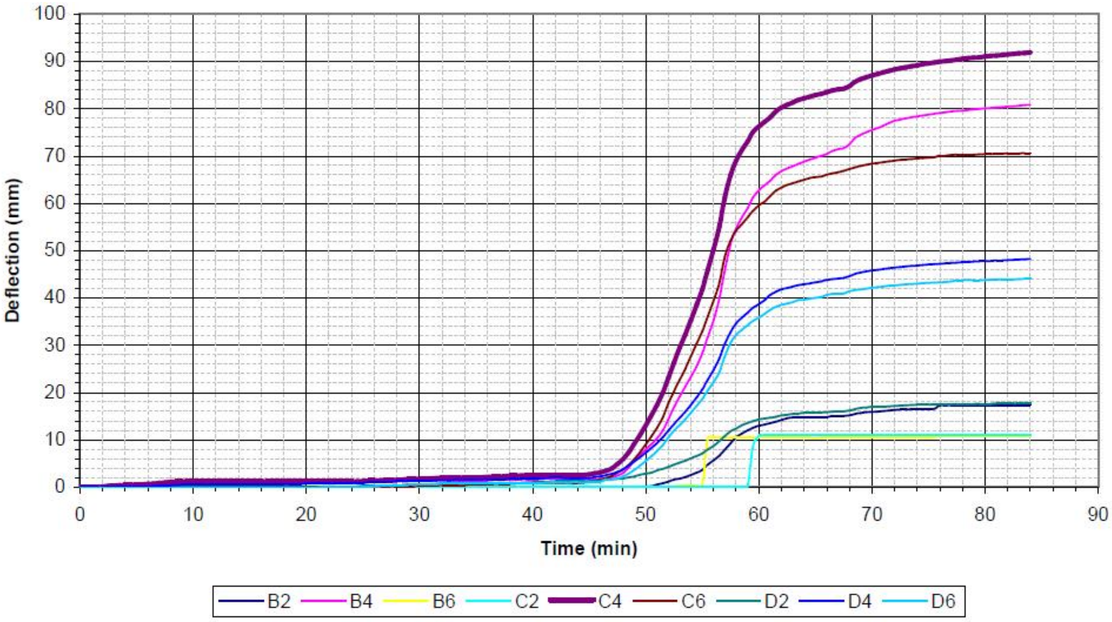


**Figure A.22. Floor deflections, Test 3**

Post-test inspection of the floor system indicated a number of potential factors that may have contributed to the onset of failure. Firstly, significant charring was clearly visible at a number of locations. In some instances a 'residual cross section' of approximately 30 mm × 50 mm was noted representing a loss of around 54% of the section (originally 45 mm × 72 mm). In addition, tearing out of the steel web trusses was also noted. An example of which is shown in Figure A.23.

**Figure A.23- Tearing out of steel web elements**

In some locations the thickness of the charred timber was equal to or in excess of the depth of the mechanical fixings joining the web steel trusses to the solid timber chords. At some locations segments of the steel web were completely detached from the joists and were found on the compartment floor. Clearly, where this occurred, the top and bottom timber flanges were no longer connected.

## 5. Discussion

The paper has presented the findings of three tests on floor joists, two of which fall under the category of ‘engineered joists’. The underlying purpose of this experimental programme was to test the hypothesis that engineered floor joists fail in a more catastrophic manner than traditional solid timber floor joists. Hence, solid timber floor joists have been tested as a comparative baseline, against which the performance of alternative joists can be measured.

### 5.1 Performance of the plasterboard lining

The three floor systems were designed by the manufacturers to achieve 60 min fire resistance. For this purpose, the engineered I-section joists were protected by 2x15 mm plasterboards whilst the solid timber and truss web joists were protected by 2x12.5mm plasterboards. The 30mm plasterboard specification for the I section joists was extremely efficient in protecting the floor. The outer layer of the lining fell away during the test. However, the internal layer remained intact until the fire entered its decay phase. Comparatively 25mm of type F plasterboard was insufficient to protect the solid and truss web joists for 60 minutes in a natural fire. Both layers of the thinner boards fell way during the steady state phase of the fire resulting in significant damage to the joists above.

The performance of the lining to the I section joists meant that the test did not provide enough information to draw a conclusion about the failure behaviour of the floor. Comparatively a large volume of new information has been collected for steel webbed engineered floor joists, and some observations have been made regarding the likely mechanisms which cause failure and the associated rate of deflection, at the point of failure. These are discussed in the sub sections that follow.

## 5.2 Comparison of the deflection behaviour of different joist systems

Figure A.24 shows the relationship between average joist temperature, deflection and time for all three tests, at the point of maximum observed deflection. This provides an opportunity to compare the performance of the two engineered floor systems against solid joists.

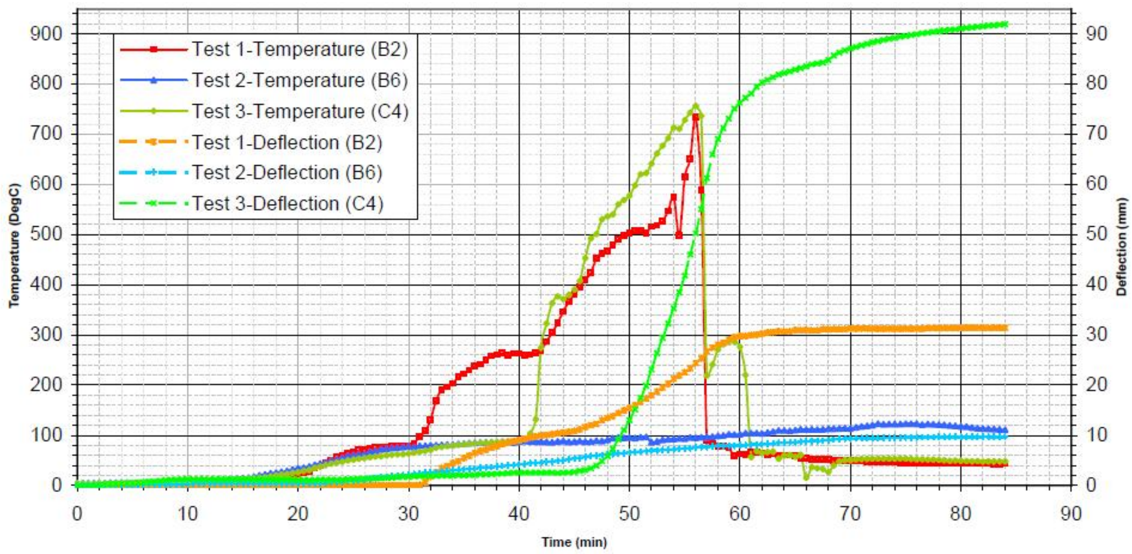


**Figure A.24. Comparisons of data for deflection and average temperature for the points of maximum observed deflection**

Initially, the deflection of the solid timber joists remained insignificant for approximately 30 minutes. After this, the maximum deflection increased at an almost steady rate to reach a maximum value of about 29mm after approximately 56 min of exposure. This is also the time at which the maximum average temperatures in all tests appears to have been reached.

The initial deflections of the two engineered floor systems were of a comparable magnitude with both reaching a value of approximately 6mm after 48minutes of exposure. This is almost half the deflection reached by the solid timber joists after the same time of exposure. After this, the rate of increase of the deflection of the engineered I-section joists remained much lower than that of the solid timber joists. With a maximum deflection of nearly 8mm reached at 56min (and remained unchanged at 60 min). Clearly, the fire protection provided by the 30mm fireline plasterboards used in this test was effective. After the first 48 minutes of fire exposure a sharp increase in the rate of deflection of the engineered truss joists started to take place. After 56 min of fire exposure the maximum deflection was nearly 70mm (75 mm after 60 min). Once the fire had been extinguished the floor continued to deflect reaching a maximum value of 90mm. To give a comparative picture of the maximum deflection of each floor system the deflections measured after 60 min at different locations across the floor are shown in Figure A.25. Where the test was terminated prior to 60 minutes the last measured deflection is shown. Clearly, the deflections of the engineered truss joists are the most significant. In general, for approximately the same joist temperature, it can be seen that the engineered truss joists deflected almost three times more than the traditional solid timber joists, after approximately 60 minutes of fire exposure.

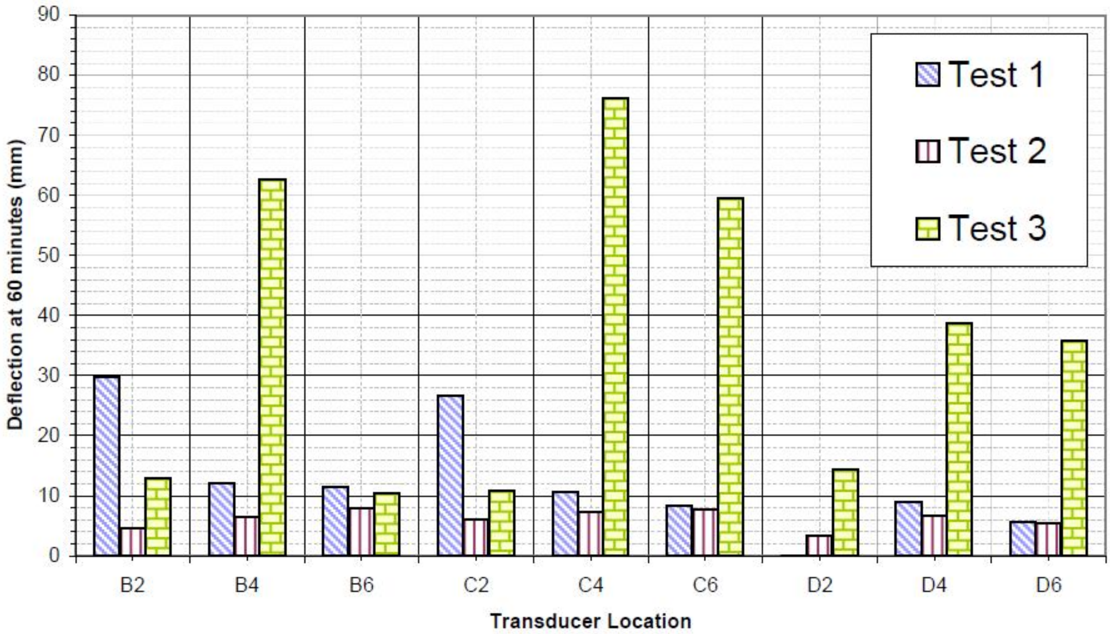


**Figure A.25. Floor deflections at various points at 60 minutes**

### 5.3 Failure modes of fire damaged timber floor joists

The rate at which deflections develop, especially prior to failure, is particularly important as far as safety is concerned and gives a good indication of the type of failure observed i.e. brittle, ductile etc. Since failure did not occur in all floors the maximum rate of deflection observed after an exposure to fire of 30 minutes will be used for comparison. Figure A.24 shows that for the solid timber joists this rate of deflection was nearly 1.0 mm/min, and remained constant over a period of about 30 min. This is a typical behaviour for this type of floor [11] giving reasonable warning before final collapse. In general, collapse occurs when the depth of char is such that the residual solid timber section is insufficient to support the load applied to the floor [12].

The rate of deflection of the engineered I-section joists was approximately 0.2 mm/min. This indicates that the floor was well protected from fire by the 30mm plasterboard, as recommended by the floor manufacturers. However, this also meant the resistance of the floor to direct fire exposure could not be evaluated in this experiment. However, tests on similar floors reported in the literature [2] suggest that, when the lining to the joists is breached then the OSB web of the I-joists may be ‘burnt through’. This may result in a loss of connectivity between the top and bottom flanges, which would be extremely detrimental to the load bearing capacity of the joists and could lead to a very sudden failure of the floor system.

The rate of deflection of the engineered truss joists was approximately 6.4 mm/min. This was accompanied by large deflections which occurred quite suddenly over a short period of time. Clearly, this indicates a more dangerous mode of failure compared to that of solid floor joists. In this test the elapsed time between the point at which deflections within the floor became noticeable and the floor suffering significant damage, was less than 10 min. At this stage a decision was made to terminate the test to prevent the floor from collapsing within the following few minutes. Clearly, this is a very undesirable mode of failure.

In general, failure of the engineered truss joists is likely to occur due to loss of material caused by combustion of the solid timber flanges. In addition, charring of the chords can cause failure of the mechanical fixings joining the steel web to the chords. This may lead to a loss of connectivity between the flanges leading to collapse of the section. However, this mode of failure is likely to be a localised one. This is because the steel web of an engineered truss joist is manufactured with a number of small discrete web modules each connected with its own mechanical fixings to the top and bottom chords. It was apparent that only a number of these modules failed during the test which prevented immediate collapse of the entire floor. It should be noted that the similar mode of failure was also reported in work undertaken in Canada on similar joists tested in a furnace [2].

### 5.4 Additional observations

The contribution of boards, fixed at the top and bottom of the floor joists, to the fire resistance of the floor systems tested in this programme is of great importance. For this to be fully utilised a high quality of workmanship must be maintained when fitting timber floors. Badly fixed internal lining boards are likely to fall off in a relatively short time after flashover. This would expose the joists to direct flames and would ultimately lead to collapse of the floor. Workmanship defects in floors adopting engineered floor joists are a particularly important issue as the time taken to ‘failure’, after direct exposure to the fire, appears to be relatively short. For this particular experimental programme the internal linings to the compartment were fixed by a general construction contractor, strictly complying to the specifications of the manufacturers of the boards [4].

The chipboard sheathing to the floor above the compartment also contributes to the overall fire resistance of the floor. For example, flaming was observed in the ceiling space of both the solid timber joists and the engineered truss joists of this study. However, in neither of these did the fire breach the test compartment or cause damage to the instrumentation above the floor. The chipboard may also contribute to the integrity of the floor system as it may behave like a stress skin at large deflections. However, further experimental and analytical modelling is needed to establish whether this has a significant effect on the survival time of the floor.

Finally, in all tests, the steel hangers supporting the joists did not fail. In addition, there was little, if any, deformation after exposure to fire. It is likely that the ceiling boards provided adequate protection which kept the hangers at a relatively low temperature. In general significant plasterboard fall off local to the joist hangers was not apparent and hence they remained relatively cool. The locations of plasterboard fall off are likely to be a function of the floor deflection which was insignificant at the joist ends.

## 6. Conclusions

Like many systems the performance of floors is currently evaluated via a standard furnace test of limited dimensions with unrealistic boundary conditions. The intention in this project was to investigate the performance of a floor system connected to loadbearing walls through proprietary connections and subject to a typical value of imposed load and a fire scenario that included direct flame impingement. The experimental programme undertaken involved testing three different floor systems for typical residential applications exposed to a realistic fire scenario under realistic conditions of load and restraint.

In each case a system representing a separating/compartment floor was selected such as would be used to separate different occupancies within an apartment building. For this reason the floors required a notional 60 minute fire resistance period. Guidance on the appropriate level of fire protection was taken from manufacturer's information.

The general conclusion that may be drawn from the results of this test programme is that engineered floors may be able to offer the same fire resistance as that of solid timber joists floors provided that the engineered joists are properly protected, from fire, by adequate boarding and that a good quality of installation is maintained during construction. However, when exposed directly to fire, some engineered joists may fail in a more rapid manner when compared to solid timber joists. This was supported by the following observations:

- The performance of the engineered I-section joists shows that this type of floor may be capable of providing 60 min fire resistance to natural fire scenarios provided that two layers of 15 mm fire resistant plasterboard are used, as recommended by the manufacturers. However, more tests are needed to assess the exact behaviour of such joists if exposed directly to fire due, for example, to failure of the lining boards.
- When exposed to fire directly, the behaviour of engineered truss joist floors, similar to the one tested, result in a more rapid mechanism of failure. The test showed that under this condition this type of floor may develop large deflections, and continue to deflect at a high rate over a short period of time leading to a sudden failure of the floor system. This mode of failure was not observed in the solid timber joists test. In this case, the steel modules forming the web of the section were detached due to charring of the timber chords which caused the connecting plate to lose its bond. More tests are needed to determine whether the use of different type of connectors, the provision of thicker plasterboards or a combination of both may improve the performance of the floor.
- The chipboard flooring offers some contribution to the overall fire resistance of the floor system by delaying the spread of fire if the ceiling void is breached. It also may offer additional structural resistance by acting as a stress skin should some of the joists become damaged.

- The joist hangers have been shown to be capable of surviving 60 minutes exposure to a natural compartment fire with little or no damage observed.
- The deflection of the engineered truss joists was almost three times that of the solid timber joists after 60 minutes of fire exposure.

## 7. Acknowledgements

The authors would like to acknowledge Communities and Local Government (CLG) for funding the fire tests research programme. The work reported herein was carried out under contract placed by CLG and any views expressed are not necessarily those of CLG.

In addition the authors would like to thank British Gypsum for kindly supplying the plasterboard for the compartment linings. Finally Mr. Hopkin would like to acknowledge EPSRC for continuing to fund his research work at Loughborough University.

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# APPENDIX B FAILURE MECHANISMS OF STRUCTURAL INSULATED PANEL (SIP) SYSTEMS UNDER NATURAL FIRE CONDITIONS

**Full Reference**

HOPKIN, D.J., LENNON, T., EL-RIMAWI, J. and SILBERSCHMIDT, V., 2010. Failure mechanisms of structural insulated panel (SIP) systems under natural fire conditions, V.R. KODUR and J.M. FRANSSEN, eds. In: Proceedings of the sixth international conference on structures in fire (SiF), 2$^{nd}$-4$^{th}$ June 2010, DEStech pp 520-527.

**Abstract**

Structural Insulated Panels (SIPs) are formed from the lamination of two oriented strand board (OSB) facing plates and a highly insulated polymer based foam, such as expanded polystyrene (EPS) or polyurethane (PUR). The resulting lightweight panels are then used as primary load bearing compression elements for buildings such as domestic dwellings, apartment blocks, schools and hotels.

The regulatory fire performance of SIPs, like many systems, is assessed via a standard fire test. However, it is widely accepted that this is merely a comparative method for determining one product's performance relative to another and hence gives little indication of a component's likely behaviour in a real fire. With this in mind BRE Global, with support from the UK Government, have undertaken a research programme to determine the fire performance of SIPs. The project comprised a programme of laboratory testing on single panels, numerical modelling and four full scale fire tests on two storey SIP structures incorporating engineered joist floors. This paper presents the findings of the large scale experiments.

The aim of this paper is to present the findings of the large scale fire experiments. In summary, it has been found that SIPs systems may be able to meet the performance requirements of the UK Building Regulations. However, combustion of floor joists may lead to excessive deflection accompanied with a large rate of deflection as collapse is approached.

**Keywords**

SIPs, structural insulated panel, fire, timber in fire, engineered floor joists

## 1. Introduction

The fire performance of SIP structures was identified as a major concern of key stakeholders including mortgage lenders, insurers, regulators and the fire and rescue service in a previous scoping study undertaken for the UK Government on Innovative Construction Products and Techniques [2]. As a result the overall aim of the project was to undertake an experimental programme to determine the resistance of a typical Structural Insulated Panel (SIP) system to a realistic fire scenario and to compare the results to the outcome from standard fire tests. Four experiments were completed in BRE's large scale test facility. The experimental work aimed to simulate realistic fires in single (houses) and multi-occupancy (apartment block) dwellings and, as such, were loaded appropriately using sand bags. The programme comprised two experiments on expanded polystyrene (EPS) SIP structures, one with a 30 minute fire resistant plasterboard lining and one with a 60 minute fire resistant plasterboard lining. In addition two experiments were conducted on structures formed from SIPs with polyurethane (PUR) cores, again with 30 and 60 minute fire resistant plasterboard linings respectively.

## 2. The test programme

Four large scale fire tests have been undertaken on structures built from SIP systems and protected from the effects of fire by fire resistant plasterboard linings. The order and configuration of the tests is as shown in Table B.1. The two most common types of insulation material used by the UK SIPS industry (EPS and PUR) have been tested. The test parameters are discussed in greater detail in the sections that follow.

**Table B.1. Experimental programme**

| Test | Design fire resistance period (min) | Core material | Height to underside of floor (m) | Floor area (m) | $1^{st}$ floor loading (kN/m²) | $2^{nd}$ floor loading (kN/m²) |
|---|---|---|---|---|---|---|
| F1 | 60 | EPS | 2.4 | 4 x 3 | 0.75 | 2.25 |
| F2 | 30 | EPS | 2.4 | 4 x 3 | 0.75 | 0.75 |
| F3 | 60 | PUR | 2.4 | 4 x 3 | 0.75 | 2.25 |
| F4 | 30 | PUR | 2.4 | 4 x 3 | 0.75 | 0.75 |

## 2.1. Description of the test compartments

In total four experimental compartments/buildings were constructed by specialist contractors experienced in SIP installations. The buildings were constructed and tested in series of two (EPS then PUR) inside BRE’s large scale test facility in Middlesbrough, UK. The structures were essentially two storey high compartments measuring 4x3m on plan and 4.8m in height (floor to first floor ceiling). The specification for the structures is summarised in Table B.2.

**Table B.2. Test building specification**

| | F1 | F2 | F3 | F4 |
|---|---|---|---|---|
| Wall lining | 30mm type F plasterboard | 15mm type F plasterboard | 30mm type F plasterboard | 15mm type F plasterboard |
| Party wall | See specification for 60 minute wall linings. | | | |
| Ceiling lining | 30mm type F plasterboard | 15mm type F plasterboard | 30mm type F plasterboard | 15mm type F plasterboard |
| SIP construction | 11mm OSB either side of 89mm EPS core apart from the two load-bearing walls which have a 140mm core due to the increased load level. | 11mm OSB either side of 89mm EPS core. | 15mm OSB either side of 114mm PUR core. | |
| First floor timber | 220mm x 58mm engineered floor joists (I section) @ 600mm centres – span in long direction. | | 245mm x 45mm engineered floor joists (I section) @ 600mm centres – span in long direction. | |
| Second floor timber | 220mm x 89mm engineered floor joists (I section) @ 400mm centres – span in long direction. | 220mm x 58mm engineered floor joists (I section) @ 600mm centres – span in long direction. | 245mm x 45mm engineered floor joists (I section) @ 400mm centres – span in short direction. | |

## 2.2. Fire load, imposed loading and test initiation

The purpose of the experiments was to evaluate the response of the SIP systems to a "realistic" fire scenario such as may occur in a room within a modern apartment building or dwelling. The dimensions and ventilation condition within the room of origin were consistent with a fire within the living area of an apartment building or house where the door is closed. They are also consistent with previous research into the performance of timber frame structures in fire [3]. In terms of the fire loading, it was determined that, based on the compartment geometry and available ventilation, a fire load density of 450 MJ/m2 was required to achieve a fire of 60 minutes ISO834 equivalent. The equivalent severity was calculated using the time equivalence method of BSEN 1991-1-2 [4] taking into account the number and size of openings and floor area. In addition the predicted compartment time-temperature response was calculated according to the parametric approach detailed in annex A of BS EN 1991-1-2 [4]. The design fire load was provided by twelve cribs each with 25 kg of solid timber, giving a total fire load of 5400 MJ for each experiment.

The 30 minute solution (F2 and F4) effectively modelled a two- storey domestic dwelling. The resulting applied load on the floor under test was 0.75 kN/m² per floor. The load was applied by sandbags weighing 25kg per bag resulting in 37 bags per floor.

The 60 minute solution (F1 and F3) modelled a four-storey building and the loading in the lower wall panels needed to reflect this. The second floor was loaded to a value of 3 x 0.75 x 4 x 3 = 27kN or 2752kg made up from 110 sandbags (equivalent of 3 levels of imposed load). The design and spacing of the floor joists for the second floor reflected this increased load. The first floor was loaded in an identical manner to the 30 minute case.

The fire load was composed of 12 timber cribs spread uniformly over the floor area each consisting of 50 sticks of 50mm x 50mm x 500mm long softwood. Each crib was connected to the adjacent crib by means of a porous fibre strip. Prior to ignition approximately 1 litre of paraffin was poured over the fibre strips. The fire was ignited by setting light to each of the fibre strips.

## 3. Results

Experiments F1 and F3 were terminated once the fire was around half way into its cooling phase. Experiments F2 and F4 were terminated at the peak of the compartment temperature development due to imminent collapse of the floor system. Termination times are summarised with other relevant information in Table B.3. In the inspection of the results discussed below Figure B.1. should be referenced.

**Table B.3. Summary of results**

| Experiment | Termination time (min) | Peak floor temp (°C) | Peak floor deflection (mm) |
|---|---|---|---|
| F1 | 75 | 200 | 10 |
| F2 | 50 | 897 | 203 |
| F3 | 87 | 237 | 16 |
| F4 | 52 | 664 | 121 |

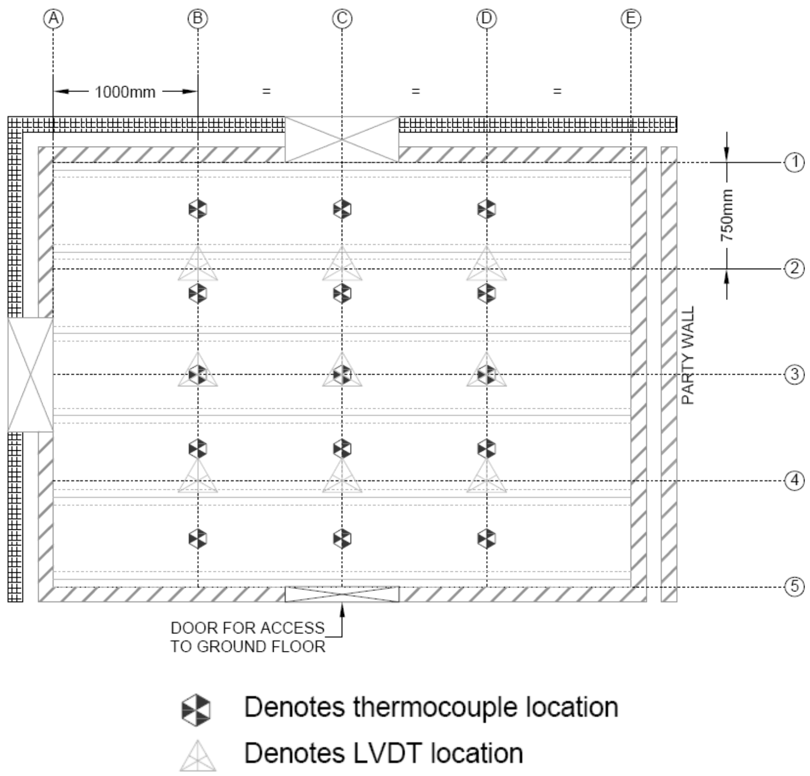


**Figure B.1. Instrumentation locations**

## 3.1. Fire development

The average measured compartment time-temperature curves for the four tests are shown in Figure B.2. The figure also includes the standard fire curve and the predicted response according to the Eurocode parametric approach [4]. Generally the agreement between the measured and parametric prediction of the compartment temperature is very good. However the parametric approach is a little un-conservative in the prediction of peak compartment temperatures. It also under predicts the duration of the heating phase of the fire where the experiments have been allowed to enter the cooling phase.

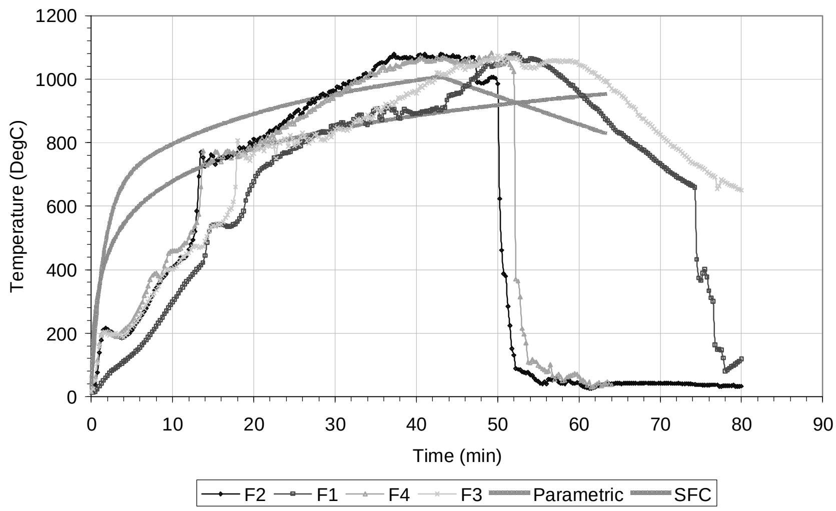


**Figure B.2. Average compartment temperatures with design parametric and standard fire curves**

### 3.2. Structural performance

The mid-span deflection of the joists spanning above the fire compartment for each experiment are shown in Figure B.3. Quarter point deflections for experiments F1 and F3 were also measured (Grid lines B and D) however these have been omitted for clarity. The clear difference between floors protected with 15mm type F plasterboard (F2 & F4) and 30mm type F plasterboard (F1 and F3) is apparent in Figure B.3. The displacements in the former tests are an order of magnitude larger than the systems protected with 30mm plasterboard. The additional deflection associated with experiments F2 and F4 can be attributed to the high temperatures noted in the floor space, as shown in Figures B.4. and B.5.

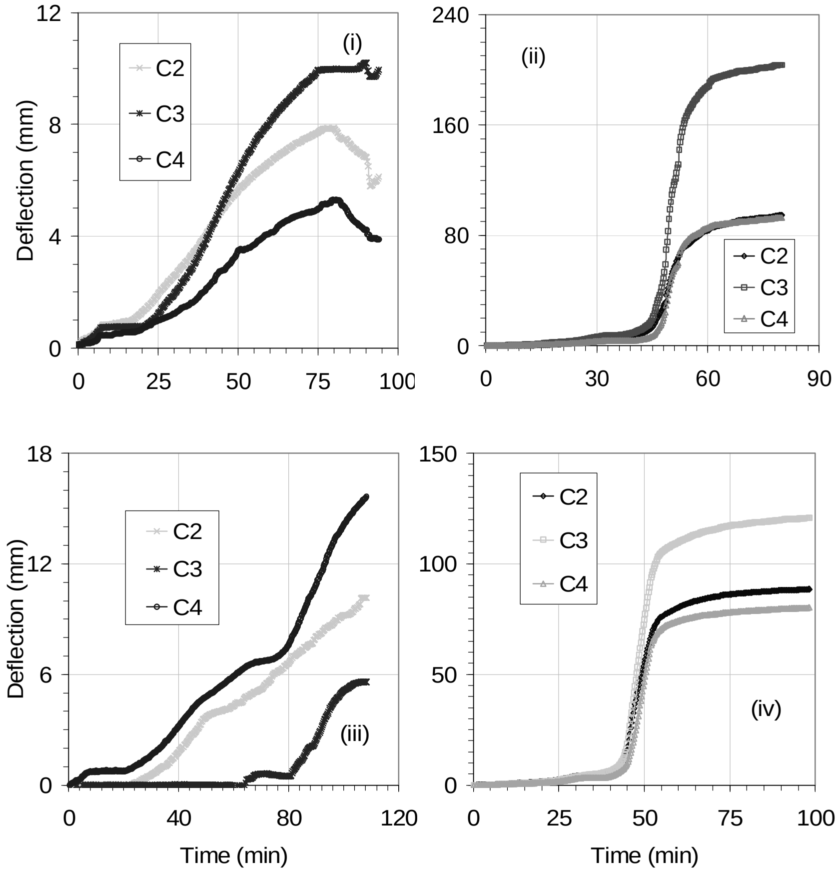


**Figure B.3. Floor deflections along grid line C for (i) F1, (ii) F2, (iii) F3 & (iv) F4**

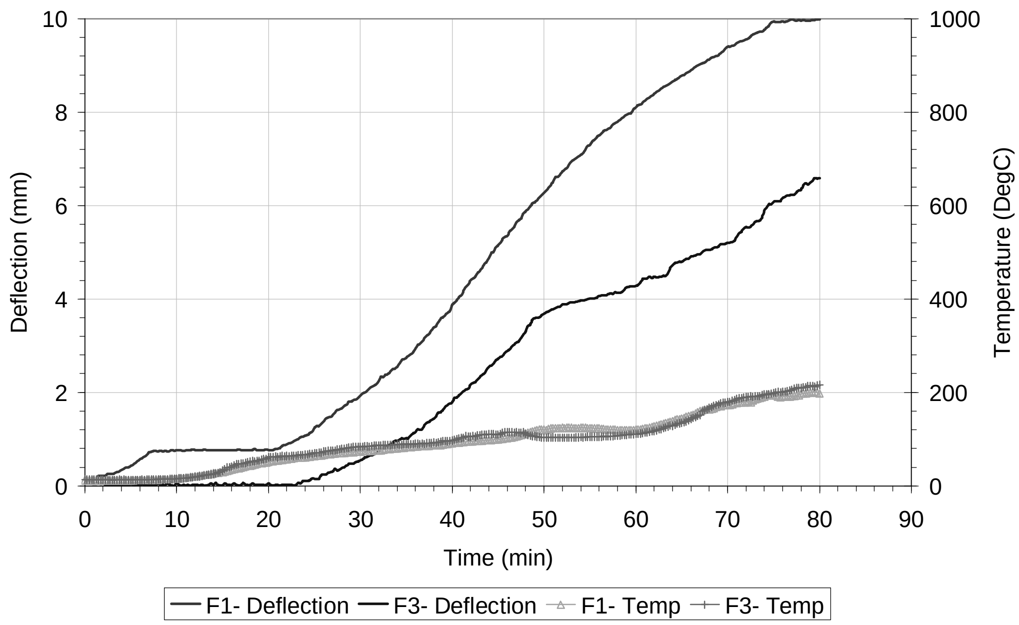


**Figure B.4. Displacement versus temperature for floor void- 60 minute buildings**

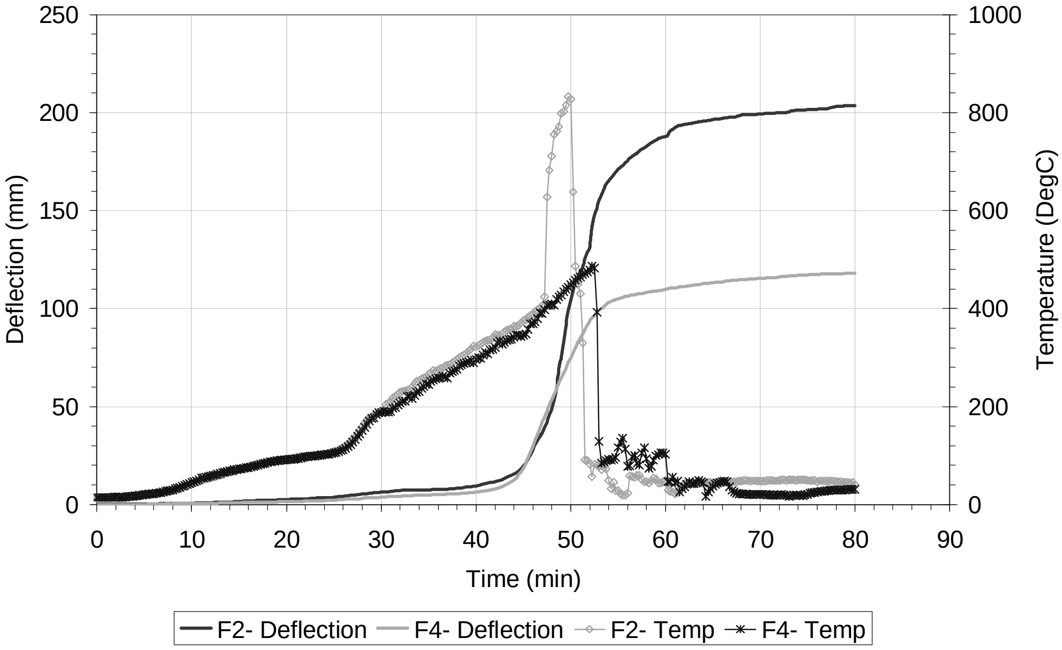


**Figure B.5. Displacement-temperature for floor void – 30 minute buildings**

4. Post test observations

The test results and observations have highlighted a number of important issues in relation to the inherent fire resistance of the structural system:

- SIP systems are capable of achieving the requirements of the performance criteria in Approved Document B to the UK Building Regulations [5] in relation to B2 internal fire spread (linings) and B3 internal fire spread (structure).

- The mode of failure of the system is excessive deflection of the first floor caused by ignition and rapid combustion of the engineered floor joists. The rate of deflection increases very rapidly as the floor system approaches collapse. This behaviour is not influenced by the performance of the SIP system and would be the same for other panellised systems, traditionally built timber frame or joists supported on masonry walls [6].

- There was no collapse of the floor in any of the tests despite the significant (>200mm or span/20) deflections. The chipboard flooring appears to have contributed to the stability of the floor at large deflections. Inevitably, in the instance of both the PUR 30 and EPS 30 compartments, the floors would have collapsed had the fire and rescue service not intervened. There was no collapse of the wall panels in any of the tests. There was also no obvious deflection or deformation of the wall members.

- There was no integrity failure of either the wall panels or the floor system.

- At the end of the tests, the composite action assumed in design can no longer be relied on due to either degradation of the inner layer of OSB and melting of the core (EPS) or degradation of the OSB and combustion of the core (PUR). As there was no collapse of the buildings, it is clear that an alternative load path was mobilised at the fire limit state. Load carrying capacity was maintained through the solid timber ring beams at first floor level and the presence of intermediate timber in the panels either at junctions between panels or around openings and the presence of timber studs in the corner.

- There was no significant damage to the ring beam in any of the tests.

- There was no evidence of any failures in the connections between the engineered floor joists and the timber ring beams.

## 5. Acknowledgements

The authors would like to acknowledge CLG for funding this programme of work. The work reported herein was carried out under a contract placed by CLG and any views expressed are not necessarily those of CLG. The authors also acknowledge the contribution of the UK SIPs Association. Mr. Hopkin would like to thank EPSRC for continuing to fund his research work with Loughborough University. Finally, the authors would like to thank La Farge for their support with the experimental programme.

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# APPENDIX C A NUMERICAL STUDY OF GYPSUM PLASTERBOARD BEHAVIOUR UNDER STANDARD AND NATURAL FIRE CONDITIONS

**Full Reference**

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., 2011. A numerical study of gypsum plasterboard behaviour under standard and natural fire conditions. *Fire and materials*. DOI: 10.1002/fam.

**Abstract**

Gypsum plasterboards are the most widely used passive fire protection for timber structures, especially in the case of light timber frame construction. Understanding the complex thermo-physical behaviour of plasterboard at elevated temperature is vital in the performance based design of any structure adopting gypsum as passive fire protection (PFP). Numerous heat transfer studies have been conducted over the years where attempts have been made to simulate the fire performance of gypsum protected assemblies, subject to standard fire exposure. However, contradictory thermal properties for gypsum plasterboard are apparent throughout. As a result it is unclear from a practitioner's perspective as to which studies represent reasonable properties for design purposes. In recognition of this the authors present a numerical study highlighting the consequences of adopting many of the differing property sets available in the literature, the sensitivity of temperature development resulting from deviations from the assumptions that underpin such properties, and the consequences of adopting plasterboard properties derived from standard fire tests, in natural fire situations. The study presents heat transfer simulations conducted using the finite element software TNO DIANA coupled with both laboratory and natural fire tests conducted on Structural Insulated Panels (SIPs) and Engineered Floor Joists (EFJs).

It is found from this study that plasterboard properties are highly sensitive to the assumed free and chemically bound moisture contents. Minor percentage changes are shown to have a significant influence on the temperature development of SIPs exposed to standard furnace fires, whilst some of the most accepted plasterboard properties available in the literature are found, in some cases, to be non-conservative when adopted in simulations of SIPs. More interestingly it is also found that the properties of plasterboard available in the literature, largely derived from standard fire tests, are not independent of heating rate. As a result when such properties are applied to natural fire problems significant inaccuracies can occur.

**Key words**

Gypsum plasterboard, timber, fire, structural fire engineering

## 1. Introduction

Gypsum plasterboards are the most widely used passive fire protection for timber structures, especially in the case of light timber frame construction. Studs and joists are typically protected with gypsum plasterboard of thickness appropriate for a given fire-resistance application.

Plasterboard is formed by pressing gypsum between two paper facing sheets. In Europe (including the UK) plasterboard is graded into type A plasterboard [1], or Wallboard, which refers to standard gypsum core board, and type F plasterboard [1], or fire-resistant board, which refers to board with enhanced core adhesion performance at elevated temperature. The improved performance in the latter case is typically achieved through the introduction of glass fibres or minerals, such as bentonite or vermiculite, which expand and prevent cracking at high temperature. Similar grading systems exist in the USA and Japan for standard wall board and fire-resistant plasterboard.

The basis of the fire-resistance characteristics of gypsum plasterboard stem from the fact that it contains both free and chemically bound water. Upon heating gypsum (which is calcium sulphate di-hydrate) undergoes two main chemical decomposition reactions. The first results in the breakdown of gypsum to calcium sulphate hemihydrate. The second reaction breaks down the calcium sulphate hemihydrate to calcium sulphate anhydrite [2]. Through both reactions water of crystallisation is released (approx. 21% of plasterboard by weight), and vast amounts of energy are required for it to evaporate before the temperature of the gypsum can rapidly increase upon heating. The temperature and energy required to mobilise such reactions have been studied widely; however, no agreed answers have been given in the reported literature. As a result, determining the properties of gypsum is still an area of research interest. During the aforementioned reactions, the energy assumed to be required to evaporate the resulting released water, and the temperatures at which the reactions occur affect significantly the thermal properties of gypsum, specifically its conductivity, specific heat and mass loss rate.

## 2. Thermo-physical characteristics of gypsum board

The heat transfer characteristics of plasterboard have been the subject of several studies over the last 20 years. A number of widely used temperature-dependent conductivity and specific heat properties exist, which have been accurately implemented in the simulation of the behaviour of timber and steel stud walls [2-5]. The most common gypsum properties adopted in such numerical models are shown in the Figures C.1. to C.3. They are based on studies reported by Harmathy [6], Anderson [7], Mehaffey, *et al.* [3], and Sultan [4]. Other, less commonly used properties were derived by Benichou, *et al.* [8], Thomas [2] based on the work of Mehaffey, *et al.* [3], Ang and Wang [9] based on the work of Thomas [2], Park, *et al.* [10], Wakili, *et al.* [11], and a further investigation by Thomas [12]. These are also included in Figures C.1. to C.3. for completeness.

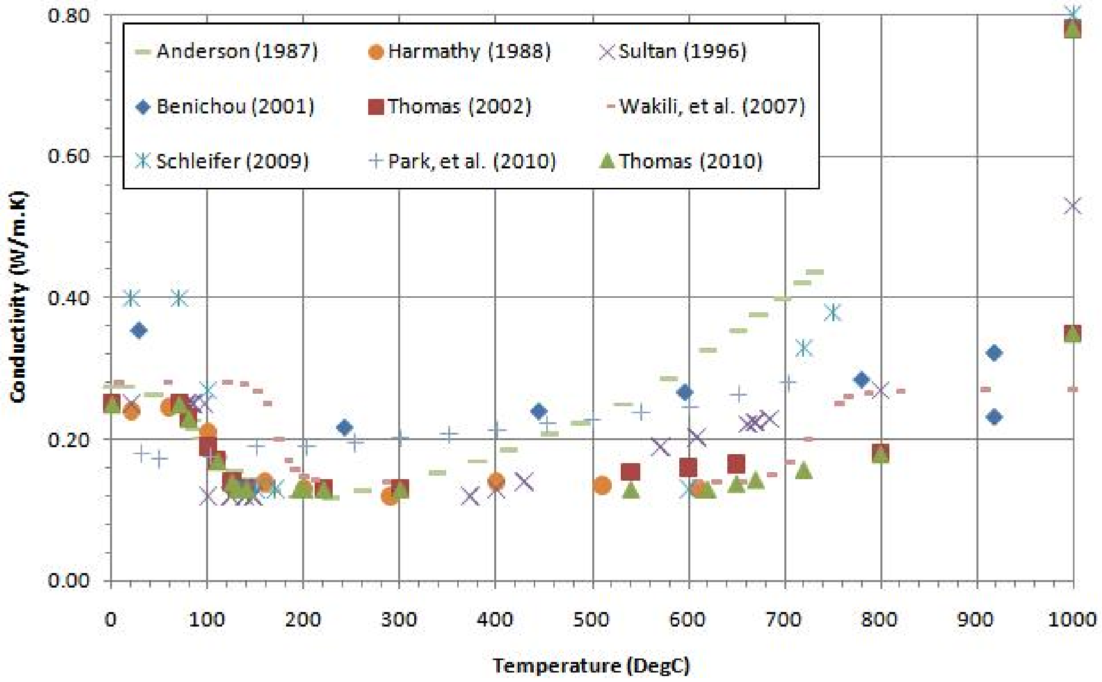
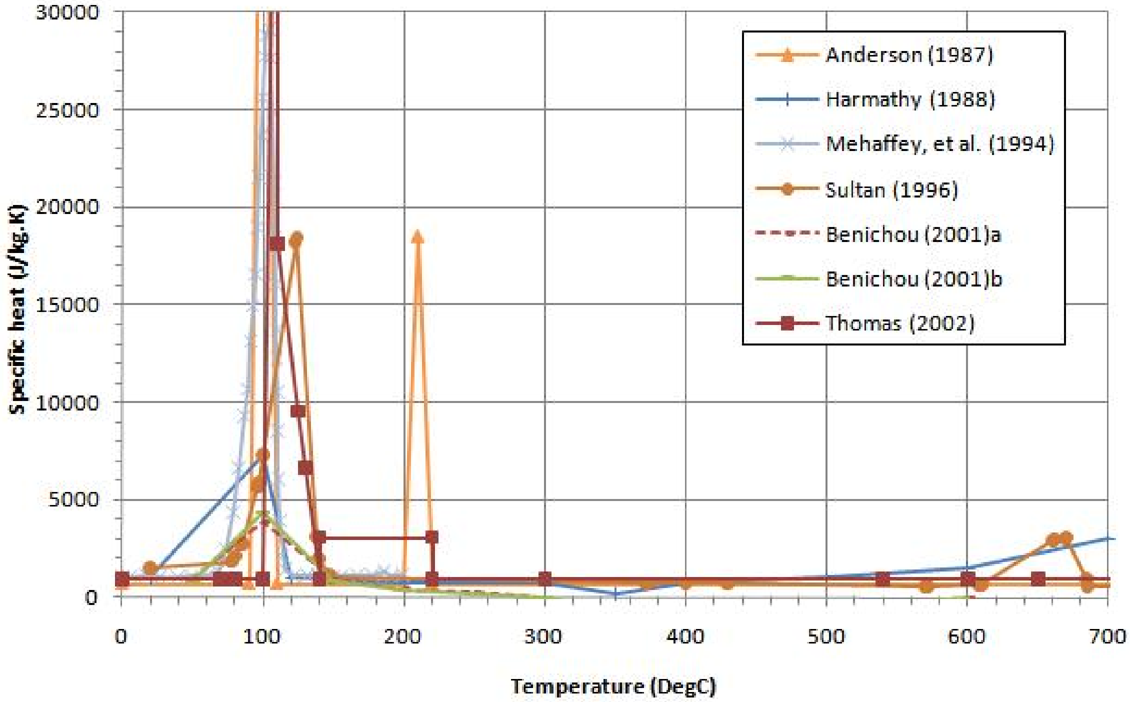



**Figure C.1. Comparison of temperature-dependant conductivity of gypsum plasterboard**



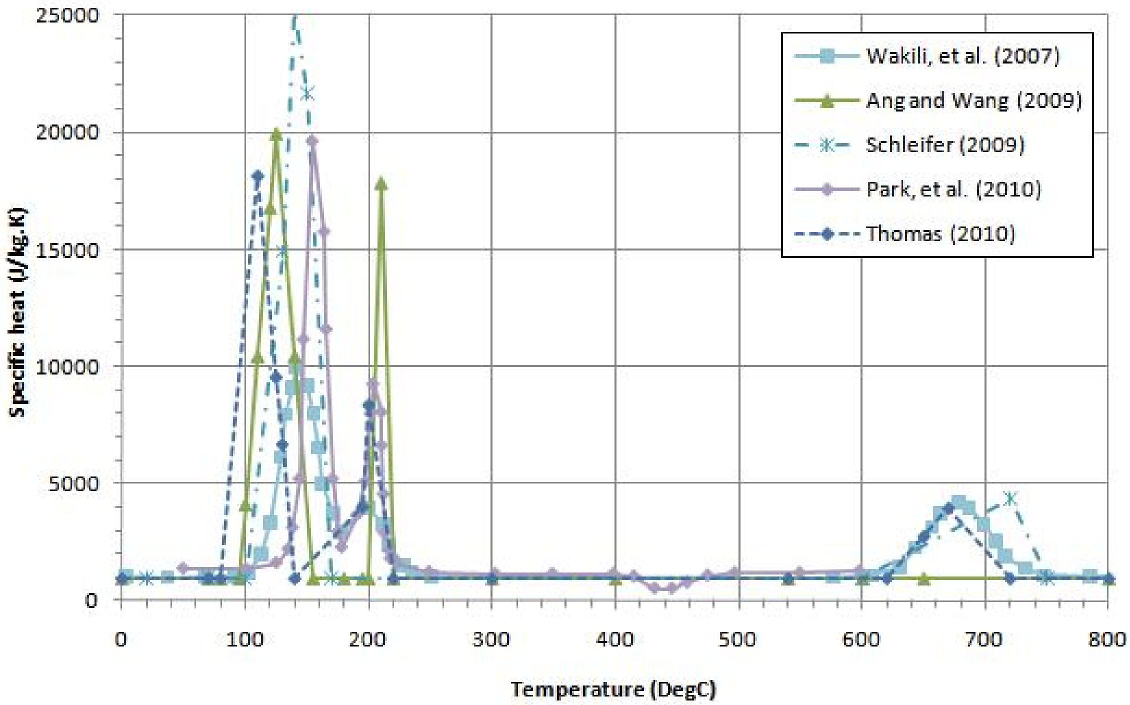




**Figure C.2. Comparison of temperature-dependant specific heat of gypsum plasterboard**

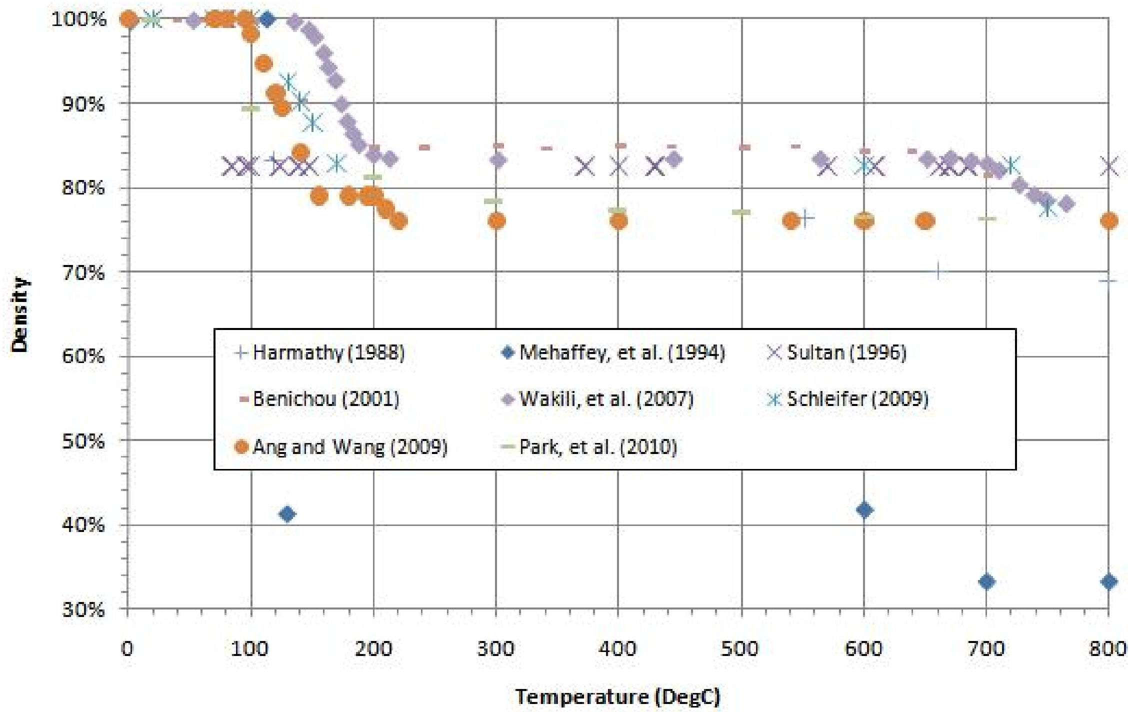



**Figure C.3. Comparison of temperature-dependant mass loss rate of plasterboard**

The study of Ang and Wang [9] is worth highlighting as it gives a mathematical formula for the specific heat of gypsum as a function of its moisture content. This was derived assuming that plasterboards undergo two de-hydration reactions. The first typically occurs within a temperature range of 80 to 120°C, whereby 75% of the chemically bound moisture content, quoted as 21% by weight by Thomas [2], and all of the free moisture content, similarly quoted by Thomas [2] as 3% by weight, are released and evaporated. In the second dehydration reaction, assumed to occur between the temperatures 200 to 240°C, the remaining 25% of chemically bound water of crystallisation is also released and evaporated. Ang and Wang [9] used these ratios, together with data reported by Thomas [2] relating to bond breaking energy, the latent heat of evaporation of water and numerical calibrations, to propose a mathematical model for determining the specific heat of gypsum plasterboard. In their model a “base” specific heat for dry gypsum, taken as 950 kJ/kg K, was assumed after Mehaffey, *et al.* [3]. Transposed upon these are two specific heat ‘spikes’ corresponding to the two dehydration reactions occurring over the temperature ranges suggested. This "additional specific heat" additive concept is shown graphically in Figure C.4.

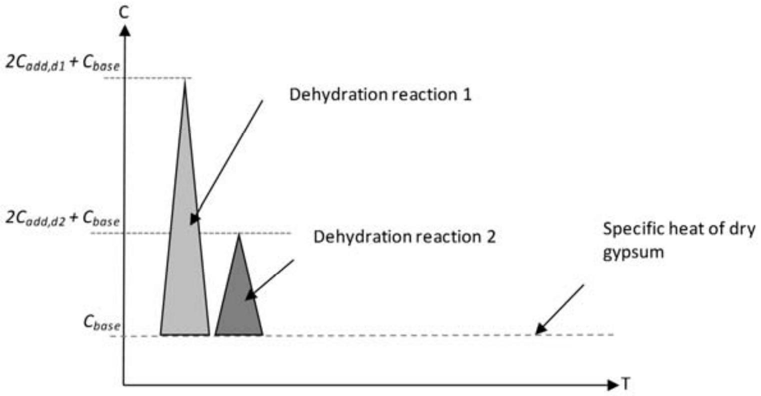



**Figure C.4. Additive specific heat concept [9]**

The expressions for the additional specific heat ($c_{add}$) associated with each of the two reactions are given by Equations C.1. and C.2. respectively:

$$c_{add,d1} = \frac{E_{d1} + \left(e_t e_{d1} + e_f\right)L_w}{\Delta T_{d1}} f_2 \qquad \textbf{[Equation C.1]}$$

$$c_{add,d2} = \frac{E_{d2} + \left(e_t e_{d2}\right)L_w}{\Delta T_{d2}} f_2 \qquad \textbf{[Equation C.2]}$$

Where:

- $c_{add,dn}$ is the additional specific heat associated with the n$^{th}$ dehydration reaction [J/kg K];
- $E_{dn}$ is the energy required for molecular bonds to be broken so that the water of crystallisation for each reaction can be released, typically 100 kJ/kg for reaction 1 and 50 kJ/kg for reaction 2;
- $e_t$ is a total chemically bound water content of gypsum by weight, typically 21% (or 0.21);

- $e_{dn}$ is a proportion of total chemically bound water content, which is released in each reaction, typically 75% (or 0.75) in reaction 1 and 25% (or 0.25) in reaction 2;

- $e_f$ is a total free moisture content by weight, typically 5% (or 0.05);

- $L_w$ is a latent heat of evaporation of water at 100°C [2260 kJ/kg];

- $\Delta T_{dn}$ is a temperature interval over which the dehydration reactions occur, typically 95 to 155°C for reaction 1 ($\Delta T_1$=60°C) and 200 to 220°C for reaction 2 ($\Delta T_2$=20°C)];

- $f_2$ is a dimensionless empirical factor to account for vaporisation and re-condensation of water in gypsum (1.8 according to Ang and Wang [9]).

Typically, the additional specific heat ($c_{add}$) is assumed to increase linearly up to the mid-point of the temperature range before decreasing linearly back down to the base value. The integral, or the area enclosed by each spike, corresponds to Ang and Wang's [9] additional specific heat ($c_{add,dn}$) as a result of the dehydration processes, giving an overall peak height of $2c_{add,dn}$ (see Figure C.5.).

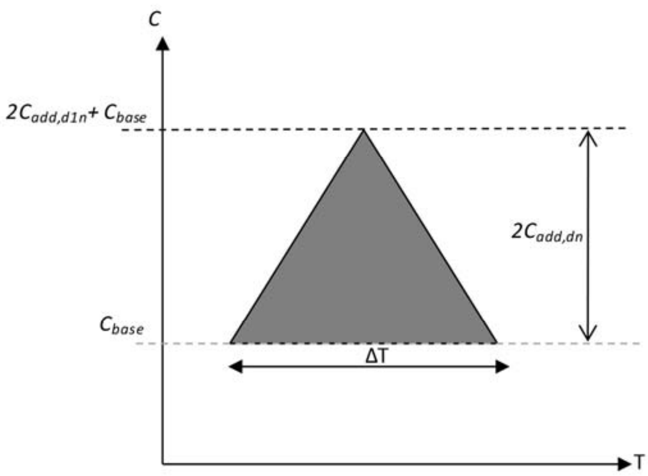



**Figure C.5. Additive specific heat concept: peak-specific heat (after Ang and Wang [9])**

In a similar manner, using the loss of moisture by weight, the mass loss rate of gypsum as a function of temperature can also be derived for various moisture contents. Assuming no dimensional change as a result of the moisture loss, this relationship can also be used to describe the density (ρ) loss rate. The density fraction as a function of temperature is simply determined according to Equation C.3 assuming a linear decrease in density over the temperature range of each dehydration reaction. This decrease corresponds to the amount of moisture by weight lost in each reaction as defined previously.

**For $T<T_{d1,a}$** $$r = r_0$$ **[Equation C.3]**

**For $T_{d1,a} \leq T \leq T_{d1,b}$** $$r = \left(\frac{\left(-e_f - e_t e_{d1}\right) r_0}{\Delta T_{d1}}\right)\left(T - T_{d1,a}\right) + r_0$$

**For $T_{d1,b} < T < T_{d2,a}$** $$r = \left(1 - e_f - e_t e_{d1}\right) r_0$$

**For $T_{d2,a} \leq T \leq T_{d2,b}$** $$r = \left(\frac{-e_t e_{d2} r_0}{\Delta T_{d2}}\right)\left(T - T_{d2,a}\right) + \left(1 - e_f - e_t e_{d1}\right) r_0$$

**For $T>T_{d2,b}$** $$r = \left(1 - e_f - e_t\right) r_0$$

where:

- $\rho_0$ is ambient-temperature density [kg/m$^3$];
- $T_{dn,a}$ is a starting temperature of dehydration reaction n [°C];
- $T_{dn,b}$ is an end temperature of dehydration reaction n [°C];
- $\Delta T_{dn} = T_{dn,b} - T_{dn,a}$ [°C].

Assuming that the conductivity properties of plasterboard are unaffected by moisture content, the above relationships derived by Ang and Wang [9] are rather useful as they can be used in sensitivity studies to determine the impact of assumed moisture content (both free and chemical) on the heat transfer characteristics of gypsum plasterboard. The impact of moisture content on the specific heat of plasterboard is shown in Figure C.6. The cases are defined in Table C.1.

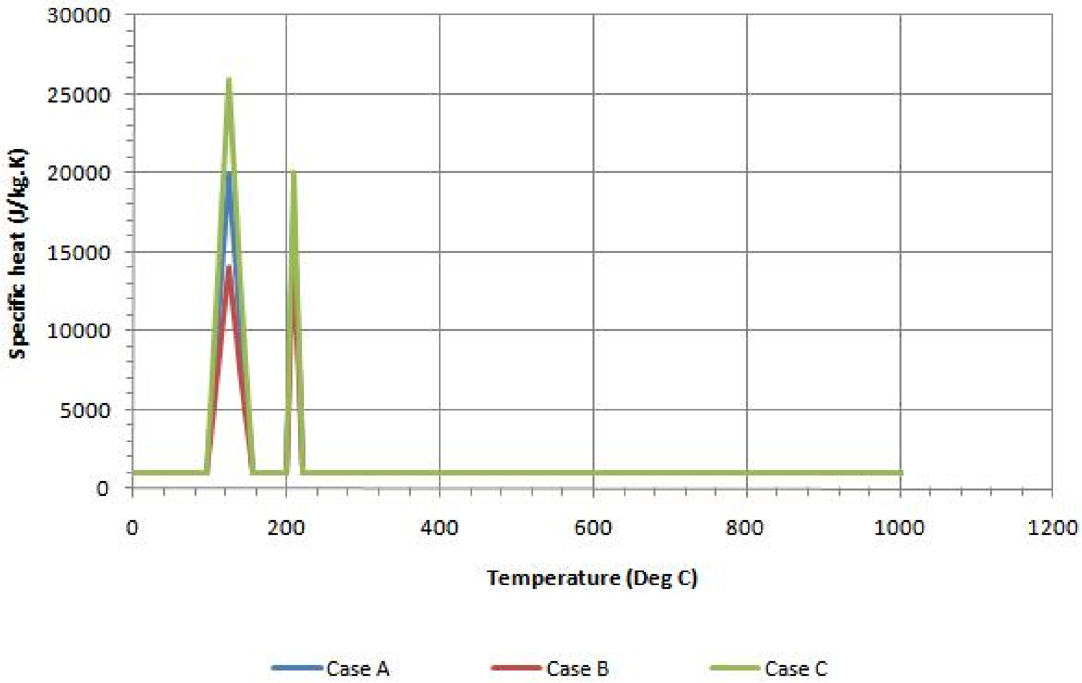



**Figure C.6. Impact of moisture content on specific heat of plasterboard (derived from Ang and Wang [9])**

**Table C.1. Parameters of moisture content in case studies**

| Case | Moisture content (%) | |
|---|---|---|
| | chemically integrated | free |
| A | 21 | 5 |
| B | 16 | 1 |
| C | 25 | 10 |

Generally, the majority of the studies reported in the literature relating to gypsum plasterboard, with some exceptions such as the work by Benichou, *et al*. [8] and Thomas [2], are limited to fire-rated gypsum products. This is largely because type X/F or similar fire-rated boards have better core stability at elevated temperatures and do not break down to the same extent as standard gypsum plasterboard. Hence, in a modelling exercise the geometry should not be treated in a dynamic way whereby the formation of cracks leads to plasterboard fall-off with increasing temperature or the lining thickness depletes due to ablation.

Most properties, presented in the supporting figures, are ‘apparent’ values, derived empirically and, hence, breakdown phenomena such as ablation and the formation of small cracks are included implicitly in the values presented. In addition to this, as eluded to previously, the relationship between the mass loss rate and temperature is commonly assumed to describe the density as a function of temperature. This implies that there is no dimensional change in the thickness of the plasterboard due to fire exposure, which is a valid simplification provided that ablation effects are inherent in the ‘apparent’ material properties adopted.

## 3. Impact of plasterboard properties on temperature development in structural insulated panels

A review of the available literature shows that several thermo-physical data sets exist for gypsum plasterboards. These are derived from a variety of mathematical constructs, based on the associated energy required for dehydration reactions, and from experimental studies. The specific heat functions suggested by different researchers have been shown to be very variable. Clearly, the adoption of a specific dataset will result in different levels of temperature development in gypsum-protected assemblies. To assess the effect of this, simulations of temperature development in structural insulated panels (SIPs) have been conducted adopting different thermo-physical properties for plasterboard. The results of these simulations are benchmarked against heat transfer experiments conducted at BRE Global on isolated SIPs [13] exposed to ISO834 [14] heating conditions.

### 3.1. Modelling approach

The commercial finite element package DIANA [15] has been adopted to simulate the behaviour of SIPs under fire conditions. The chosen panel geometry and sizing coincides with those of the panels tested as part of a research project funded by the UK Department for Communities and Local Government  (CLG) titled "Performance in Fire of Structural Insulated Panels", for which supporting experimental data exists [13, 16-17]. The overall dimensions of the panels tested by BRE Global were 1800 mm x 1200 mm x 150 mm. The panels had 15 mm oriented strand board (OSB) skins giving an insulation depth of 120 mm. The SIPs were protected with type F (fire-resistant) plasterboard [1] of either 15 mm or 30 mm thickness depending on the required survival time to fire exposure  - 30 or 60 minutes. This was fixed on vertical timber battens (50 mm x 25 mm) spaced at 600 mm centres. Both polyurethane (PUR) and expanded polystyrene (EPS) core panels were tested under direct fire exposure, in a small-scale furnace, following the ISO834 [14] temperature regime and controlled using a plate thermometer. The tests were either 30 or 60 minutes in duration depending on the specification of the lining. Temperatures were measured with type K thermo-couples, which were inserted through the holes drilled from the rear of the panels. The thermo-couples were inserted at varying depths according to desired measurement locations. A full instrumentation specification can be found in another paper [13].

In respective simulations, a 2D mesh, representing one half of the panel geometry tested by BRE Global [13, 17], was developed using first-order quadrilateral heat flow elements (DIANA element Q4HT). Linear boundary elements (DIANA element B2HT) were placed at solid-to-gas interfaces, where the calculation of external fluxes was requireC. This included the boundary that enclosed a convex fully-closed air void, formed between the SIP, the plasterboard lining, and the timber battens. The resulting mesh, complete with assigned material properties, is shown in Figure C.7.

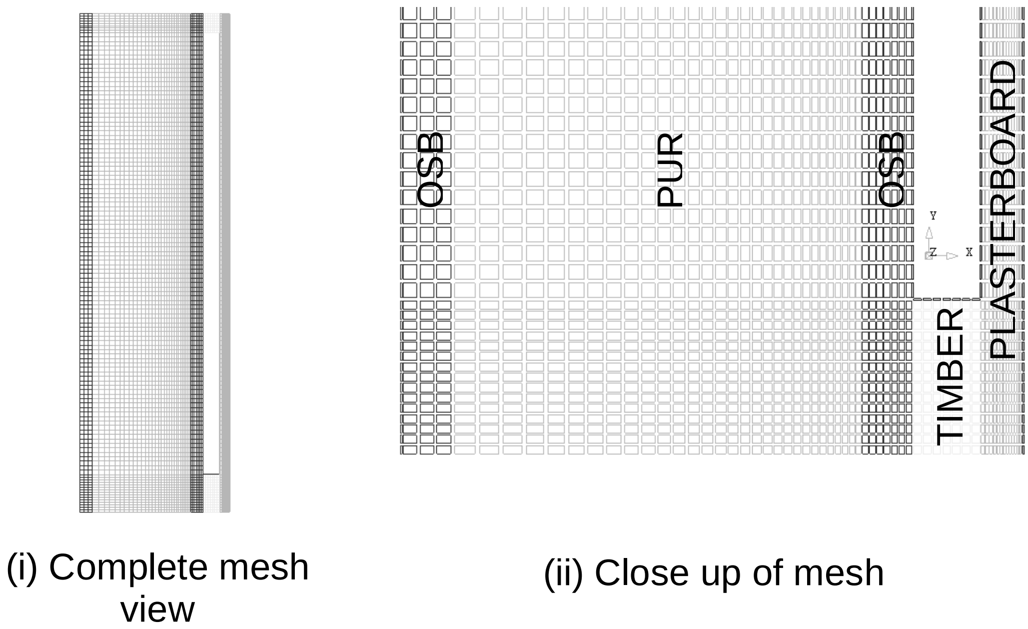



**Figure C.7. Typical finite element mesh**

The boundary elements were assigned with specific properties, which determine the net energy flow into and out of the panel, and the energy exchange across the internal air void (Table C.2.).

**Table C.2. Boundary properties in FE simulations**

| **Boundary** | α **(W/m °C)** | ε |
|---|---|---|
| Heated | 25 | 0.7 |
| Ambient | 4 | 0.7 |
| Void | 25 | 0.7 |

These properties were based on the values proposed in EN 1991-1-2 [18] and include the boundary convection coefficient α and the resultant emissivity ε. For the former, 25 $W/m^2$ K was adopted as it is recommended for standard fire exposure in Eurocode 1 part 1.2. This differs slightly from similar parameters adopted in other studies [2-4], although further investigations by the authors demonstrated that the magnitude of α has little influence on overall temperature development in timber structures [19].

The resultant emissivity was derived from the following equation [20]:

$$e = \frac{1}{1/e_e + 1/e_r - 1}, \qquad \textbf{[Equation C.4]}$$

where $\varepsilon_r$ and $\varepsilon_e$ are the emissivity of the receiving surface and the source (in this instance a flame), respectively. Assuming that the emitter (fire) is a black body ($\varepsilon_e$=1.0) and the receiver has an emissivity of 0.70 then this gives a resultant emissivity of 0.70. The surface of the plasterboard side of the panel, as in the BRE Global experiments [13], is heated according to the ISO834 fire curve [14] whilst the unexposed face is kept at a constant external temperature of 20°C. This is consistent with the mean ambient temperature during testing of the panels. The temperature within the internal air void is determined at each time step using the DIANA void function, based on a calculation method initially developed by Fellinger [21] for the analysis of hollow-core concrete sections.

### 3.2. Development of parametric study

A numerical study was designed to independently establish the quality of different plasterboard datasets against third-party experimental data. Plasterboard properties proposed by Sultan [4], Thomas [2], Schleifer [22] and Ang and Wang [9] were chosen as a basis for comparison. The resulting modelling outputs were then benchmarked against the results of the experimental work undertaken by BRE Global [13]. The study matrix developed for this work is shown in Table C.3.

**Table C.3. Matrix of numerical simulations**

| Run | Plasterboard thickness (mm) | Run duration (min) | Plasterboard property | OSB property | Polymer core property |
|---|---|---|---|---|---|
| 1 | 15 | 30 | Ref [2] | Ref [25]*† | Ref [24] |
| 2 | 15 | 30 | [4] | | |
| 3 | 15 | 30 | [9] | | |
| 4 | 15 | 30 | [22] | | |
| 5 | 30 | 60 | [2] | | |
| 6 | 30 | 60 | [4] | | |
| 7 | 30 | 60 | [9] | | |
| 8 | 30 | 60 | [22] | | |

*Density (ρ=700 kg/m³) and moisture content (8%) corresponding to OSB. †Density ratio as per BS EN 1995-1-2 [23].

In all instances the properties for solid timber (assuming an initial moisture content of 12% by weight from BS EN 1995-1-2 [23]) were applied to the timber battens. Properties for the polymer foam insulation were taken from Hobbs and Lemmon for PUR [24]. For oriented strand board (OSB), a density- and moisture-dependant specific heat for timber, proposed by Cachim and Franssen [25], were adopted, with appropriate density and moisture content.

For validation purposes, runs 1 to 4 were designed to correspond with BRE Global experiments PUR_301 to PUR_303 (See Figure C.8.). Similarly, runs 5 to 8 are modelled after BRE Global experiments PUR_601 to PUR_603.

### 3.3. Results of simulations

The output from the numerical analysis focussed on temperatures in key areas within the panels. This enabled a direct comparison with temperatures measured during the laboratory studies undertaken by BRE Global on SIPs exposed to furnace conditions. These main areas are: (a) the rear of plasterboard; (b) the face of the OSB nearest to the fire side and (c) the mid-depth of the core material. For each area of interest (areas (a) to (c)) the temperatures calculated using the properties shown in Table C.3. were compared with respective experimental measurements. In addition, separate results are shown for the 30-minute and the 60-minute simulations, where 15 mm and 30 mm type F plasterboard were used, respectively.

The results of the 30 minute simulations are shown in Figure C.8. Supporting test data from the BRE Global programme of work [13] is also shown for comparison. The experimental results are denoted "PUR_30" while the results of our finite element simulations are denoted by the name of the author of the adopted thermo-physical plasterboard model (e.g. "Thomas"). The trend "Mean" is an arithmetic average of the BRE Global experimental data. The results of the 60-minute simulations are shown in Figure C.9. Again, relevant results from the BRE Global programme of work are shown for comparison. The experimental results are denoted "PUR_60"; The mean of the experimental results is also shown in the figure.

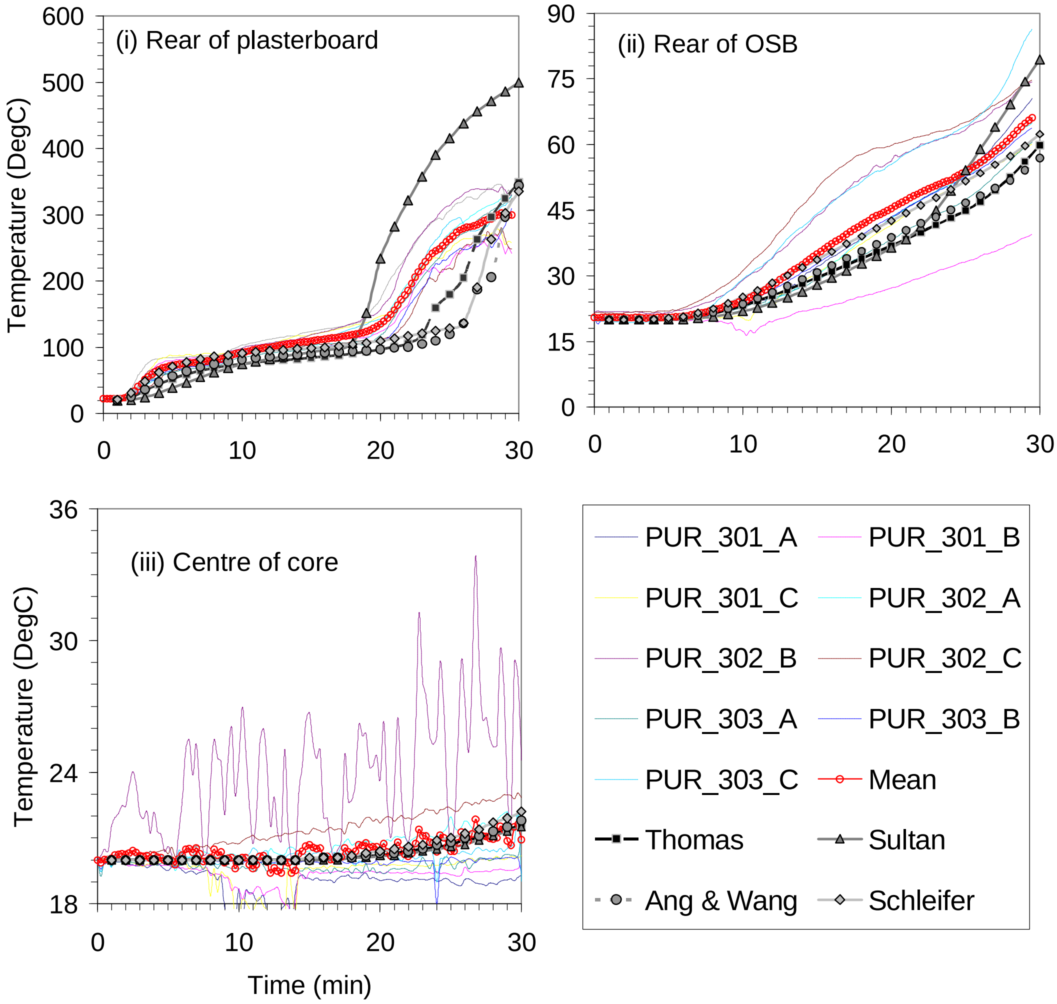



**Figure C.8. Comparison of 30-minute model predictions with experimental data**

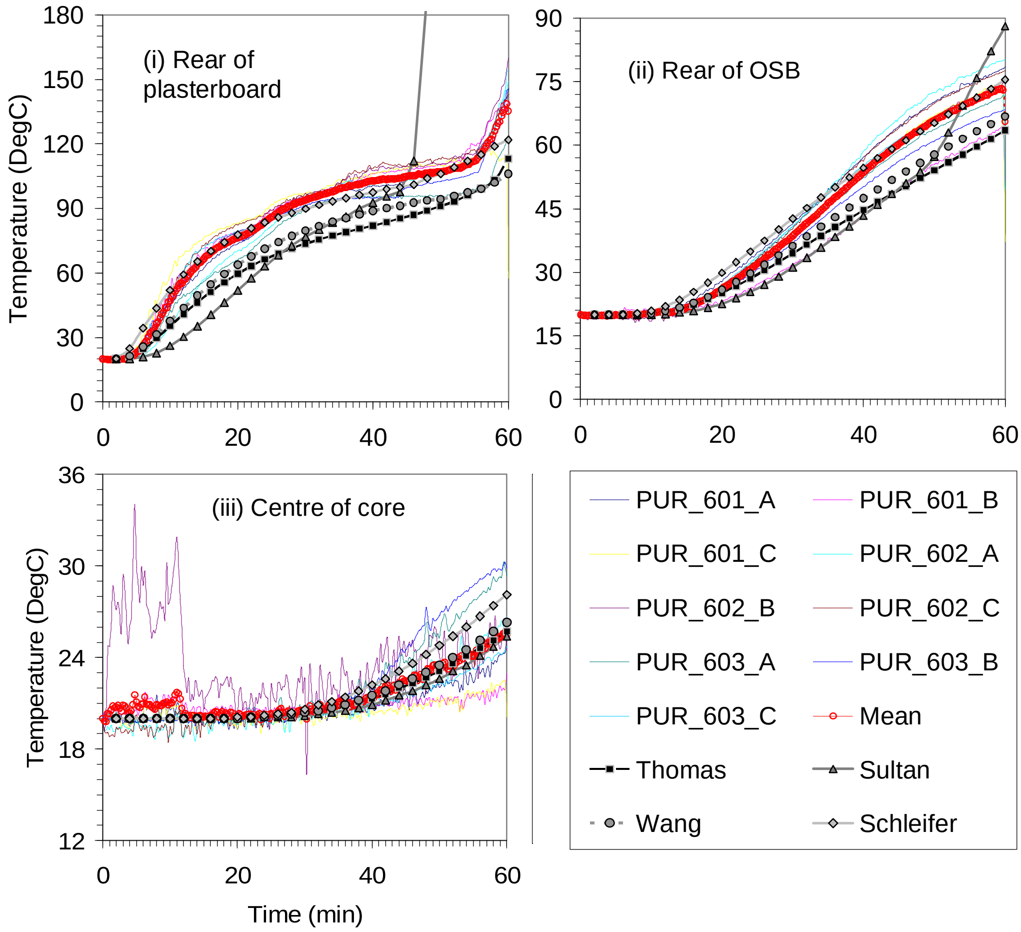



**Figure C.9. Comparison of 60-minute model predictions with experimental data**

4. Sensitivity studies under standard fire exposure conditions

It is apparent from Section 3 that the choice of common plasterboard thermo-physical properties can result in vastly different temperature-development behaviours in protected structures. The example set out previously highlights this fact for structural insulated panel (SIP) assemblies. It is also apparent from comparison of the simulations conducted with third-party experiments, that the current plasterboard properties available in the literature do not produce temperature predictions that are agreeable with the results obtained through the experiments. Many plasterboard properties, particularly those of Thomas [2] and Ang and Wang [9], were developed on the basis of mathematical constructs with assumed moisture contents (both free and chemically bound). Thomas [2] gives these contents as 3 and 21%

respectively. Thomas [2] further adds that approximately 75% of the chemically bound and 100% of the free water content is vaporised in a first dehydration reaction between 95 and 155°C, whilst the remaining 25% of chemical bound water content is vaporised between 200 to 220°C. Based on this, and using the latent heat of evaporation of water and the specific heat of dry gypsum, Ang and Wang [9] proposed the analytical relationships set out previously for specific heat and density ratio. Given the approximate nature of the moisture contents assumed by Thomas [2] it is not unreasonable to expect variations of ±3% in either the chemically bound, or free moisture contents, depending upon environmental or manufacturing conditions. Based on this, and using the correlations provided by Ang and Wang [9], it is possible to not only assess the sensitivity of temperature development in gypsum-lined assemblies for differing levels of moisture content, but also to calibrate thermo-physical properties against measured temperatures in experiments, such as those conducted by BRE Global on SIPs [13]. Given this, a numerical parametric study was designed, again based around temperature development in SIPs using the parameters set out in Section 3, to assess the relative variations in temperature development for moisture contents that differ from those assumed by Thomas [2] and Ang and Wang [9]. The factors used to derive specific heat and density-ratio relationships for the parametric study are summarised in Table C.4.

**Table C.4. Sensitivity matrix**

| Run No. | Free moisture content (%) | Chemically bound moisture content (%) | % split of chemical moisture content | Base specific heat (J/kg K) |
|---|---|---|---|---|
| 1.1 | 3 | 21 | 75-25 | 950 |
| 1.2 | 1 | 21 | 75-25 | 950 |
| 1.3 | 5 | 21 | 75-25 | 950 |
| 2.1 | 3 | 19 | 75-25 | 950 |
| 2.2 | 3 | 23 | 75-25 | 950 |
| 3.1 | 1 | 19 | 75-25 | 950 |
| 3.2 | 5 | 23 | 75-25 | 950 |
| 4.1 | 3 | 21 | 70-30 | 950 |
| 4.2 | 3 | 21 | 80-20 | 950 |
| 5.1 | 3 | 21 | 75-25 | 1100 |
| 5.2 | 3 | 21 | 75-25 | 1200 |
| 5.3 | 3 | 21 | 75-25 | 800 |
| 5.4 | 3 | 21 | 75-25 | 700 |

The value of base specific heat, typically taken as 950 J/kg K by Ang and Wang [9], was also introduced as a variable. The conductivity properties in all instances correspond to those given by Thomas [2] and are assumed to be independent of moisture content. In all instances the f2 factor given by Ang and Wang [9] is taken as unity as this appeared to be the source of the non-conservatism noted in the simulations presented in Section 3. In all instances 15 mm type F plasterboard was modelled and simulations ran for 30 minutes with the criteria, mesh and boundary properties set out in Section 3. The relationship between plasterboard enthalpy moisture content and base specific heat can be noted with reference to Figure C.10.

$$E = \int c(T).r(T).dT \ . \qquad \textbf{[Equation C.5]}$$

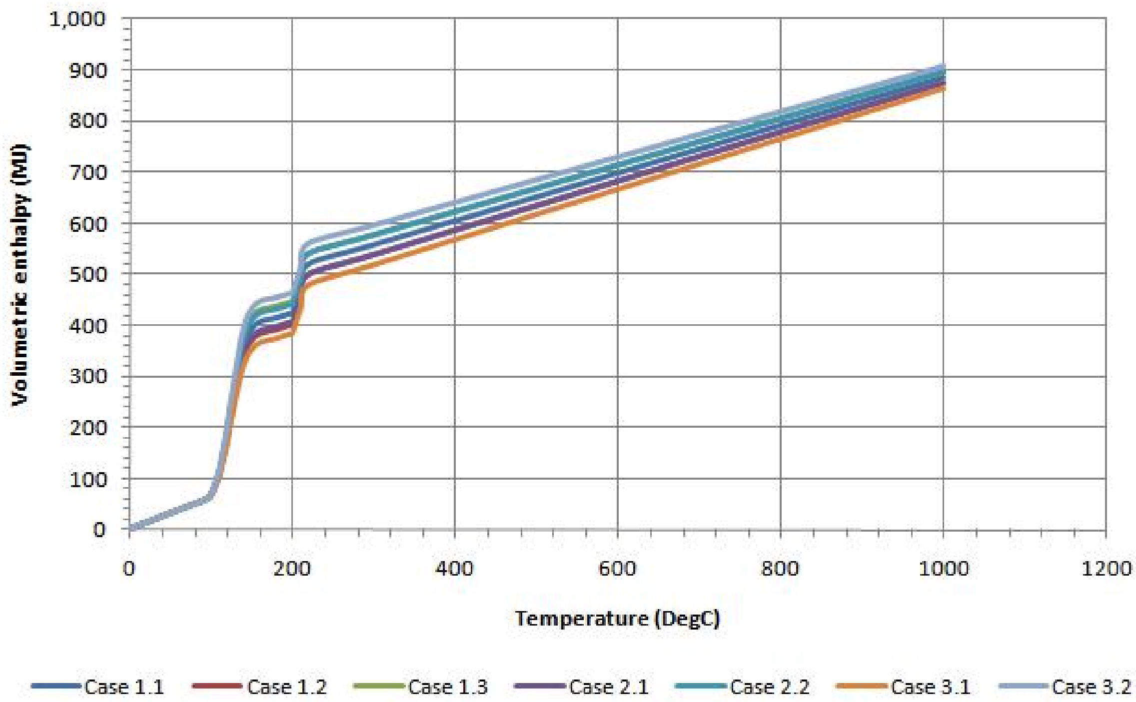



**Figure C.10. Impact of plasterboard moisture content on enthalpy**

Temperature development at the rear of the plasterboard in protected SIPs is shown in Figures C.11. and C.12. Supporting experimental data from BRE Global [13] is also shown for comparison. The experimental results are identifiable as the dashed lines denoted "PUR_30X"; they were included corresponding to the geometry and plasterboard specification modelled. These figures highlight the impact of inherent assumptions, in relation to moisture content and dry gypsum specific heat, on the temperature development calculated with numerical models. It is apparent from the figures that moisture contents lower than those assumed by Thomas [2] and Ang and Wang [9] give better correlation with experimentally measured temperature development in gypsum lined SIPs. However, it should be noted that further improvements are possible, particularly for temperature correlation in the early phases of heating and in the post moisture plateau phases of temperature development.

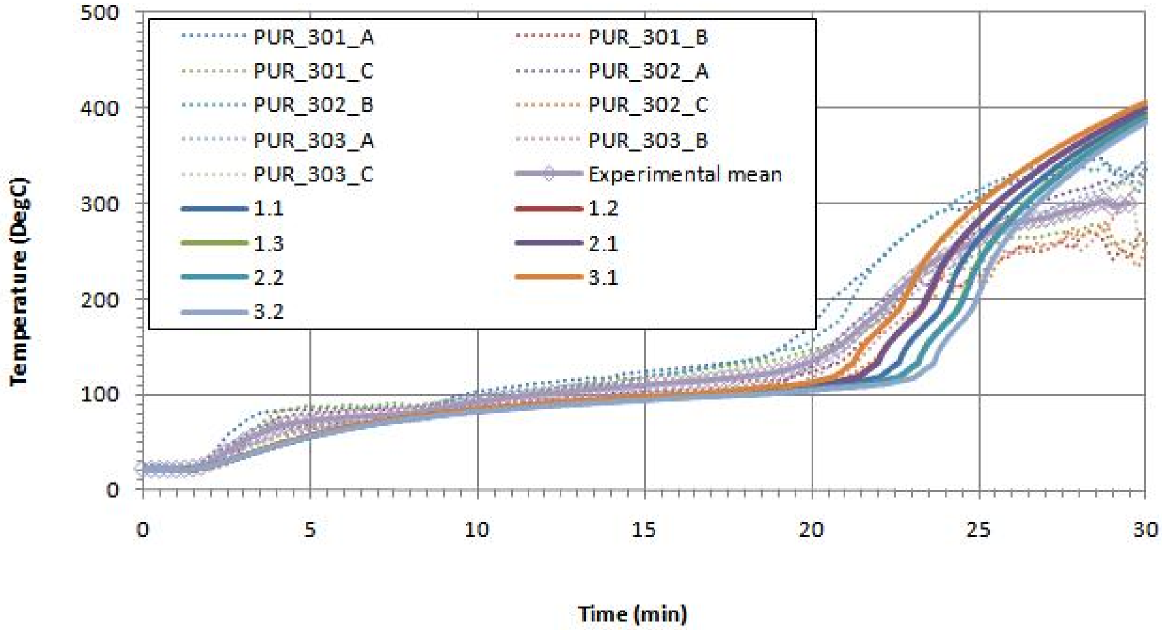



**Figure C.11. Temperature development for cases 1 to 3**

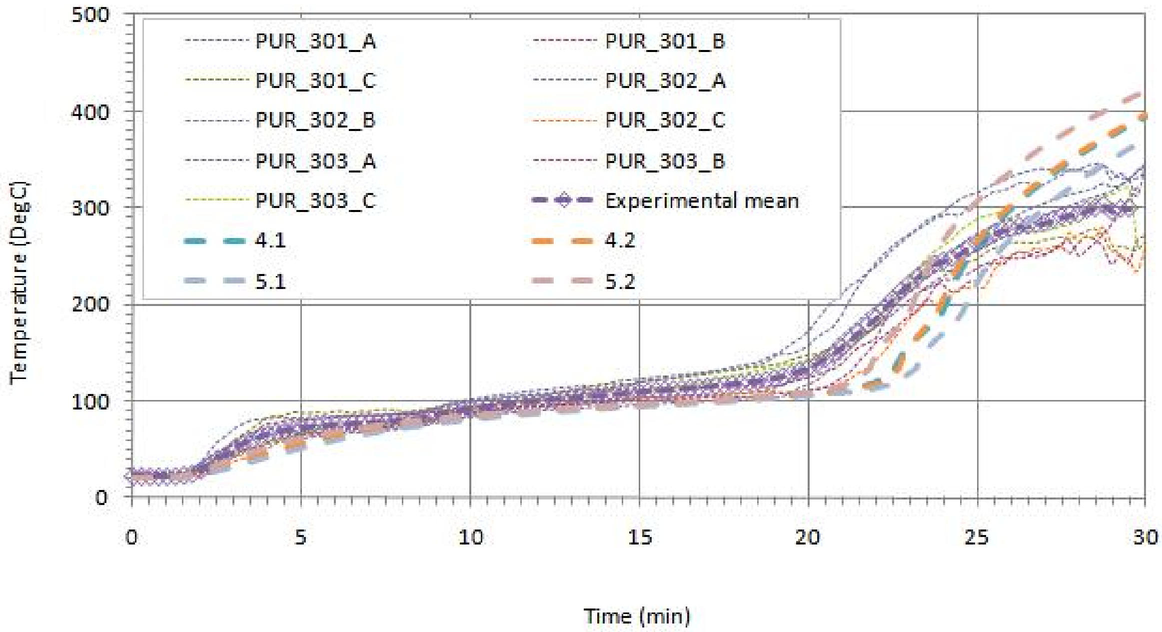



**Figure C.12. Temperature development for cases 4 to 5**

Temperature development in the early phase of heating is governed by the specific heat and conductivity of ambient-temperature moist gypsum. Schleifer [22], unlike most authors, gives an ambient-temperature conductivity of 0.4 W/m K for gypsum to account for the early anomalies typically noted in gypsum temperature development. The post-moisture plateau phase of temperature development is governed by the specific heat of dry gypsum. Many authors use a value of 950 J/kg K as suggested by Mehaffey, *et al.* [3]. Based on these factors, and using the observations made in relation to moisture content, it is possible to calibrate gypsum plasterboard specific heat and density ratio properties such that improved correlation between measured and simulated temperatures can be achieved. This was achieved with a further series of simulations summarised in Table C.5. Figure C.13. shows the resulting simulated temperature development. Similarly, comparisons are made against experiments conducted by BRE Global, denoted as “Experimental mean”.

**Table C.5. Calibration matrix.**

| Run No. | Free moisture content (%) | Chemically bound moisture content (%) | % split of chemical moisture content | Base specific heat (J/kg K) | Additional info. |
|---|---|---|---|---|---|
| 6.1 | 1 | 19 | 75-25 | 950 before de-hydration, 1200 after. | N/A |
| 6.2 | | | | 200 (0-70°C) then 950 before de-hydration, 1200 after. | N/A |
| 6.3 | | | | 950 before de-hydration, 1200 after. | Increased conductivity before 70°C (0.4 W/m K) |

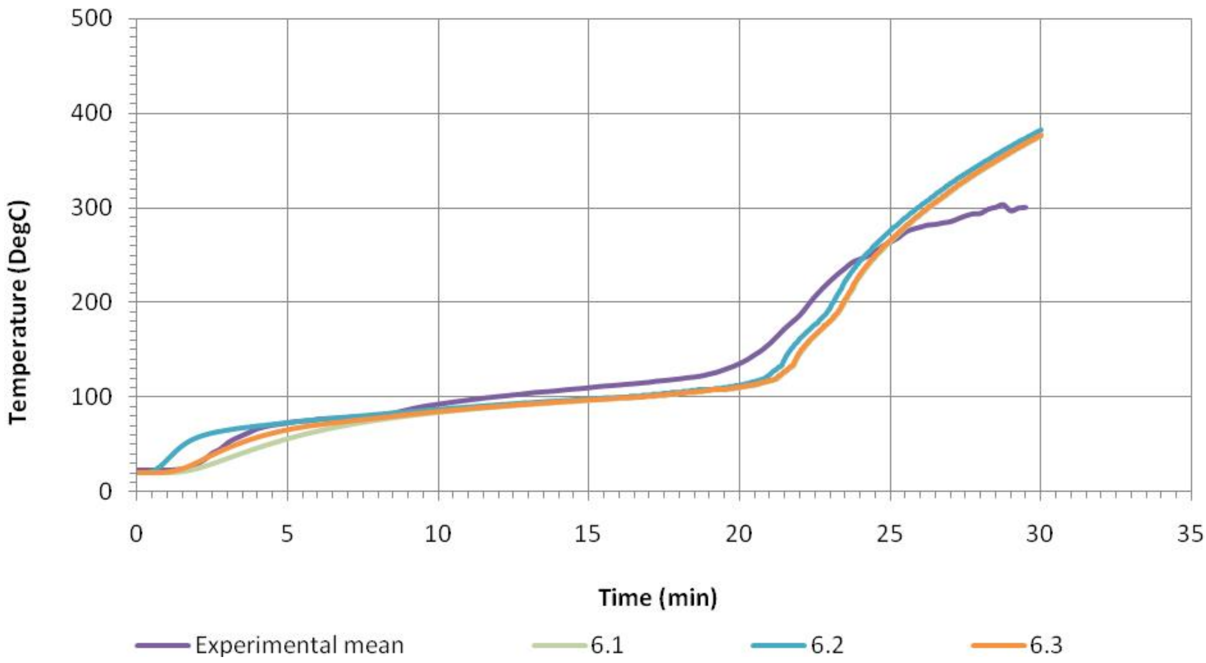



**Figure C.13. Temperature development for case 6**

Once calibrated, the properties noted in run 6.3 of Table C.5. were applied to analyse temperature development in a SIP protected by a single layer of 12.5-mm wall board (Type A). In this instance the temperature-dependant enthalpy was modified to reflect the difference in density between Types A and F. In all studies conducted previously, an ambient density of 780 kg/m$^3$ was assumed for Type F gypsum. In comparison, typical Type A plasterboard has a density in the region of 650 kg/m$^3$, and this was the value adopted. The resulting temperature development at the rear of the exposed side of plasterboard is shown in Figure C.14. The SIP geometry is identical to that defined in Section 3. Supporting experimental data of plasterboard temperatures collected by BRE Global are shown in the same figure for comparison. All experimental results are denoted “EPS_W..”.

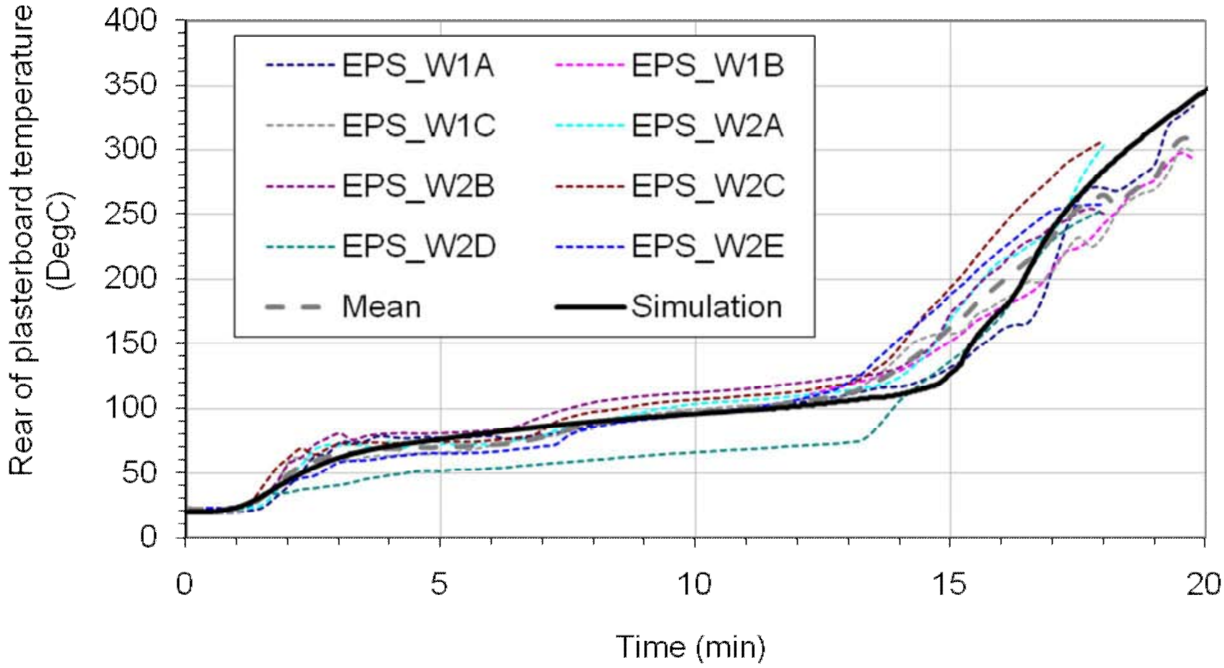



**Figure C.14. Temperature at rear of plasterboard: Measured and simulation results**

5. Simulation of temperature development in real fires

To date, little, if any, experimental observation of temperature development in gypsum-protected assemblies exposed to natural fires has been reported in the literature. BRE Global, in studies of both engineered floor and structural insulated panel behaviour at elevated temperature [13, 16, 17 & 26], have collected numerous series of temperature data for gypsum exposed to natural fires. One of these datasets for a test on gypsum lined solid joists floors was adopted as a case study to investigate the applicability of common plasterboard thermo-physical properties in the simulation of natural fire-induced temperature development. The application of such properties is likely to be an important development in the performance-based design of timber assemblies for fire resistance.

### 5.1. Summary of experiments and modelling approach

The full-scale fire test of a timber-floor system (referred to as Test 1), protected by 25-mm Type F plasterboard and exposed to a natural fire, was published previously by the authors and is summarised in a paper by Lennon, *et al*. [26]. Instrumentation specification can be found in [26]. The experiment exposed a 4 m by 3m floor plate, constructed from 220 mm x 45 mm solid joists at 400 mm centres, to a natural fire designed to represent a typical domestic fire. Temperature measurements were taken in locations throughout the floor void formed by the joists, chipboard sheathing above and plasterboard below. The floor was designed to achieve 60 minutes UK regulatory fire resistance and as a result was protected by 25-mm Type F gypsum plasterboard. The plasterboard was fixed via resilient channels to the supporting joists.

In the simulation of the experiment, a cross-section of the void formed between the joists, sheathing and plasterboard was modelled using the finite element software DIANA. A fully-closed convex void was assumed to be formed in between the components specifieC. The resulting geometry and finite element mesh are shown in Figure C.15.

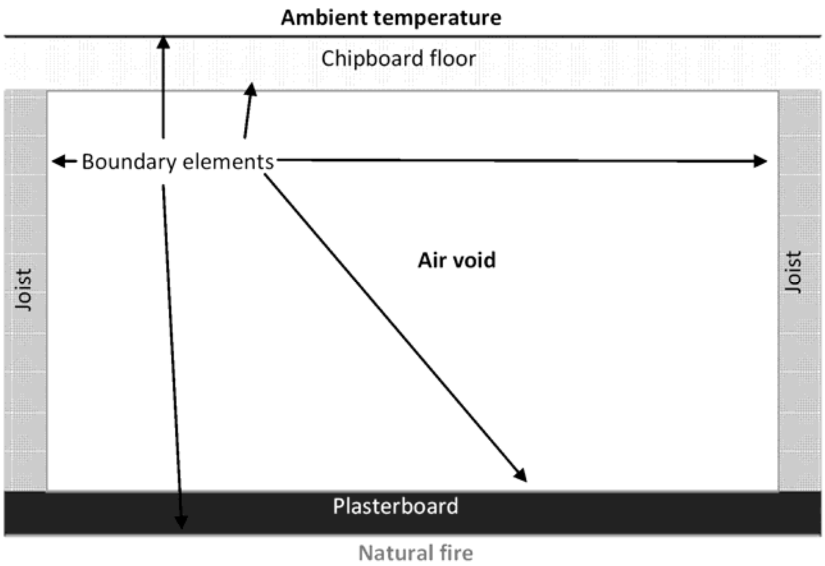



**Figure C.15. Idealised mesh/geometry**

The modelling approach is essentially identical to that set out in Section 3.1. Two-dimensional quad flow and straight boundary first-order elements were adopted throughout.

In this instance the boundary coefficients were modified as the simulation sought to represent a natural fire (refer to Table C.6.); these were based upon the heat transfer coefficients set out in EC1-1-2 [18] for parametric fires.

**Table C.6. Boundary properties in natural-fire FE simulations**

| Boundary | α (W/m °C) | ε |
|---|---|---|
| Heated | 35 | 0.7 |
| Ambient | 4 | 0.7 |
| Void | 35 | 0.7 |

As previously, the DIANA void function (developed by Fellinger [21] ) was adopted to account for radiative heat flow from the hot surfaces enclosing the void and convective heat transfer from the heated gas bound within. The input fire curve was taken directly from the experiment [26]. This was smoothed slightly to prevent numerical issues and was idealised as shown in Figure C.16. The corresponding measured ceiling temperature is shown for comparison.

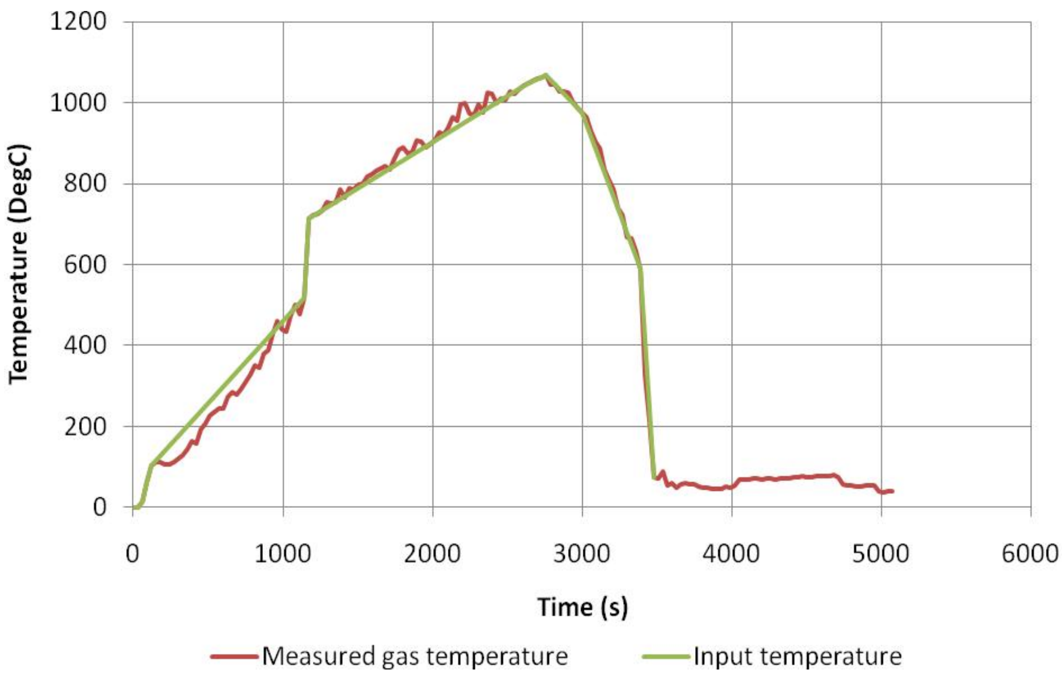



**Figure C.16. Input thermal condition for simulations (from Lennon et al. [25])**

The thermo-physical properties adopted for the various materials are summarised in Table C.7. In this instance plasterboard properties proposed by Sultan [4], Thomas [2] and Schleifer [22] are contrasted. In addition, new properties proposed by Thomas [12] are included that account for a third gypsum-dehydration reaction. The final plasterboard property dataset studied includes those calibrated previously in Section 4 case 4.3. The use of timber properties from EC5-1-2 and those proposed by Cachim and Franssen [25], although limited to standard fire exposure due to the method of derivation, are deemed suitable for this analysis for two reasons. Firstly, the experimentally measured timber temperatures remained low, and, secondly, the heating rate and overall severity of the fire was not significantly different from that of the standard fire curve. In the former case Konig [27] noted that only the thermal properties of the char layer, in particular its conductivity, are dependent upon heating rate. Char formation is typically assumed to develop at 300°C, and the joist temperatures prior to plasterboard fall off, noted in Lennon, *et al*. [26], were only marginally higher than this value.

**Table C.7- Material properties used in natural fire simulations**

| Run number | Plasterboard | Timber | Chipboard |
|---|---|---|---|
| 7.1 | Thomas [10] ¥ | | |
| 7.2 | Schleifer [22] ¥ | EN 1995-1-2 [23] | Cachim & Franssen [25]*† |
| 7.3 | Sultan [4] ¥ | | |
| 7.4 | Thomas [2] ¥ | | |
| 7.5 | Calibrated¥ | | |

*¥ Ambient density of 780 kg/$m^3$.*
**Density ($\rho$=700 kg/$m^3$) and moisture content (8%) corresponding to ChipboarC.*
*†Density ratio as per BS EN 1995-1-2.*

## 5.2. Summary of experiments and modelling approach

Simulation results of predicted temperature development at the rear of plasterboard and mid-point of the timber joist are shown in Figures C.17 and C.18. These are benchmarked against experimental measurements from the natural fire test. The former corresponds with the temperature measured at the lower quarter point of the joist noted by Lennon, *et al*. [26]. The latter case corresponds with the temperature measured at the mid-height of the joist. Some fluctuation in the experimentally measured values can be noted due to the sporadic issuing of hot gases through openings and discontinuities in the plasterboard. The simulations are discontinued in accordance with termination times noted in the experimental programme [26].

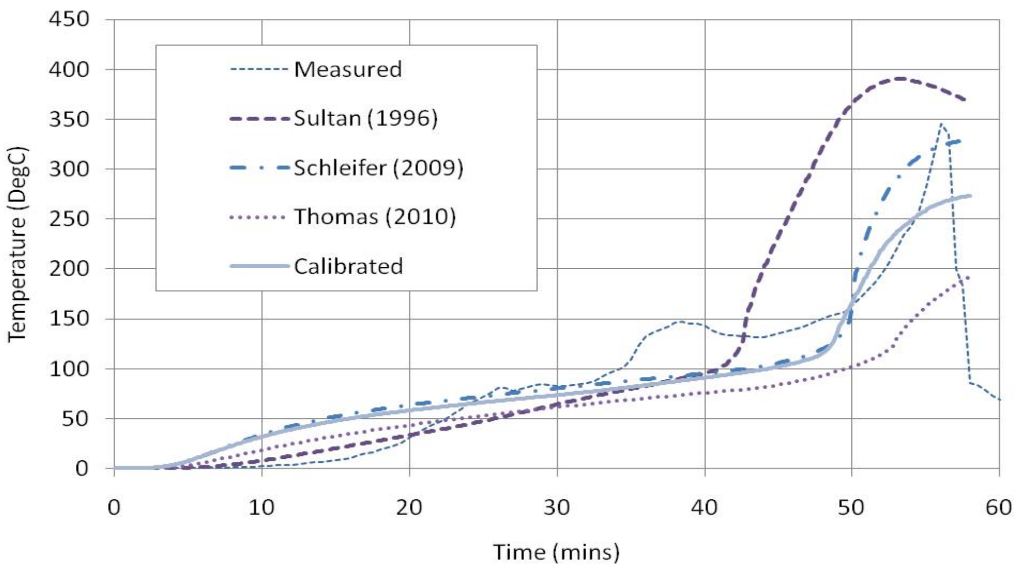



**Figure C.17. Joist quarter point temperatures**

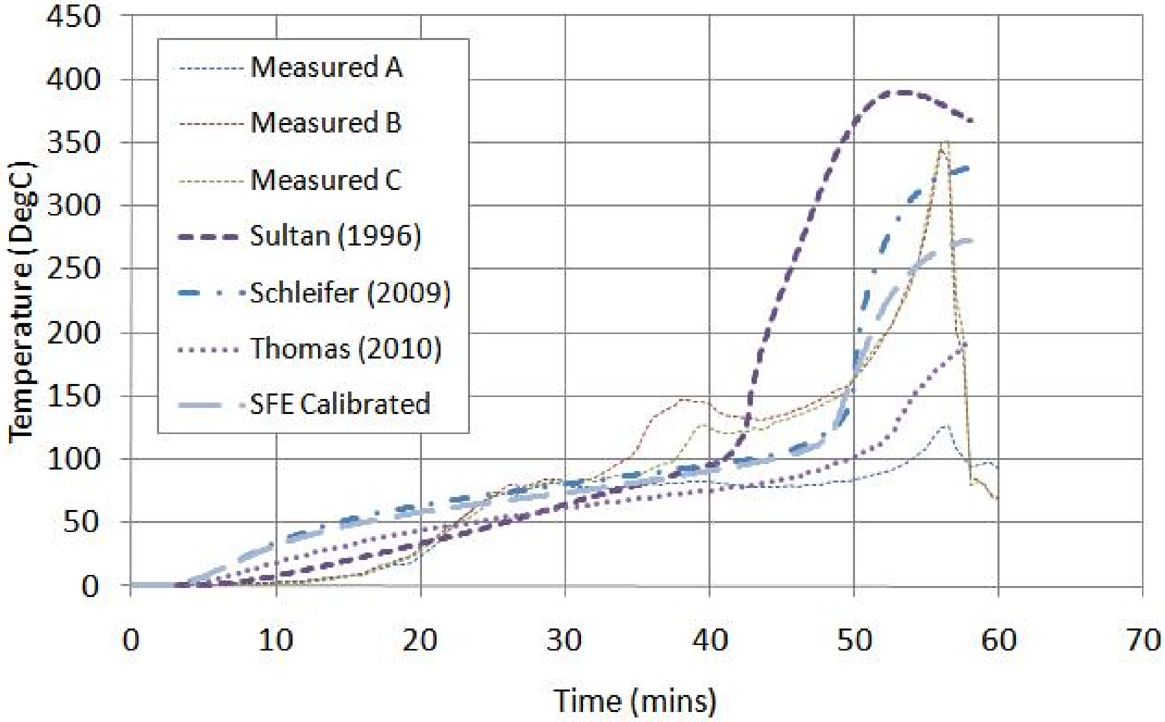



**Figure C.18- Joist mid-point temperatures**

6. Discussion of results

A multi-faceted study of the heat-transfer behaviour of gypsum plasterboard exposed to furnace and natural fires has been presented. It is apparent from the literature that a plethora of data exists for the thermo-physical properties of gypsum board. The method of derivation for these properties varies for experimental measurements, numerical calibration and analytical calculations based upon latent heat of evaporation. It is very clear from the literature that the properties of plasterboard proposed by numerous authors are not in agreement. This is likely to be due to the range of methods adopted in determining the thermal properties of gypsum, in particular specific heat, which can lead to determination of ‘apparent’ or ‘true’ thermo-physical characteristics. In the former case, complex behaviour like mass transfer (due to moisture flow), cracking and ablation are implicitly accounted for in the properties adopted. In the latter case such behaviours need to be considered explicitly. The consequences of these differing thermo-physical properties was made apparent in

Section 3 of this paper, which shows vastly different scenarios of temperature development in structural insulated panels, when one plasterboard dataset is chosen in preference to another.

The work of authors like Mehaffey, *et al*. [3], Sultan [4] and Thomas [2] have underpinned many recent numerical investigations of gypsum-plasterboard protected assemblies. The work of Mehaffey*, et al*. [3] and Thomas [2] in particular quantitatively assessed the specific heat of plasterboard through consideration of bond dislocation energy and the latent heat of evaporation of associated released moisture. This study was built upon by Ang and Wang [9], who proposed analytical equations for the derivation of the specific heat of plasterboard. These equations formed the basis of a numerical parametric study that allowed the authors to calibrate a new set of plasterboard thermo-physical properties on the basis of a number of experiments conducted by BRE Global [13, 16, 17]. It was found, in the experimental conditions investigated by the authors, that assumed moisture contents of 21% chemically bound and 3% free proposed by Thomas [2] and subsequently by Ang and Wang [9] may overpredict the thermal fire resistance capability of gypsum board when it is used to protect SIPs. The addition of a mass-transfer calibrated factor (defined as f2 by Ang and Wang [9]; shown in Equations C.1 & C.2) again was also shown to overpredict the fire resistance capability of gypsum board, in comparison to the experimental evidence gathered. Such differences apparent in the authors' research can be attributed to a number of factors. Firstly, most, if not all, studies relating to the behaviour of gypsum have investigated temperature development within timber stud walls [2-4]. In such cases the gypsum board is backed by either an empty or partially insulated void measuring anything from 80 to 220 mm. A (partial) void allows heat energy to be dissipated from the rear of the gypsum through a combination of radiation and convection into the enclosed air space, coupled with the unobstructed flow of moisture through the rear of the gypsum board. Compared to polymeric foams, an air void is a poor insulator. As a result, when the same thermo-physical properties (i.e. [2-4]), derived via calibration against experiments on stud walls, are applied to structural insulated panels then the plasterboard becomes proportionally hotter. This is due to the restriction of heat flow from the rear of plasterboard as the underlying SIP is highly insulating, with a low thermal inertia, resulting in an overall increase in gypsum temperature.

From calibration against heat transfer experiments, the authors propose the thermal properties for gypsum board shown in Figure C.19, which should be adopted when plasterboard is backed by a highly insulating substrate, such as a SIP. Separate properties are shown for wallboard (Type A) and fire resistant board (Type F). These properties, when compared against those in the literature, gave the best correlation with measured temperatures in experiments conducted by BRE Global [13].

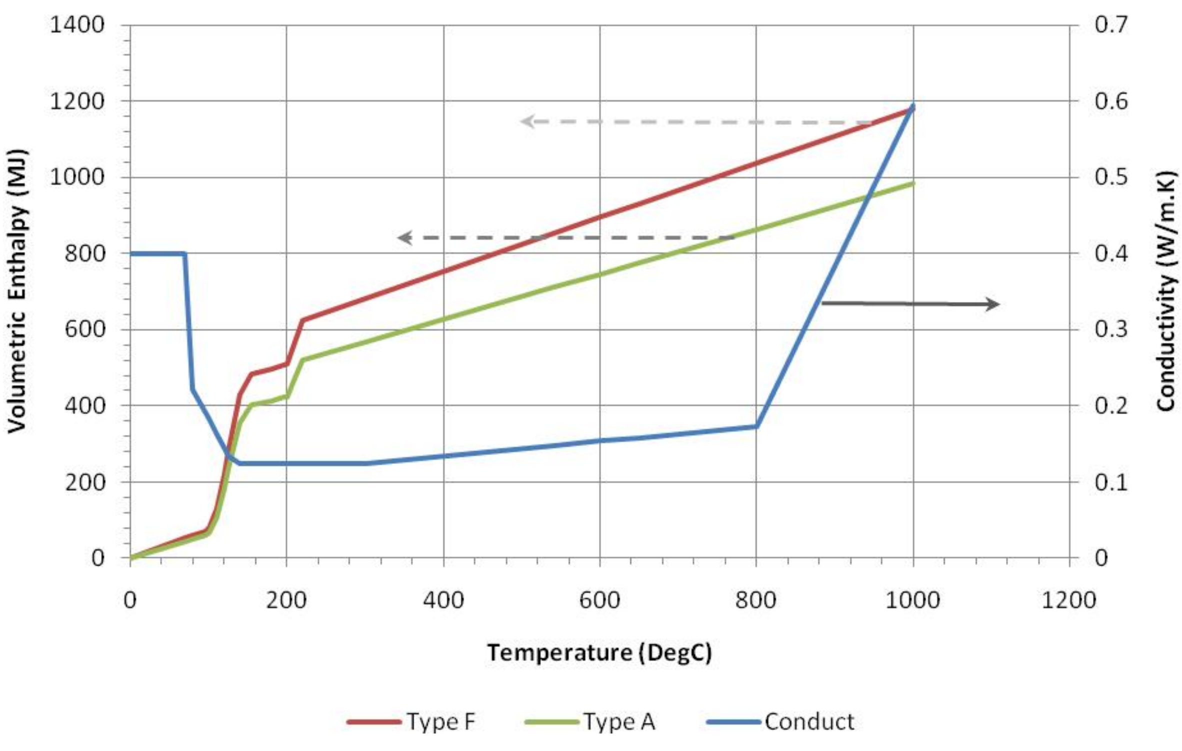



**Figure C.19. Recommended values for plasterboard thermo-physical behaviour under standard fire exposure conditions**

The properties proposed amalgamated conductivity values proposed by Thomas [2] with those of Schleifer [22] to achieve better correlation within the moisture-plateau phase of the authors' experiments. Further to this, the specific heat properties were derived using the correlations proposed by Ang and Wang [9] by ignoring mass transfer and assuming free and chemically bound moisture contents of 1% and 19%, respectively. Another modification was made to the temperature-dependent specific heat by increasing the post-dehydration specific heat of plasterboard to 1200 J/kg K.

Following this study, simulations of temperature development in timber joist floors exposed to a natural fire were conducted. The experiments, again conducted by BRE Global, are summarised in an earlier paper by the authors [26]. The plasterboard properties previously calibrated were compared with established studies by Sultan [4], Thomas [2], Schleifer [22] and Thomas [12] and were benchmarked against measured temperatures in the floor cavity. Interestingly, the properties proposed by Schleifer [22], which gave the worst correlation with measured temperatures for SIP standard fire exposure cases, were closest to experimental measurements of floor-void temperature. Those properties calibrated by the authors, in addition to properties given by Sultan [4] and Thomas [2, 12], formed the upper and lower bound temperature predictions and differed significantly from the measured temperatures of plasterboard and joist. The more recent properties of Thomas [12] gave particularly poor predictions of temperature development in the joist floor and significantly under-predicted temperature development. It appears that the inclusion of an additional specific heat spike, corresponding with a third gypsum dehydration reaction, may result in over-predictions of the fire-resistance performance of gypsum, particularly when exposed to natural fires.

It is apparent as a result of this study that those properties developed, calibrated and validated against standard fire exposure experiments are likely, as is the case for timber, to be valid only for this exposure condition. In addition, the inclusion of a void rather than a highly insulating substrate, appears to highly influence the thermal properties that should be adopted, for the reasons highlighted previously. In addition, it appears that various properties proposed in the literature for gypsum board are not independent of heating rate, and that some modification should be made in light of this. The better correlation with simulations using the thermo-physical properties for gypsum proposed by Schleifer [22] suggest that conductivity and not the specific heat is likely to be the controlling factor in heating rate-dependent properties for natural fires. The conductivity of gypsum proposed by Schleifer [22] notes much higher initial and post-dehydration conductivity than many properties present in the wider literature. This is rather similar to the case for solid timber, for which Konig [27] noted that the conductivity of the char layer should be modified in accordance with heating rate. Heating-rate-depend-nt conductivity values, like those apparent for timber [27], could arise for a number of reasons. Firstly, as most material models include the effects of mass transfer implicitly, it is highly likely that moisture flow within gypsum varies depending upon the heating rate. In addition to this, again similar to timber [27], the formation of cracks and the ablation rate are also likely to vary depending upon the severity of heating.

More generally, the paper has shown that the modelling of plasterboard exposed to natural fires is a realistic possibility. The experiments conducted by BRE Global [26] were vastly simplified to allow heat-transfer modelling to be conducted. However, correlations between simulation results and experimental measurements are acceptable. It is apparent that it may be necessary to consider complex behaviour like cracking, ablation and mass transfer in a more explicit manner for gypsum exposed to non-standard (or natural fires). Further to this, complex behaviour not considered in FE models such as the issuing of hot gasses through openings in the plasterboard joints, and, more importantly, plasterboard fall off, are likely to affect significantly temperature development as a failure of a protected structure is approached. The prediction of plasterboard failure under standard fire exposure conditions is a difficult task, however, recent methods have been proposed both by Just, *et al*. [28] and Sultan [29]. It is hoped that ultimately, through further supporting experimentation, those concepts could be extended to predict plasterboard failure times in natural fires. With such developments, phased thermal FE models can be implemented to simulate pre- and post-protection failure phases of behaviour in fire.

It is also important to note here that plasterboards are not generic, they vary by manufacturer and application. Hence, the properties proposed in literature and those in this paper are only indicative properties based upon the products tested as part of specific research projects. With this in mind, it is not surprising to see variations in published thermo-physical properties of plasterboards or in resulting predictions of temperature development in protected assemblies.

## 9. Conclusions

This study highlighted a number of important conclusions relating to the simulation of plasterboard assemblies exposed to both standard and natural fire conditions:

1. The widespread variation in properties available for gypsum plasterboard result in significantly different predictions of temperature development in protected structural insulated panels. This is not surprising given the variation in methods of derivation and in products investigated.

2. Based on existing equations proposed by Ang and Wang [9], new indicative plasterboard thermal properties are proposed for standard fire exposure and the modelling of gypsum-lined SIPs, which distinguish between Type A (standard) and Type F (fire-resistance plasterboard). These properties were validated/calibrated against heat transfer experiments conducted at BRE Global [13] and are shown to give good agreement with the experiments conducted.

3. The properties proposed in the wider literature for gypsum board can be applied in the simulation of protected timber assemblies under natural fire conditions. However, these properties can only be applied up to the point, at which large gaps open in the plasterboard joints and the board ultimately fails. After this phase, heat transfer into floor cavities is largely driven by buoyant gasses entering the space from the fire below.

4. Many of the properties for gypsum presented in the literature, including those calibrated by the authors in this study, appear not to be independent of heating rate and can result in inaccuracies, when applied in the simulation of temperature development under natural fire conditions. The likely cause of this is the relationship between heating rate and other behaviours such as moisture flow, ablation and cracking.

5. If modelling of gypsum-protected assemblies exposed to natural fires is to be conducted, it is recommended that behaviour mentioned above is considered in a more explicit manner and not through ‘effective’ or ‘apparent’ thermal properties.

6. The inclusion of modified specific heat associated with mass transfer and a third dehydration reaction in gypsum board, as proposed by Ang and Wang [9] and Thomas [12], respectively, results in the underestimation of temperature development in SIPs and joist floors according to the experimental measurements taken and the simulations conducted. However, their applicability in stud wall assemblies has not been investigated.

## 10. Acknowledgements

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# APPENDIX D AN EFFECTIVE THERMAL PROPERTY FRAMEWORK FOR SOFTWOOD IN PARAMETRIC DESIGN FIRES: COMPARISON OF THE EUROCODE 5 PARAMETRIC CHARRING APPROACH AND ADVANCED CALCULATION MODELS

**Full Reference**

HOPKIN, D.J., EL-RIMAWI, J, SILBERSCHMIDT, V., and LENNON, T., 2011. An effective thermal property framework for softwood in parametric design fires: Comparison of the Eurocode 5 parametric charring approach and advanced calculation models. *Journal of construction and building materials,* **25**(5), pp 2584-2595.

**Abstract**

Timber, like other structural materials such as concrete and steel, has its own Eurocode (Eurocode 5 part 1.2) for the structural fire design of buildings. However unlike other fire parts of the Eurocodes it is not widely adopted due to its inherent limitations. With the exception of a single annex, the timber Eurocode (EN 1995-1-2) is only applicable to standard fire exposure. Annex A gives guidance on the charring rates of initially un-protected timber members in parametric fires, however in the UK the use of the Annex is prohibited by the national annex to the code.

The concrete and steel industries have undoubtedly benefited from performance based design whereby the structural fire design strategy is centred on a design fire (typically a parametric fire), which is more credible than the standard fire curve. Such an approach has resulted in more flexible, innovative buildings which have been designed based upon fundamental structural mechanics at elevated temperature, using advanced numerical models. At present however the same principals cannot be applied to the advanced fire design of timber buildings

due to current limitations in the timber Eurocode. Where advanced calculation procedures are considered by the code (Annex B), much like many of the methods contained therein, the procedures are only applicable to standard fire exposure.

The scope of applicability of the code stems from a fundamental problem regarding a lack of understanding of the heat transfer characteristics of timber in natural fires. The thermo-physical properties contained in the code are ‘effective’ properties. This essentially means that they are calibrated against test results to account for a lack of understanding regarding mass transfer, cracking and ablation both within the timber and char layer. Such calibrations have only been performed on timber members exposed to standard furnace conditions.

To attempt to overcome this barrier and extend the scope of thermo-physical properties in the code a study has been undertaken to establish how the conductivity properties of the char layer influence the depth of char in parametric fires. Through calibration of an effective conductivity of the char layer against the parametric charring method contained in Annex A of EN 1995-1-2, it has been possible to establish a relationship between ‘heating rate’ and the effective conductivity of the char layer, in the heating phase of parametric fires. The modified conductivity model is shown to be applicable to a range of densities and moisture contents of timber and also variations in heating rate and fire load density. The latter is a direct result of the method used in the adaptation of the properties. The modified model is objectively critiqued and proposed further work is discussed in detail. The applicability of the modified model in the cooling phase of fires is also discussed.

**Keywords**

Timber, charring, parametric fires, modelling, structural fire engineering, timber in fire, wood

## Nomenclature

$\beta_0$- One dimensional charring rate under IS0834 heating (mm/min)

$\beta_n$- Notional charring rate under ISO834 heating (mm/min)

$\beta_{par}$- Notional parametric charring rate (mm/min)

$\beta_{par,\ 1D}$- One dimensional parametric charring rate (mm/min)

$\beta_{\omega\rho 0}$- Density and moisture modified 1D charring rate (mm/min), after Cachim and Franssen

$d_{ch}$- Depth of char (mm)

$d_0$- Depth of zero strength layer (mm)

$\Gamma$- Heating rate relative to the standard fire curve (dimensionless)

$q_{td}$- Fire load density related to the total area of the enclosure (MJ/m$^2$)

O- Opening factor (m$^{0.5}$)

b- Compartment thermal inertia (J/m$^2$s$^{0.5}$K)

$t_0$- Time period with a constant charring rate (min)

$t_{max}$- Duration of the heating phase of a given parametric fire (min)

$\lambda$- Conductivity (W/m.K)

c- Specific heat of timber (J/kg.K)

$\varepsilon$- Emissivity (dimensionless)

$\alpha$- Convection coefficient (W/m$^2$.K)

$k_{\lambda,mod}$- Heating rate and fire dependant conductivity modification factor (dimensionless)

$k_{\Gamma,mod}$- Heating rate dependant conductivity modification factor (dimensionless)

$k_{qtd,mod}$- Fire load dependant conductivity modification factor (dimensionless)

$\rho_0$- Ambient density of timber at 12% moisture content (kg/m$^3$)

G- Density ratio (dimensionless)

$\omega$- Moisture content of timber (%)

$\omega_f$- Moisture content mass fraction (dimensionless)

## 1. Introduction

Timber is an inherently sustainable material that has an important future in the design of low or zero carbon buildings. However, in recent years the sustainability benefits of working with timber are often overlooked due to a stigma associated with the fire performance of buildings, both during and after construction. Some of these issues could be addressed through a more rationalised approach to the fire engineering design of timber structures.

At present in the UK timber structures are generally adopted where the building height does not exceed 18m. This is governed by the UK Building Regulations [1] which, for structures in excess of this height, require in excess of 60 minutes fire resistance. Although 90 and 120 minutes regulatory fire resistance of timber structures is achievable it is often very uneconomic, when compared to alternatives such as steel or concrete. As a result the construction of tall timber buildings in the UK is a rare occurrence. The steel and concrete industries appear to have achieved reductions in section size and efficiency savings in passive fire protection and volume of material through performance based design. The efficiency savings achievable by these industries are not currently possible for timber structures and improvements could be made through applied research.

At present the Eurocode [2] for the design of timber (EN 1995-1-2), compared to the concrete and steel codes, is less advanced especially where exposure to fires other than that defined by the standard ISO834 [3] curve are considered. Currently, the most common procedure for the fire design of timber structures is the residual cross section method which is popular due to its simplicity. It accounts for reduction in load bearing capacity, caused by a fire, through consideration of the depth of char ($d_{ch}$) and depth of pyrolysis layer (often referred to as a zero strength layer, $d_0$) as a function of time. The depth of char is most commonly calculated for standard fire exposure and is based on either one dimensional charring rates (defined as $\beta_0$), where corner rounding must be considered explicitly, or notional charring rates ($\beta_n$), where corner rounding is considered implicitly. These charring rates (both one-dimensional and notional) are independent of both density and moisture content of timber. However, studies have shown [4] that they are consistent with a timber moisture content of 12% and an initial density of 450kg/m$^3$. The zero strength layer ($d_o$) is nominally 7mm as defined by the code. Where performance based design using parametric fires is explicitly covered by the code it is limited to initially unprotected member design using empirical functions which relate heating rate to a modified 'parametric charring rate'. In the formulation for parametric charring rate ($\beta_{par}$) a modification factor, which is dependant upon heating rate (indirectly $\Gamma$), is applied to a notional or one dimensional charring rate to account for differences in heating rate relative to the standard fire curve.

The use of advanced finite element analysis (FEA) of timber structures is currently limited to standard fire exposure conditions (covered in Annex B of EN 1995-1-2). This renders the use of FEA tools a fruitless and often pointless exercise in a practical design environment. For timber to effectively compete with other forms of construction for building heights in excess of 18 meters, and to ensure fire safety in completed buildings, the 'performance based' element of structural design must be introduced where non-standard fires are considered. The most effective avenue for achieving this could be through whole building finite element (FE) models.

## 2. Background to research

The design of timber structures for non-standard fire conditions is not simple. Limited test data exists on temperature development within timber structural elements in non-standard fire conditions. In addition, as timber is an organic material, the thermo-physical properties are complex and depend on a number of factors. The complex phenomena present in heated timber elements are difficult to model explicitly and hence to date ‘effective properties’ are often defined. Such properties implicitly account for the effects of complex behaviour, such as the flow of pyrolysis gases and water vapour, through calibration against known temperatures, in limited experimental configurations. König [5] has been instrumental in initiating such a process for timber and has calibrated ‘effective’ thermal properties for standard fire exposure conditions. These properties form the basis of the advanced calculation models contained in Annex B of EN 1995-1-2. However, additional studies by König [6] proved (both experimentally and numerically) that these properties exhibit very conservative predictions of char depth when applied to non-standard (parametric) fire conditions where the heating rates are in excess of that of the standard fire curve. Similarly the properties from the code were shown to result in un-conservative predictions of timber temperature and depth of char when the heating rate was lower than that of the standard fire curve. As a result EN 1995-1-2 explicitly states that the thermal properties present in Annex B should only be adopted for standard fire exposure and not parametric fire exposure.

There is a perception in the timber research community that the implementation of advanced heat and mass transfer models may allow heat transfer in timber elements to be more accurately determined for realistic fire conditions [6]. Such models would more accurately consider the flow of moisture and gases within the timber, which appear to be hugely important factors in heat transfer under natural fire conditions. Such developments, although academically interesting, would however be of limited use to practitioners, as models would be overly complex. In addition, although mass transfer effects could be considered, the consequences of complex stochastic phenomena such as ablation and char cracking would still not be considered. To this end the use of 'effective' thermal properties, in finite element formulations, still represents the most realistic concept for the advanced fire design of timber structures. The remainder of this paper discusses the current range of application of EN 1995-1-2 and proposes a modified conductivity model which can be applied to parametric design fires.

## 3. Performance based fire design of timber structures using EN 1995-1-2

Current provisions in EN 1995-1-2 for the 'performance based' fire design of timber structures include the advanced calculation annex (Annex B) and the parametric charring method for initially unprotected timber members (Annex A). It has been previously discussed in this paper and noted in others [6] that the applicability of the advanced calculation methods of EN 1995-1-2 are limited to standard fire exposure.

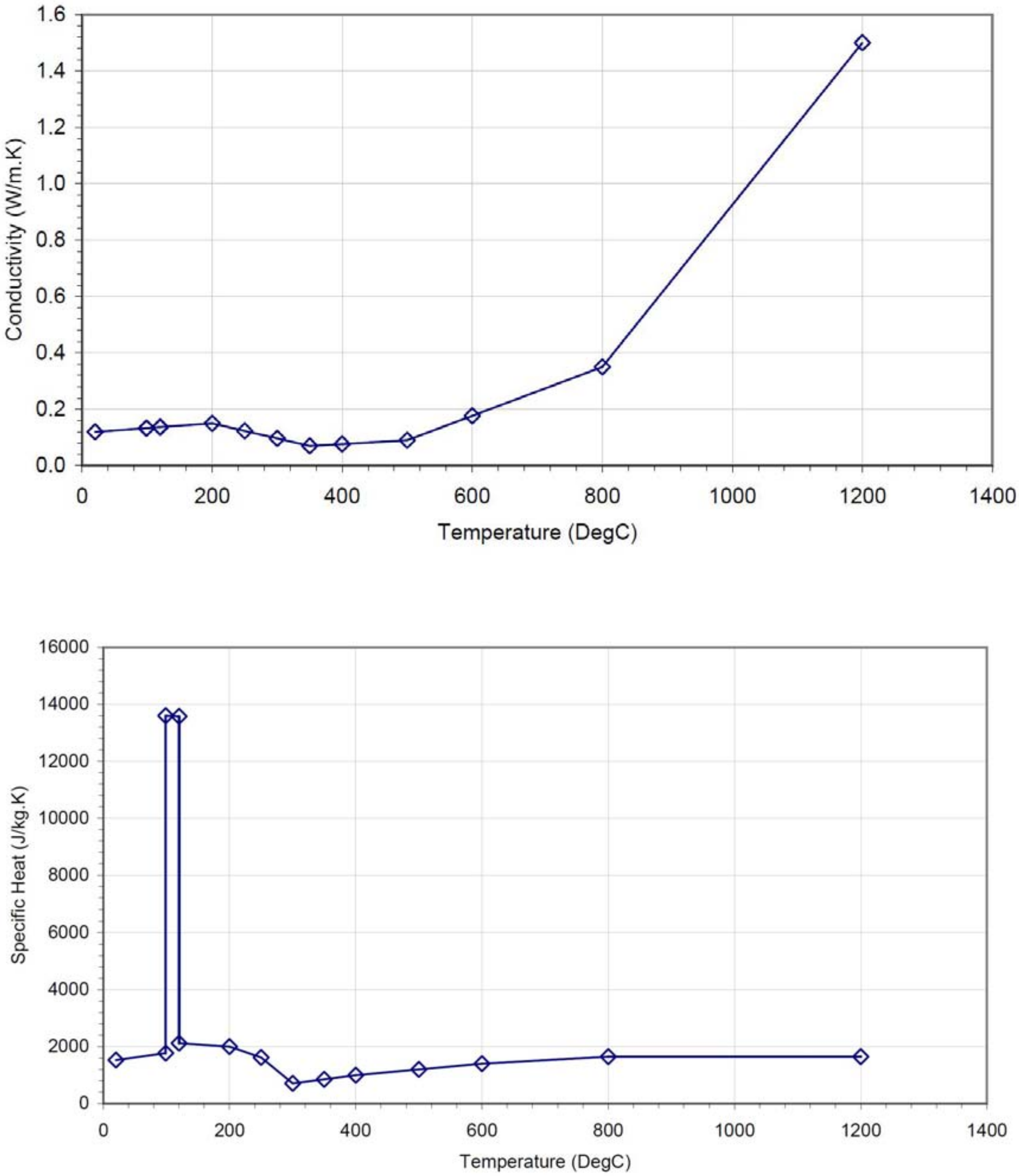


**Figure D.1. Conductivity and Specific heat f(T) from EN 1995-1-2 [2]**

König [6] has shown that when such properties (see Figure D.1) are applied outside of this exposure scenario the results are inconsistent. For example, when the heating rate is in excess of that of the ISO834 [3] curve the predicted charring depth is extremely conservative. On the other hand, lower heating rates produce un-conservative results. In addition, the applicability of the properties during the cooling phase of a fire is also questionable due to char oxidation [6]. The advanced thermo-physical properties proposed in the code are largely based on the work of König [5], derived from a number of experiments performed in Sweden in the 1990's [7].

The parametric charring method in EN 1995-1-2 is entirely empirical and is based on a limited number of natural and non-standard furnace experiments performed in Scandinavia [8-9]. To this end the parametric charring rate can be considered partially 'validated'. The parametric charring method proposed in EN 1995-1-2 relates the charring rate of timber in standard fire conditions ($\beta_n$ or $\beta_0$) to the charring rate in parametric fires ($\beta_{par}$) via a heating rate ($\Gamma$) dependant function. For one dimensional parametric charring the notional standard fire charring rate ($\beta_n$) can be replaced by the one dimensional charring rate ($\beta_0$). The relevant functions for heating rate and parametric charring rate are shown in Equation D.1.

$$b_{par} = 1.5 b_n \frac{0.2\sqrt{\Gamma} - 0.04}{0.16\sqrt{\Gamma} + 0.08} \text{ or } b_{par,1D} = 1.5 b_0 \frac{0.2\sqrt{\Gamma} - 0.04}{0.16\sqrt{\Gamma} + 0.08} \qquad \textbf{[Equation D.1]}$$

**Where:** $$\Gamma = \frac{(O/b)^2}{(0.04/1160)^2}$$

The char depth ($d_{ch}$) of unprotected timber in parametric fires is assumed to increase linearly from time zero to the end of the 'constant charring phase' ($t_0$) by $\beta_{par}$ mm/min. After this, the charring rate ($\beta_{par}$) decreases linearly to zero during the steady state and cooling phases of the fire over a period of ($2t_0$). This is shown diagrammatically in Figure D.2.

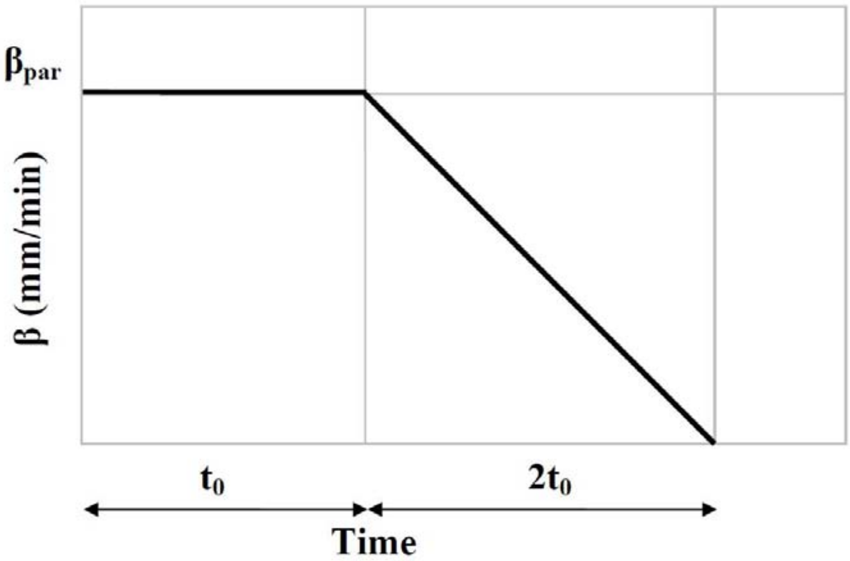


**Figure D.2. Parametric charring rate diagram [2]**

The time of constant charring rate ($t_0$) is given as a function of fire load density ($q_{td}$) and opening factor (O) as in Equation D.2.

$$t_0 = 0.009\left(\frac{q_{td}}{O}\right) \leq 40\,\text{min}$$ **[Equation D.2]**

The parametric charring modification factor (i.e. the function multiplying $\beta_n$) is assumed to equal one when the parametric heating rate is identical to that of the standard curve (i.e. $\Gamma$=1.0). This however implies two things. Firstly that heat transfer via convection is identical in both furnace (standard) and natural (parametric) fire conditions. This is inconsistent with Eurocode 1 which indicates that heat transfer via convection is more pronounced in natural fires and encourages the use of $\alpha$=35 W/m$^2$K for parametric fires, compared to $\alpha$=25 W/m$^2$K for furnace conditions [10-11]. Where $\alpha$ corresponds to the convection coefficient adopted in heat transfer calculations. Secondly, that the parametric curve function yields an identical time temperature relationship to that of the standard fire curve when $\Gamma$=1.0 This is only approximately true in the early phases of heating.

## 4. Proposed changes to advanced calculation model properties

To implement whole building or sub-assembly structural fire engineering models in the design of timber buildings, appropriate levels of accuracy are required in the prediction of structure temperatures. At present this is only achievable for standard fire exposure due to limitations in thermal properties and thus efficiency savings against credible design fire scenarios are impossible. To overcome this barrier the authors propose a very simple change to the effective thermal properties, and in particular to conductivity, presented in Annex B of EN 1995-1-2 which would make them applicable to parametric design fires.

### 4.1 Background

Previous studies [4] have proposed that the charring rates and advanced calculation properties contained in EN 1995-1-2 be modified to ensure consistency and to account for variations in density and moisture content of softwood. Cachim and Franssen [4] noted in a recent study that the specific heat properties presented in EN 1995-1-2 are limited to timber with 12% moisture content, whilst the temperature dependant density ratio properties are inconsistently presented as a function of moisture content. To modify this Cachim and Franssen [4] proposed that a modified specific heat relationship should be adopted which is dependant on moisture content. In a similar manner it is not unreasonable to propose modifications to the effective conductivity properties of softwood to ensure that the parametric charring rates, contained in Annex A of EN 1995-1-2, are consistent with advanced calculation predictions using computational techniques. König [6] has previously proposed that consistency between parametric charring measurements in experiments and computational predictions can be achieved via subtle modifications to the conductivity versus temperature relationships proposed in Annex B of EN 1995-1-2, for standard fire exposure. König [6] noted that only those properties in excess of 350°C need to be modified as these represent the 'effective' properties of the char layer. It is phenomena in the char layer which is governed by heating rate, such as "reverse cooling pyrolysis flows", cracking and ablation [6]. Although König [6] made the observation that the thermal properties present in Annex B of EN 1995-1-2 are not appropriate for parametric fire applications and that better agreement can be seen through adaptation of the char layer conductivity, no follow on research has been conducted to quantify how exactly the conductivity of the char layer should be modified.

The remainder of this paper, through the results of numerical calibrations, attempts to quantify a relationship between fire severity (measured in terms of heating rate) and effective char layer conductivity.

### 4.2 Calibration of EN 1995-1-2 conductivity model against parametric charring rates during the heating phase of parametric fires

In numerical simulations of timber in fire the position of the 300°C isotherm is typically assumed to represent the boundary between charred and un-charred timber. Therefore, the distance from the initial timber surface and the 300°C isotherm can be considered as the depth of char ($d_{ch}$). This concept has been utilised by many researchers [4- 6]. However, for completeness this was verified by the authors for standard fire exposure using the finite element software DIANA, applying the thermal properties proposed in EN 1995-1-2 (Figure D.1), the boundary properties proposed in EC1-1-2 for standard fire and parametric fire exposure ($\alpha$=25/35 W/m$^2$.K; $\varepsilon$=0.7) and a boundary heating condition corresponding with that of the ISO834 curve. The resulting FEA prediction of the depth of char is compared with the one dimensional charring rate of softwood (from EN 1995-1-2) in Figure D.3. In the simulation densities of 300 and 450kg/m$^3$ are compared for a constant moisture content of 12%. These are benchmarked against the depth of char derived using the 1D charring rates for high density softwood tabulated in EN 1995-1-2.

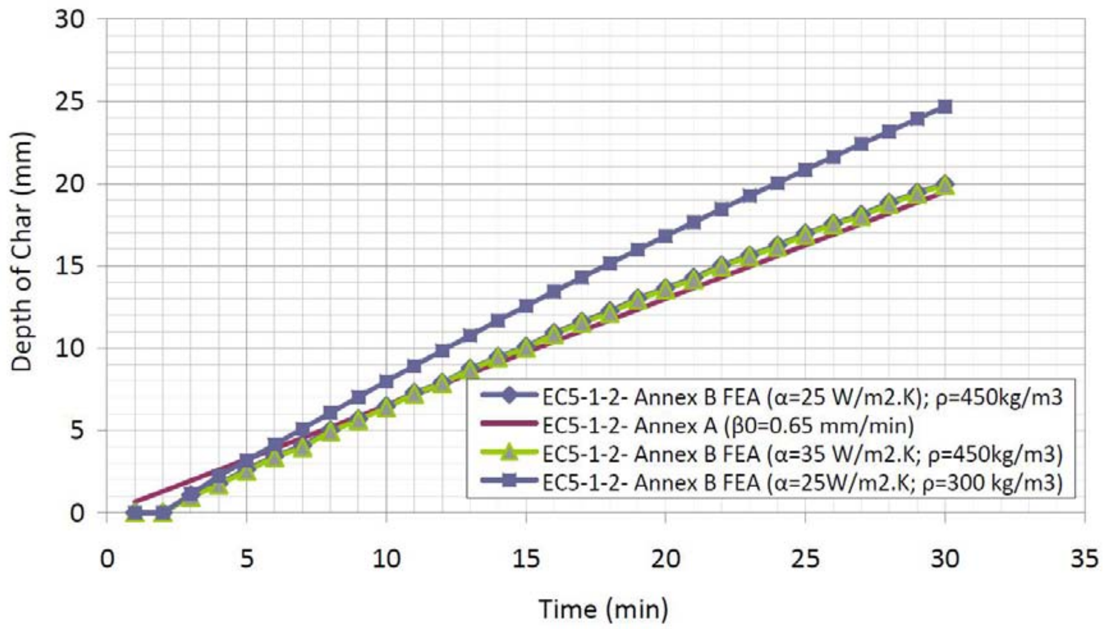


**Figure D.3. Predicted depth of char using FEA (300°C isotherm position) and EN 1995-1-2 standard fire exposure charring rates (ω=12%)**

To achieve consistency between advanced calculation predictions (FEA) and the simple parametric charring rates proposed in EN 1995-1-2, it is proposed that the modified conductivity model takes the form shown in Table D.1. This is based on early work performed by König [6] and implies (as noted previously) that only the 'effective properties' of the char layer are affected by heating rate.

**Table D. 1- Proposed conductivity model modification for softwood**

| Temperature (°C) | Conductivity (W/m.K) |
|---|---|
| 20 | 0.12 |
| 200 | 0.15 |
| 350 | 0.07 |
| 500 | $0.09k_{\lambda,mod}$ |
| 800 | $0.35k_{\lambda,mod}$ |
| 1200 | $1.50k_{\lambda,mod}$ |

In Table D.1 $k_{\lambda,\ mod}$ is a linear modification factor, which is constant for all temperatures in excess of 350°C, but is dependant on heating rate ($\Gamma$). Initially the exact relation between heating rate ($\Gamma$) and the conductivity modification factor ($k_{\lambda,\ mod}$ ) is unknown. To determine values of $k_{\lambda,\ mod}$ an acceptance criteria must be defined. For a constant fire load density and heating rate an appropriate value of $k_{\lambda,\ mod}$ was assumed to be achieved when the position of the 300°C isotherm, calculated via FEA, and the depth of char ($d_{ch}$), determined via the EN 1995-1-2 Annex A approach, were found to reach consistent values at a pre-defined time. For the purpose of this study the pre-determined time was taken as the duration of the heating phase of a given parametric fire ($t_{max}$). This is a conservative criterion as in reality the constant charring rate phase ($t_0$) is assumed to run for a period which is consistently shorter (25% less) than the duration of the heating phase of a parametric fire [2]. To this end the procedure for determining appropriate values of $k_{\lambda,\ mod}$ can be defined as follows:

1. For a given fire load density (initially trialled as $q_{td}$= 210 MJ/m$^2$), a constant compartment thermal inertia (b=520 J/m$^2$s$^{1/2}$K corresponding to a gypsum lined compartment which would be fairly typical of a timber building) and variable heating rates (O=0.02 to 0.1m$^{1/2}$), determine the transient position of the 300°C isotherm for $k_{\lambda,\ mod}$= 1.0, $\Gamma^{-1}$, $\Gamma^{-1/2}$ and $\Gamma^{-1/3}$. Initially a $k_{\lambda,\ mod}$ of 1.0 was adopted to indicate the consequences of using standard fire exposure

thermal properties in a non-standard scenario. Later values of $k_{\lambda,\ mod}$ were assumed to follow a power relationship.

2. Compare the FEA position of the 300°C isotherm with the depth of char ($d_{ch}$) determined via the Annex A method in EN 1995-1-2, for the given heating rate (Γ) and variations in $k_{\lambda,\ mod}$, at $t_{max}$ minutes
3. Determine, via regression using a power line of best fit, the value of $k_{\lambda,\ mod}$ which results in consistency with the position of the 300°C isotherm and the depth of char ($d_{ch}$) determined via the Annex A approach after $t_{max}$ minutes., for a given value of Γ.
4. Repeat the process for variations in heating rate (Γ/O) whilst maintaining a constant fire load density. This yields a relationship between heating rate and $k_{\lambda,\ mod}$ for a given fire load density.

This process is shown graphically in Figure D.4 for Γ=19.9. This example was chosen as it represents an opening factor of 0.08 which, for the assumed compartment geometry, presents an obvious deviation between parametric and standard fire charring rates. It can be seen that, using regression, the computationally predicted and Annex A depth of char at $t_{max}$ minutes are aligned for $k_{\lambda,\ mod} = 0.353\ (\Gamma^{-0.35})$.

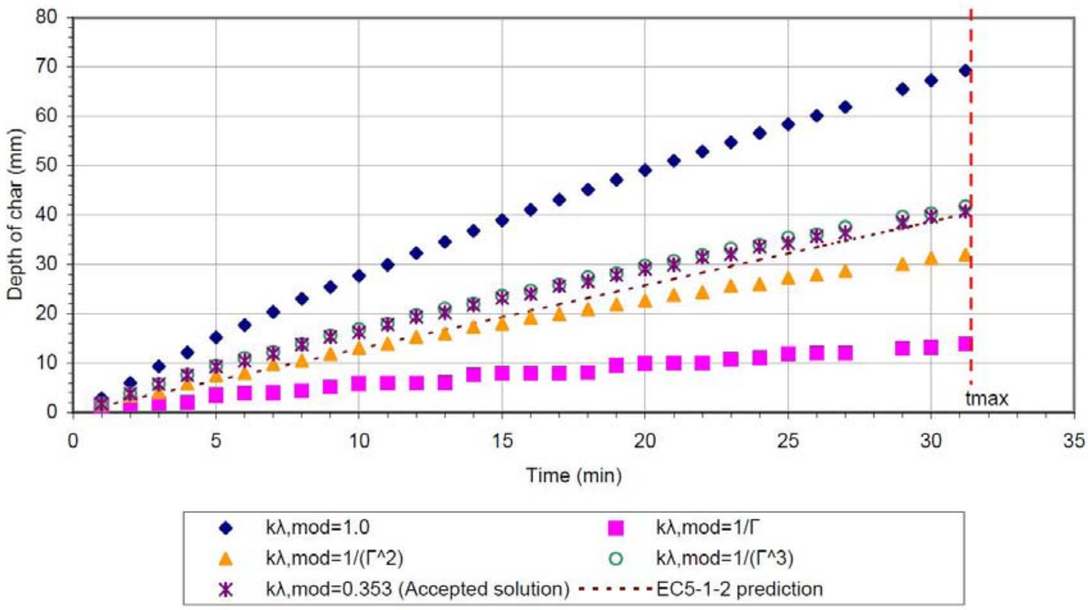


**Figure D.4. The impact of assumed $k_{\lambda, mod}$ on the transient char depth for Γ=19.9 and $t_{max}$=31mins**

The numerical simulations were performed assuming one dimensional heat flow, using the finite element model DIANA and the properties shown in Table D.2. In the direction of heat flow, elements 0.5mm in size were adopted.

**Table D.2. Important modelling properties**

| | |
|---|---|
| Softwood density | 300 kg/m$^3$ |
| Moisture content | 12% |
| Density ratio | EC5-1-2 Annex B |
| Specific heat | EC5-1-2 Annex B |
| Boundary emmisivity | 0.7 |
| Boundary convection coefficient | 35 W/m$^2$.K |

A transient non-linear analysis was performed with explicit time steps of 3 seconds and an Euler backward time integration scheme. In the case of the EN 1995-1-2 parametric charring model, a density modified 1D standard fire charring rate ($\beta_{\omega, \rho,0}$) was adopted as a density of 300kg/m$^3$ was used in simulations instead of 450kg/m$^3$. This modified charring rate was based on the work undertaken by Cachim and Franssen [4] who note that the tabulated values of charring rate in EN 1995-1-2 are only valid for densities of 450kg/m$^3$ and moisture contents of 12%. This has been confirmed in Figure D.3. The density and moisture modified charring rate ($\beta_{\omega,\rho,0}$) are given by Cachim and Franssen [4] as per Equation D.3. Key parameters input into the EN 1995-1-2 Annex A method are summarised in Table D.3.

$$b_{v,r,0} = b_0 \left( \frac{1.12}{1+w_f} \right)^{1.5} \sqrt{\frac{450}{r_0}} = 0.65 mm/\min \times 1. \times \sqrt{\frac{450}{300}} \approx 0.8 mm/\min$$

**[Equation D.3]**

Once the procedure has been adopted for every increment of opening factor (O), a series of $k_{\lambda,\ mod}$ values are apparent for different variations in Γ. The results yield a unique relationship between heating rate (Γ) and $k_{\lambda,\ mod}$, for a given fire load density ($q_{td}$) of 210 MJ/m$^2$, which is shown in Figure D.5.

**Table D.3. Summary of parametric charring parameters used in calibration**

| | **Opening factor (m$^{1/2}$)** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 0.02 | 0.03 | 0.04 | 0.05 | 0.06 | 0.07 | 0.08 | 0.09 | 0.1 |
| b (J/m$^2$s$^{1/2}$K) | 520 | | | | | | | | |
| Γ (dimensionless) | 1.24 | 2.80 | 4.98 | 7.78 | 11.20 | 15.24 | 19.91 | 25.19 | 31.10 |
| $\beta_{\rho,0}$ (mm/min) | 0.796 | | | | | | | | |
| $\beta_{par,1D}$ (mm/min) | 0.85 | 1.01 | 1.11 | 1.17 | 1.22 | 1.26 | 1.28 | 1.30 | 1.32 |

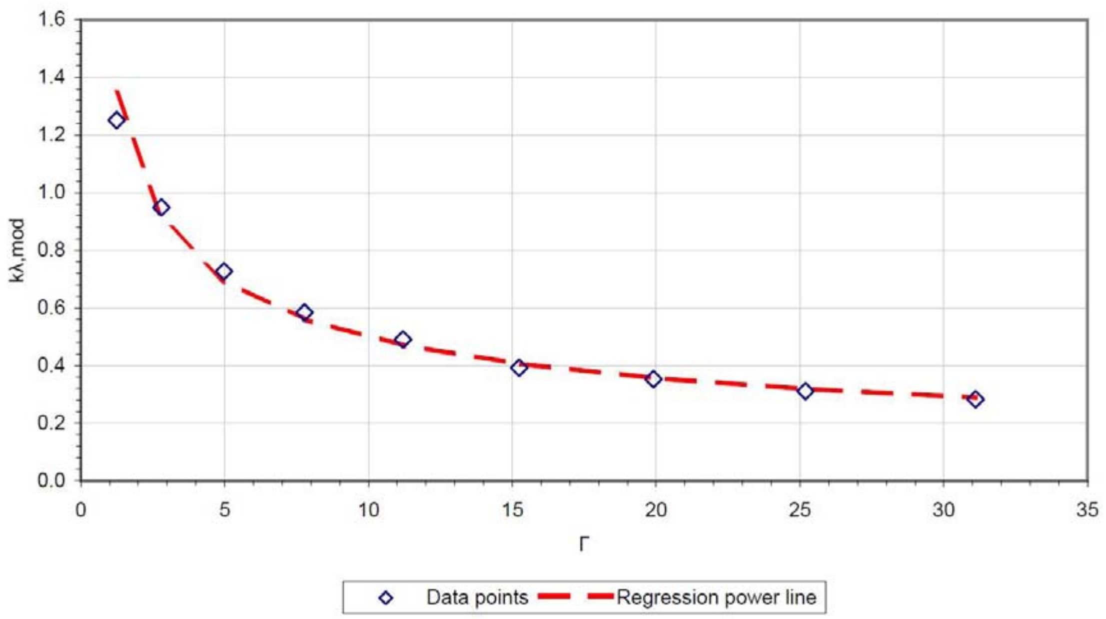


**Figure D.5. Established relationship between Γ and $k_{\lambda,\,mod}$ for $q_{td}$=210 MJ/m²**

A regression line of best fit has been added which is given by Equation D.4.

$$k_{l,\mathrm{mod}} = 1.45\Gamma^{-0.48} \qquad \textbf{[Equation D.4]}$$

Using Equation D.4, again assuming a fire load density ($q_{td}$) of 210 MJ/m$^2$, predictions of transient depth of char development can be performed using the model for various values of Γ. This is presented simply by plotting transient FEA predicted depths of char versus the depth of char derived using Annex A (Figure D.6). The "exact match" dataset corresponds to the point at which the computed and benchmark EN 1995-1-2 depths of char are perfectly consistent. 10% upper and lower bounds relative to the "exact match" dataset are shown for completeness.

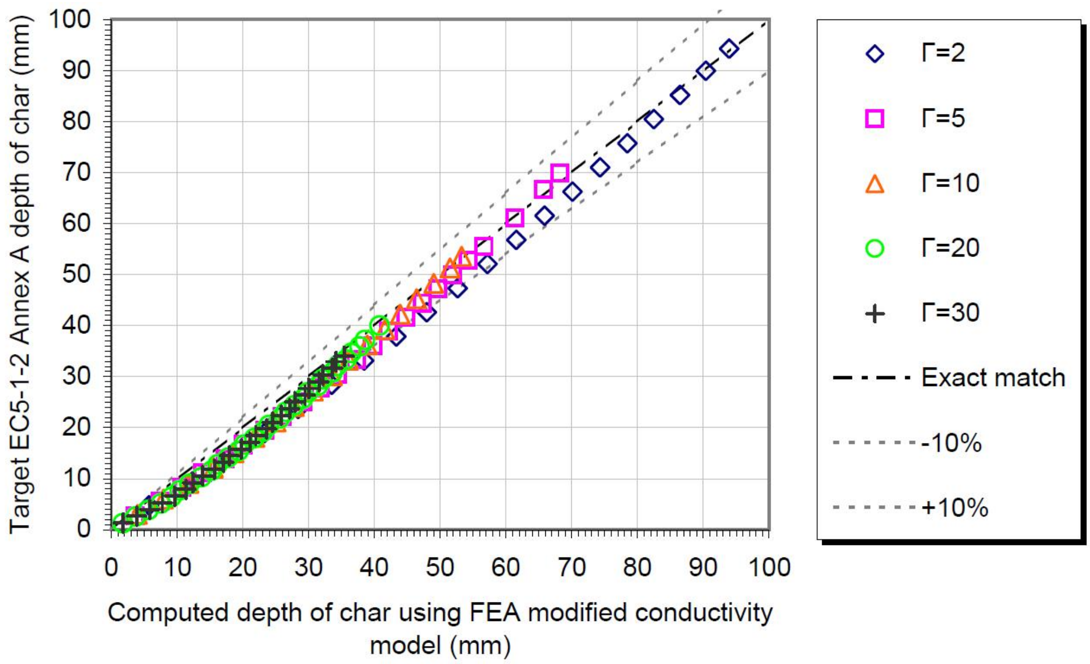


**Figure D.6. Computed transient depth of char using modified conductivity properties versus EN 1995-1-2 benchmark parametric charring approach ($q_{td}$=210 MJ/m$^2$ $\rho_0$ = 300 kg/m$^{3,}$ ω=12%)**

### 4.3 Extensions to other densities and moisture contents

Given that the conductivity relationship specified in EN 1995-1-2, and thus the modifications proposed to it, are both independent on density and moisture content, it follows that the proposed modified conductivity model should be applicable to alternative magnitudes of timber density and moisture content. The proposed conductivity modifications were calibrated initially on the basis of 12% moisture content and softwood density 300kg/m$^3$. Any changes in these values would result in variations to the enthalpy versus temperature function that describes the heat capacity of timber. Using the modified specific heat model proposed by Cachim and Franssen [4] for variations in moisture content (shown in Table D.4), in conjunction with variations in timber density, the validity of the proposed modified conductivity model can be assessed for different densities and moisture contents.

**Table D.4. Moisture content modified specific heat proposed by Cachim and Franssen [4]**

| Temperature (DegC) | Density ratio, G | Specific heat capacity of wood (EC5-1-2) (J/kg.K) | Cachim and Franssen moisture modified specific heat (J/kg.K) |
|---|---|---|---|
| 20 | $1+\omega$ | 1530 | $(1210+4190\omega)/G$ |
| 99 | $1+\omega$ | 1770 | $(1480+4190\omega)/G$ |
| 99 | $1+\omega$ | 13600 | $(1480+114600\omega)/G$ |
| 120 | 1.00 | 13580 | $(2120+95500\ \omega)/G$ |
| 120 | 1.00 | 2120 | 2120/G |
| 200 | 1.00 | 2000 | 2000/G |

The ‘validity’ of the proposed conductivity changes can be assessed again via benchmarking against the Annex A parametric charring equations, using the density and moisture content modified one dimensional charring rates previously discussed. In a similar manner to Figure D.6 the resulting plots of transient FEA depth of char (using the modified conductivity model) can be plotted against the benchmark EN 1995-1-2 approach, with appropriate changes for the impacts of moisture content and density on the one dimensional charring rate. Plots are shown for densities of 450 and 600 kg/m$^3$, as well as an alternative moisture content of 10% (Figures D.7-D.9 respectively). Similarly 10% upper and lower bounds are shown for completeness.

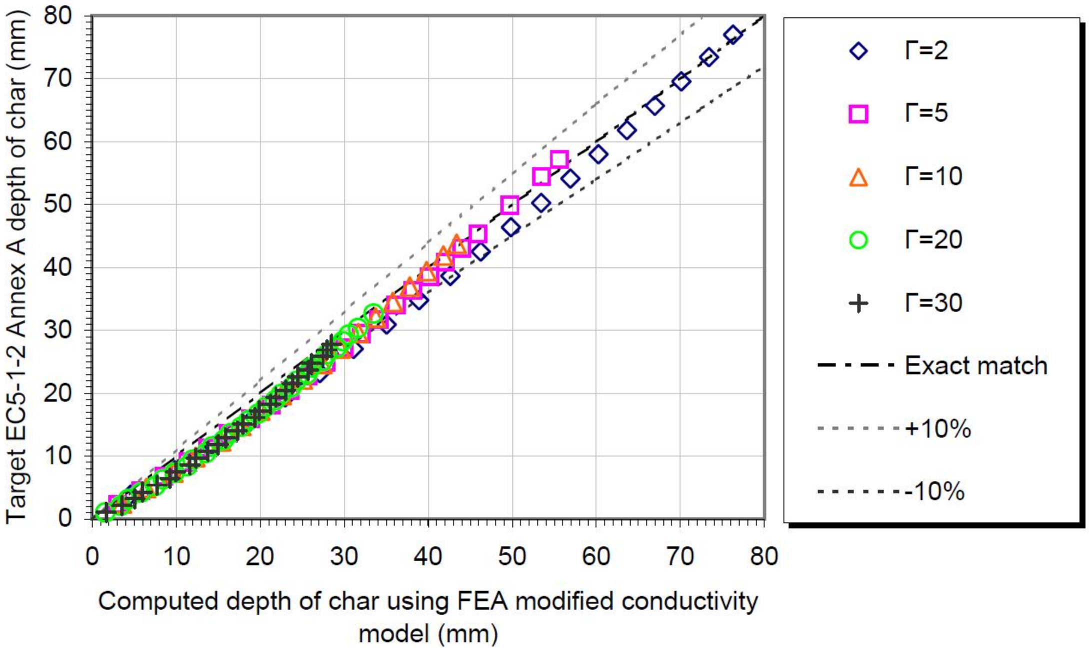


**Figure D.7. Computed transient depth of char using modified conductivity properties versus EN 1995-1-2 benchmark parametric charring approach ($q_{td}$=210 MJ/m$^2$, $\rho_0$ = 450 kg/m$^{3,}$ ω=12%)**

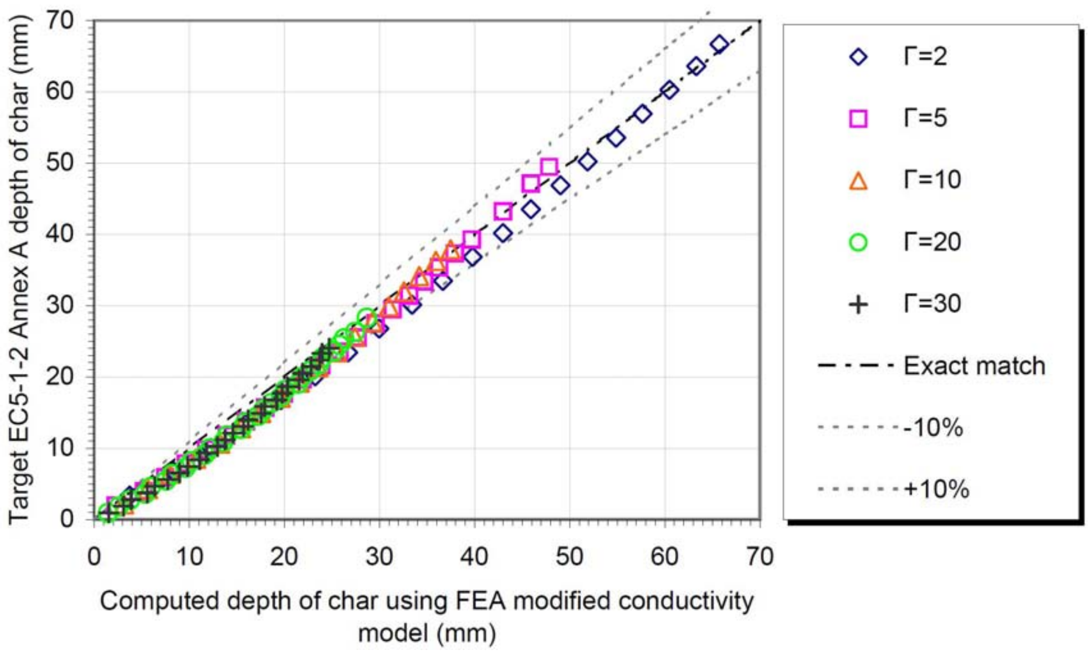


**Figure D.8. Computed transient depth of char using modified conductivity properties versus EN 1995-1-2 benchmark parametric charring approach ($q_{td}$=210 MJ/m$^2$, $\rho_0$ = 600 kg/m$^3$, ω=12%)**

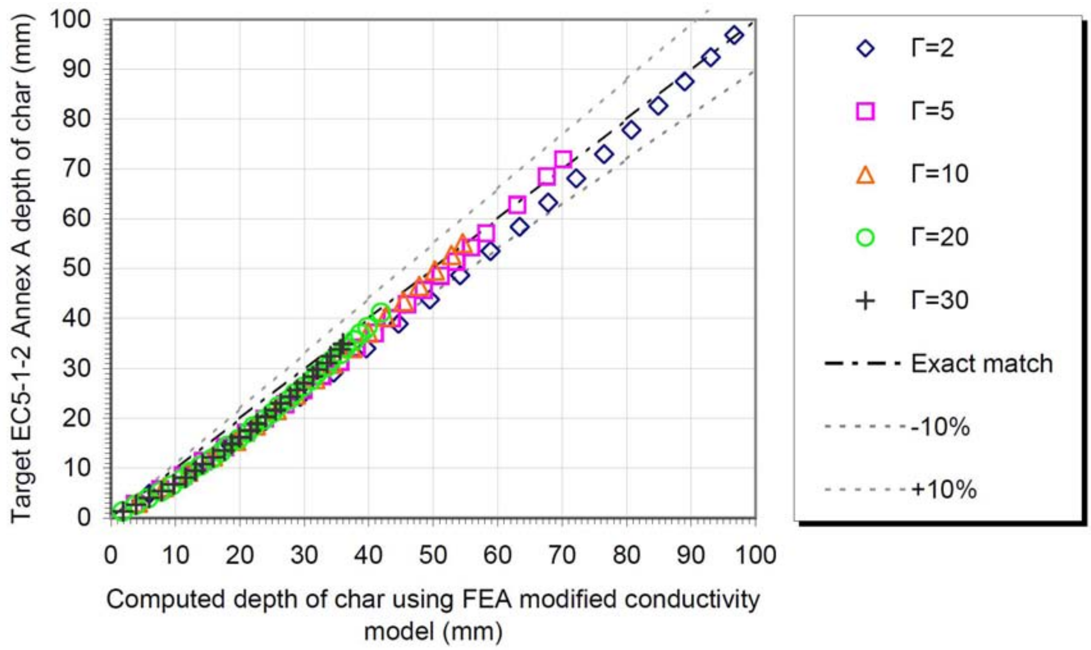


**Figure D.9. Computed transient depth of char using modified conductivity properties versus EN 1995-1-2 benchmark parametric charring approach ($q_{td}$=210 MJ/m$^2$, $\rho_0$ = 300 kg/m$^3$, ω=10%)**

It can be seen from these figures that the variations in moisture content and density have a significant impact on predicted numerical charring rates. Physically this influence is a result of an increase in heat capacitance of a constant volume element of timber, thus requiring more energy for the timber to char. For a timber specimen of density 600 kg/m$^3$ a 13.4% reduction in charring rate can be noted compared to softwood of density 450 kg/m$^3$. The latter density corresponds with the value assumed in EN 1995-1-2 for softwood with a one dimensional charring rate of 0.65mm/min. Similarly a 22% increase in charring rate can be expected for a timber specimen of density 300 kg/m$^3$ (compared to a sample of density 450 kg/m$^3$). Comparable findings can be seen for increases and decreases in moisture content. The influence of density on charring is supported both numerically in the findings of this research and also by the originators Cachim and Franssen [4].

### 4.4 Extensions to other fire load densities

It has been explained that the modified conductivity model was calibrated such that there would be 'consistency' between depth of char predicted using FEA and Annex B at the end of the heating phase of a parametric fire ($t_{max}$). This alignment was achieved for a defined value of fire load density ($q_{td}$) of 210 MJ/m$^2$. It is important to note that two important aspects govern the heating phase of a parametric fire. Firstly the overall severity, measured in terms of growth and peak temperature are influenced by boundary inertia and ventilation, i.e. $\Gamma$. Secondly, the time to reach peak temperature and duration of the heating phase are governed by the fire loading. The parametric fire curve is constructed such that the heating time temperature path is unaffected by fire loading density and is governed entirely by $\Gamma$. However, this development phase is truncated according to the available fuel ($q_{td}$) governing both the time to and overall peak temperature. Clearly, if the proposed conductivity changes were applied to fire load densities ($q_{td}$) in excess of the calibrated 210 MJ/m$^2$ then the resulting computed depth of char would yield un-conservative predictions towards the end of the heating phase of a fire (relative to the EN 1995-1-2 parametric approach). Similarly, where fire load densities below that of 210 MJ/m$^2$ are adopted, the depth of char computed at the end of the heating phase would be very conservative. This concept is illustrated in Figure D.10.

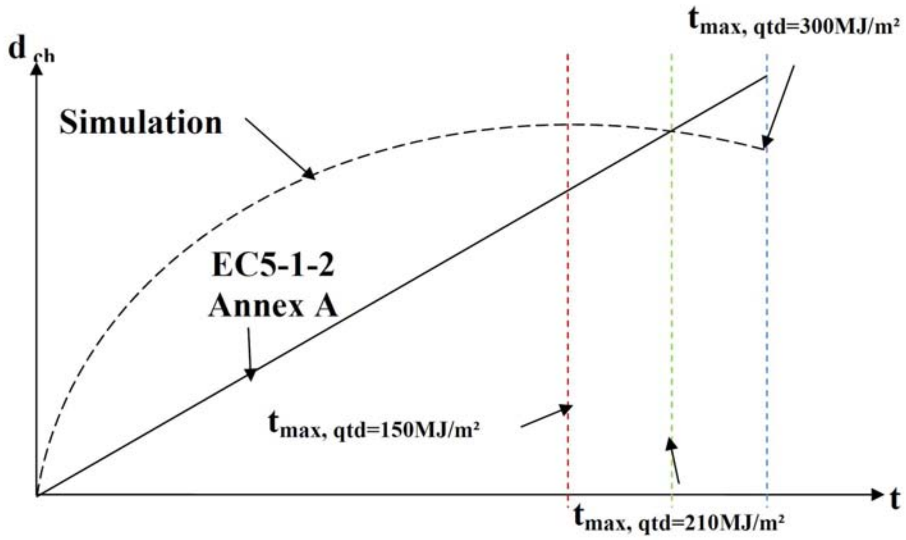


**Figure D.10. Illustration of requirement for a fire load factor**

It then follows that, for the given method of calibration, any proposed changes to the 'effective' conductivity should not only consider the impact of heating rate but also that of the heating duration. This modification is necessary as a by-product of the method chosen to determine $k_{\lambda,mod}$ (i.e. calibration at a given fire load) but does not infer that in reality the charring rate in natural fires is dependant upon fire load density.

To account for the impact of fire load density it is proposed that the conductivity modification factor ($k_{\lambda,\ mod}$) should be revised to incorporate not only a heating rate factor ($k_{\Gamma.mod}$) but also a fire load density factor ($k_{qtd,mod}$) as shown in Equation D.5.

$$k_{l,\mathrm{mod}} = k_{\Gamma,\mathrm{mod}} k_{qtd,\mathrm{mod}} \qquad \textbf{[Equation D.5]}$$

In this instance the effect of heating rate is incorporated via the previously defined 'power' function ($k_{\Gamma.mod}$) and the impact of fire load is incorporated via a new function ($k_{qtd,mod}$). These functions are shown in Equations D.6 and D.7.

$$k_{\Gamma,\text{mod}} = 1.5\Gamma^{-0.48} \quad \text{and} \quad k_{qtd,\text{mod}} = \sqrt{\frac{q_{td}}{210}}$$ **[Equation D.6 and D.7]**

The form of this function was chosen on the basis of the density variation function proposed by Cachim and Franssen [4]. In a similar manner, the ‘validity’ of the modification factor is measured relative to the EN 1995-1-2 charring method. Fire load densities ($q_{td}$) of 100 and 300 MJ/m$^2$ were chosen and the resulting comparative plots are shown in Figure D.11 and D.12 respectively. Again, 10% and 20% upper and lower bounds are included for clarity.

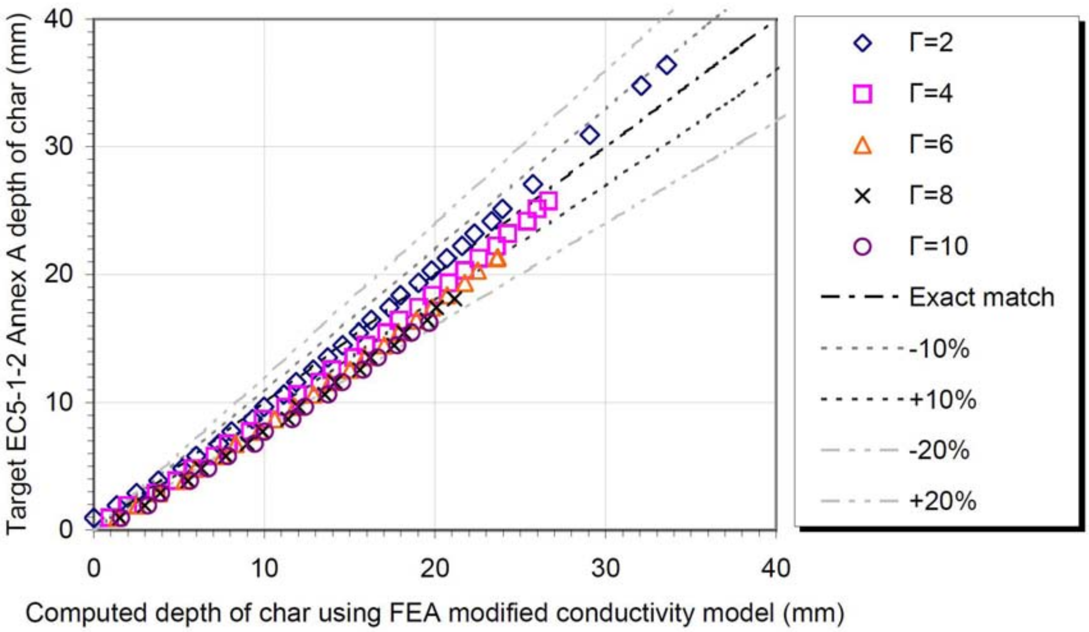


**Figure D.11. Computed transient depth of char using modified conductivity properties versus EN 1995-1-2 benchmark parametric charring approach ($q_{td}$=100 MJ/m$^2$, $\rho_0$ = 450 kg/m$^3$, $\omega$=12%)**

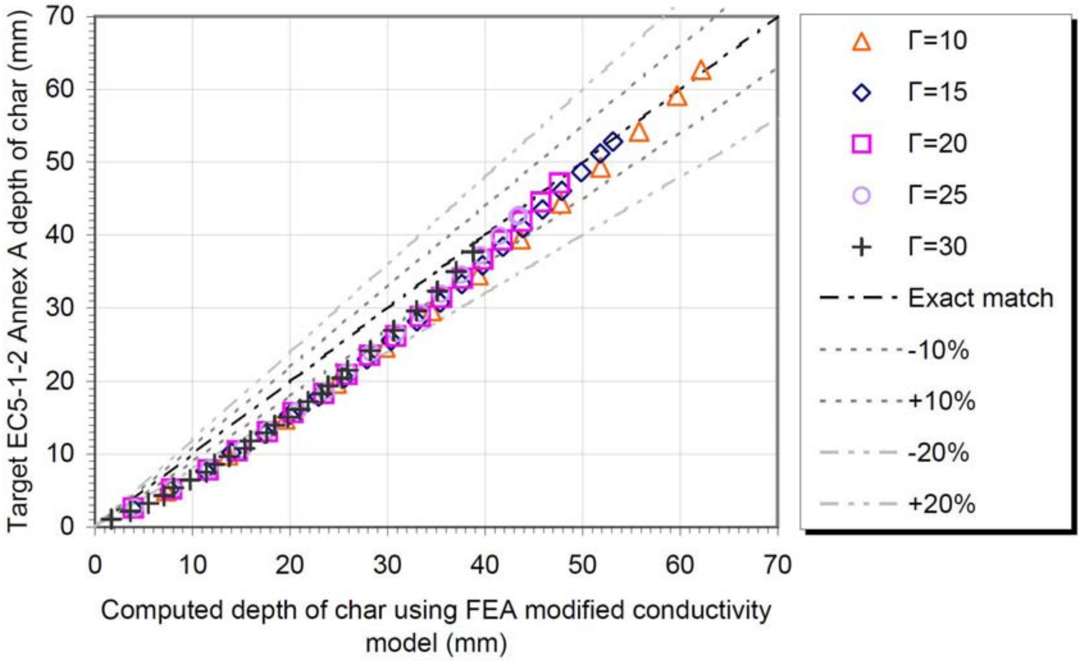


**Figure D.12. Computed transient depth of char using modified conductivity properties versus EN 1995-1-2 benchmark parametric charring approach ($q_{td}$=300 MJ/m$^2$, $\rho_0$ = 450 kg/m$^3$, ω=12%)**

5. Discussion

The performance based fire design of timber structures exposed to time temperature responses differing from that of the ISO834 heating curve [3] is currently not possible as the material aspects of timber behaviour in fire are complex. There have been calls in the literature [6] for mass and heat transfer models to be developed to accurately simulate temperature development in timber members. The properties input into such models however would still be 'effective' as phenomena such as ablation and cracking would not be correctly considered in any such models. With practitioners in mind it thus seems logical to persist with heat transfer models and extend the 'effective' properties to consider the impacts of heating rate and other important factors.

This paper has shown one modification to the 'effective' conductivity values proposed in Annex B of EN 1995-1-2 which could make the properties applicable to a limited number of parametric fires, during the heating phase of temperature development. The properties have been modified to account for fire load density and heating rate (refer to Figure D.13).

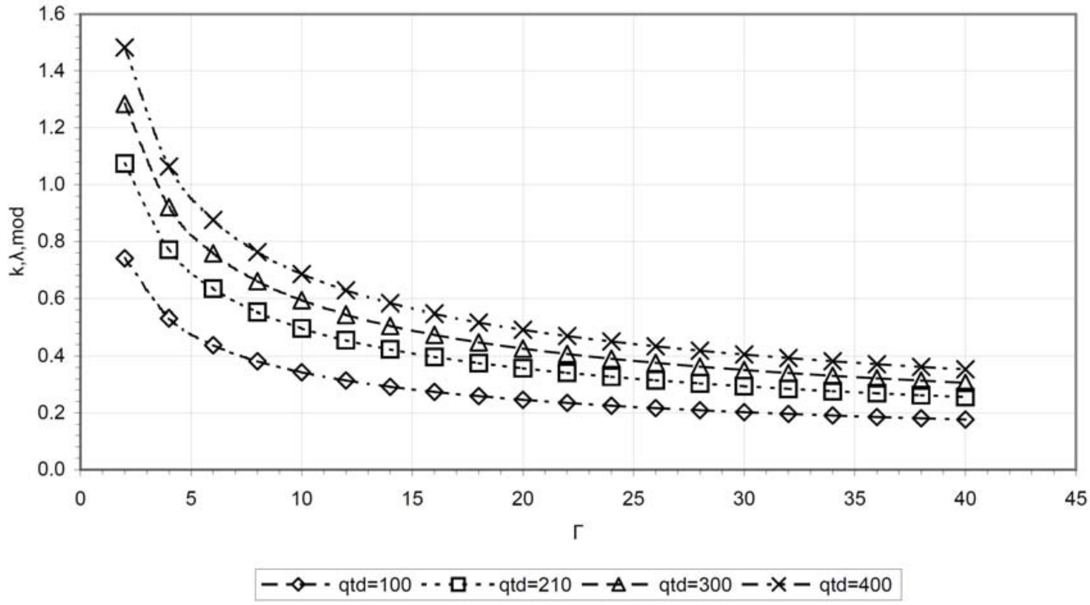


**Figure D.13. $k_{\lambda,mod}$ as a function of both fire load (MJ/m$^2$) and heating rate (Γ)**

The impact of this modification on the conductivity vs. temperature relationship for solid timber is shown in Figure D.14.

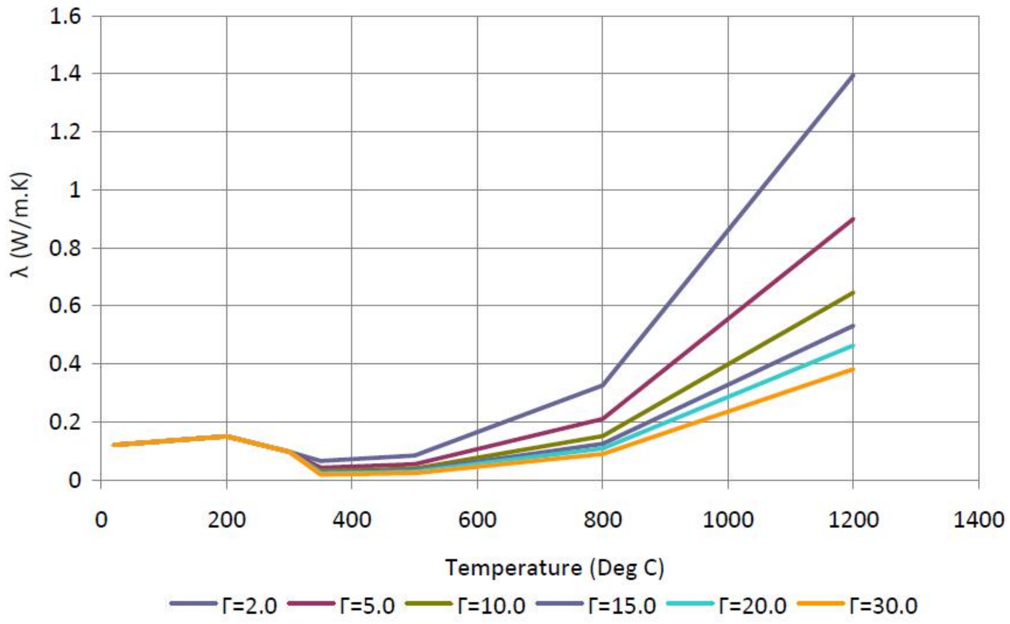


**Figure D.14. Conductivity versus temperature for different heating rates (Consistency at $t_0$, qtd = 210 MJ/m$^2$)**

The credibility of the proposed model is largely dependant on the accuracy of the EN 1995-1-2 parametric charring equations, which are understood to be empirical. Further to this, the model has been calibrated on the basis of charring depths and not temperature development within timber cross sections. As a result the proposed properties are centred upon two fundamental assumptions. Firstly, the transient timber temperatures in excess of 300°C do not need to be accurately predicted as above this temperature timber has no strength. It thus follows that the structural consequences are unaffected if model predictions are incorrect at temperatures in excess of 300°C. Secondly, the thermal properties of un-charred wood (specifically conductivity) are not dependant on heating rate. Thus, if the position of the 300°C isotherm can be accurately determined, then all subsequent lower temperature contours should also be correct. The validity of this assumption has been implied by König [6] in recent studies. The proposed modified conductivity model would undoubtedly benefit from careful validation against experimental results. However non-standard fire tests performed on timber structures, which have been appropriately instrumented, are scarce. The experiments that have been conducted have largely been undertaken in furnaces where heat transfer via convection would be less pronounced than for natural fires [7]. Experimental data is particularly important for validation of the key assumption noted above regarding temperature development at temperatures below 300°C.

The proposed conductivity changes have been simply implemented in the commercial finite element code DIANA, via a two dimensional heat transfer formulation, with one dimensional heat flow. The resulting predicted depths of char are shown to be consistent with current codified charring equations for parametric fires. Where there are inconsistencies the results are generally conservative and within a 10% error band. Predictably, the modified conductivity model yields the most promising results for the fire load density at which it was initially calibrated (210 MJ/m$^2$). However the proposed extension to alternative fire load densities has yielded initially positive conservative findings. It is important to stress that the fire load modification factor ($k_{qtd,mod}$) is a mathematical concept which is necessary due to the calibration method adopted and does not imply that charring rates are physically dependant upon fuel levels.

At present, the proposed conductivity model is limited to the heating phase of fires only. Extensions to the cooling phase to check consistency with the EN 1995-1-2 parametric charring method is at present work in progress. It is however anticipated that phenomena like char oxidation may not be appropriately addressed by the proposed changes. This may result in un-conservative predicted depths of char during the cooling phase of parametric fires. As a result in may be necessary to consider alternative timber thermal properties for the cooling phase of fires.

Critical analysis of the model indicates some inconsistencies with the EN 1995-1-2 parametric charring approach. For example, the heating rate dependant function increases beyond unity at heating rates corresponding to that of the ISO834 heating curve (i.e. $\Gamma$=1.0). This implies that the conductivities proposed by EN 1995-1-2 for standard fire exposure need to be increased for parametric fires. This at first glance appears logical as, although the parametric curve heating rate would be consistent with that of the standard fire curve, the heat transfer via convection would be more pronounced. This is explicitly acknowledged in the loading code (EC1-1-2) where different coefficients are adopted for the two fire types. However, through further investigation, it can be seen that the convection coefficient ($\alpha$) has only a nominal influence on the development of the char layer, as shown in Figure D.15. As a result further calibrations of $k_{\Gamma,mod}$ are required for values of $\Gamma$ between 0.1 and 2.0.

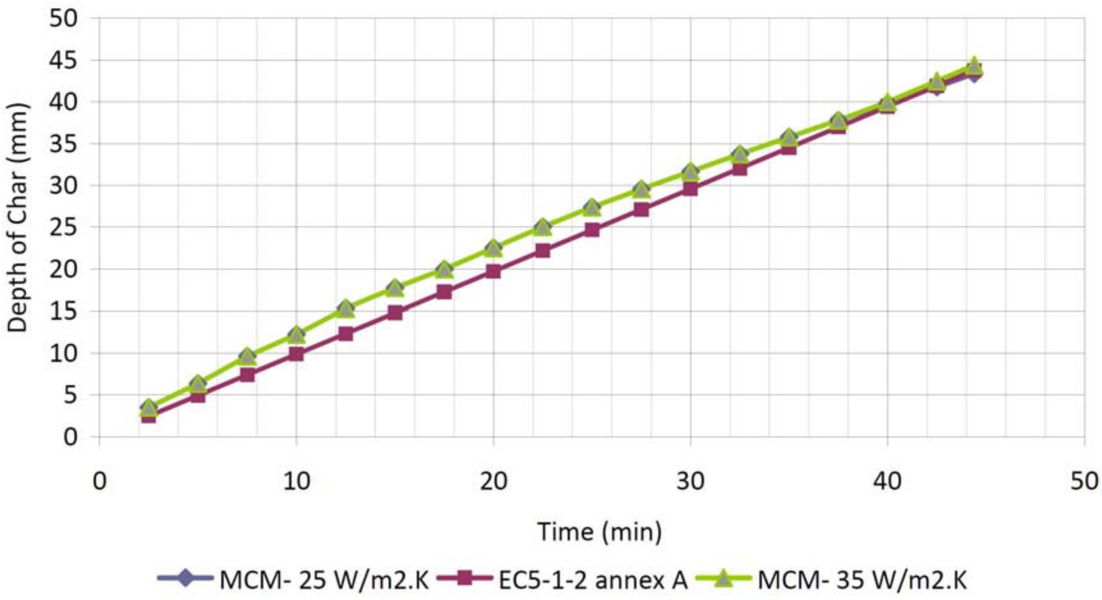


**Figure D.15. Comparison of FEA and Annex A depths of char for $\alpha$= 25 and 35 W/m$^2$.K ($\Gamma$=10, $q_{td}$=210 MJ/m$^2$, $\rho_0$ = 450 kg/m$^3$, $\omega$=12%)**

In addition, for the purpose of this study, the Annex A [2] equations have been applied outside of their validated scope of applicability. In reality the scope of application of the proposed conductivity changes is limited by the small number of parametric fires which are described in the parametric charring equations (i.e. for $t_0$ between 20 and 40mins). The calibration of charring depths at the end of the heating phase (up to $t_{max}$) of parametric fires rather than the 'constant charring phase' ($t_0$) results in some inconsistency with EN 1995-1-2. However, calibrating on this basis ensures conservative approximations of the 300°C isotherm position predicted by FEA, compared to the depth of char predictions derived from the Annex A method. This is because EN 1995-1-2 assumes that charring rate begins to reduce before peak temperatures in the compartment are reached. If however consistent depths of char between the proposed modified conductivity model and Annex A charring rates are sought for the end of the constant charring phase ($t_0$), then the modification factor describing fire load density ($k_{qtd,mod}$) can be simply modified as per Equation D.8.

$$k_{qtd,\mathrm{mod}} = \sqrt{\frac{0.75 q_{td}}{210}}$$ **[Equation D.8]**

This simple modification is due to the fact that the duration of the constant charring phase ($t_0$) is 75% of the overall heating phase of the parametric fire ($t_{max}$) and the functions describing these durations are a linear variation of fire load ($q_{td}$), for a constant opening factor (O).

## 6. Conclusions

The work presented herein has attempted to achieve consistency between the advanced calculation thermo-physical properties and un-protected timber parametric charring methods proposed in EN 1995-1-2, during the heating phase of fires. The research proposes modifications to the current thermal properties which couple specific heat modifications proposed by Cachim and Franssen [4] with new conductivity properties that are not only dependant on temperature, but also heating rate and fire loading. The key conclusions from the research can be summarised as follows:

- Heating rate and fire load density dependant linear modifications have been found which can be conservatively applied to conductivity values proposed in Annex B of EN 1995-1-2, for temperatures in excess of 350°C. These factors result in consistent depths of char with those predicted via the calculation method presented in Annex A of EN 1995-1-2, when applied in a finite element heat transfer model. Where there are inconsistencies between the two approaches they are generally conservative and fall within the $^{+}/_{-}$ 10% bounds relative to the EN 1995-1-2 benchmark.
- Although the calibrations have been performed on charring durations ($t_0$) outside of the validated range of the EN 1995-1-2 Annex A method, the scope

of application of the proposed modifications is limited by the scope of the Annex A parametric charring approach i.e. $t_0 \leq 40$ minutes. Further experiments for an extended range of heating rates and fire loads would extend both the range of application of the annex A method and also the proposed conductivity modifications.

- The consequences of the changes in conductivity values on predicted timber temperature profiles have not been studied, however it is assumed that temperature predictions below 300°C will remain unaffected. This would make the properties applicable to coupled thermal and mechanical finite element models involving parametric fires. However, to provide confidence in this assumption, experimental data is required for a range of parametric fires. Comparisons should then be made between predicted and measured temperature profiles in timber members exposed to parametric fires.
- At present the proposed model is only applicable in the heating phase of parametric fires. However work is in progress to determine its validity when applied to the cooling phase of parametric fires. Un-considered phenomena particularly pronounced in the cooling phase, such as char oxidation, may render model predictions un-conservative in this phase. In addition to this, where the model is to be coupled with mechanical response, consideration should be made as to the strength characteristics of cooling timber, which are likely to be different to heated timber.
- This paper has presented a framework for how ‘effective’ thermal properties of timber can be extended to incorporate parametric fires through consideration of heating rates and fire load densities. The proposal is not intended to be a final solution, however it represents a meaningful first step

and negates the need for complex mass and heat transfer models. A more accurate series of 'effective' properties could be developed on the basis of experiments, supported by numerical studies. Alternatively the proposed model could be further developed using available experimental data.

## 7. Acknowledgments

Mr. Hopkin would like to acknowledge EPSRC and BRE Global for their support.

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# APPENDIX E EFFECT OF FIRE-INDUCED DAMAGE ON THE UNIAXIAL STRENGTH CHARACTERISTICS OF SOLID TIMBER: A NUMERICAL STUDY

**Full Reference**

HOPKIN, D., EL-RIMAWI, J., SILBERSCHMIDT, V., and LENNON, T., Effect of fire-induced damage on the uniaxial strength characteristics of solid timber: A numerical study. *Journal of Physics: Conference Series,* **305**(1), doi: 10.1088/1742-6596/305/1/012039.

**Abstract**

The advent of the structural Eurocodes has allowed civil engineers to be more creative in the design of structures exposed to fire. Rather than rely upon regulatory guidance and prescriptive methods engineers are now able to use such codes to design buildings on the basis of credible design fires rather than accepted unrealistic standard-fire time-temperature curves. Through this process safer and more efficient structural designs are achievable. The key development in enabling performance-based fire design is the emergence of validated numerical models capable of predicting the mechanical response of a whole building or sub-assemblies at elevated temperature. In such a way, efficiency savings have been achieved in the design of steel, concrete and composite structures. However, at present, due to a combination of limited fundamental research and restrictions in the UK National Annex to the timber Eurocode, the design of fire-exposed timber structures using numerical modelling techniques is not generally undertaken.

The ‘fire design’ of timber structures is covered in Eurocode 5 part 1.2 (EN 1995-1-2). In this code there is an advanced calculation annex (Annex B) intended to facilitate the implementation of numerical models in the design of fire-exposed timber structures. The properties contained in the code can, at present, only be applied to standard-fire exposure

conditions. This is due to existing limitations related to the available thermal properties which are only valid for standard fire exposure. In an attempt to overcome this barrier the authors have proposed a ‘modified conductivity model’ (MCM) for determining the temperature of timber structural elements during the heating phase of non-standard fires. This is briefly outlined in this paper. In addition, in a further study, the MCM has been implemented in a coupled thermo-mechanical analysis of uniaxially loaded timber elements exposed to non-standard fires. The finite element package DIANA was adopted with plane-strain elements assuming two-dimensional heat flow. The resulting predictions of failure time for given levels of load are discussed and compared with the simplified ‘effective cross section’ method presented in EN 1995-1-2.

## 1. Introduction

Timber, like other structural materials such as concrete and steel, has its own Eurocode (Eurocode 5 part 1.2) for the structural fire design of buildings [1]. However, unlike other fire parts of the Eurocodes it is not widely adopted due to its inherent limitations. With the exception of a single annex, the timber Eurocode (EN 1995-1-2) is only applicable to standard-fire exposure [2]. This exposure condition describes an ever increasing time-temperature regime, typically used in the fire-resistance testing of materials and systems. Annex A of EN 1995-1-2 gives guidance on the charring rates of initially un-protected timber members exposed to non-standard fires. In this instance these non-standard fires refer to parametric fires [3], which are a marginally more realistic means of specifying a time-temperature response, based upon the specific ventilation conditions and fire loadings of a given building/enclosure. However, in the UK the use of Annex A is prohibited by the national annex to EN 1995-1-2.

At present, in the UK (and most of Europe) timber structures are designed for fire resistance using a combination of prescriptive rules and fire-resistance testing, using the concept of fire-resistance ratings. These fire ratings are based upon standard-fire exposure and bear little relation to real fires in real buildings. There are moves towards the concept of performance-based fire design for structures, whereby a given building is designed to survive the entire duration (including cooling) of a design fire, which is dependent upon the building's use and geometry. Much of the ability to design structures specifically for design fires (such as parametric fires) stems from the development and implementation of advanced numerical models capable of simulating multiple element interactions at elevated temperature. For this approach to be realised for large-section timber structures, then, firstly, accurate temperatures in timber sections need to be determined for non-standard fires and, secondly, the thermo-mechanical behaviour of timber needs to be further considered. This paper discusses these issues in more detail and also briefly proposes a solution to the problematic thermal physical aspects of timber behaviour in natural fires. Finally, a study highlighting how new thermo-physical models can be coupled with mechanical behaviour to implement simple simulations of timber exposed to non-standard fires, is presented.

## 2. Strength characteristics of solid timber

Timber, like most structural materials, undergoes some degradation in strength and stiffness at elevated temperature. However, unlike materials such as steel and concrete, this degradation for timber is over a much shorter temperature range due to its inherent combustibility. Typically, timber (both soft and hardwoods) loses all of both its strength and stiffness (regardless of grain orientation) over the temperature range of 20 to 300°C.

Many textbooks and reference documents for timber in fire have collated various strength and stiffness retention factors using numerous classical studies [4-17]. These have been summarised in Figures E.1 to E.3 for tensile strength, compressive strength and elastic modulus, respectively. In all instances the study of Konig & Walleij [11] is also shown as these properties are codified in EN 1995-1-2.

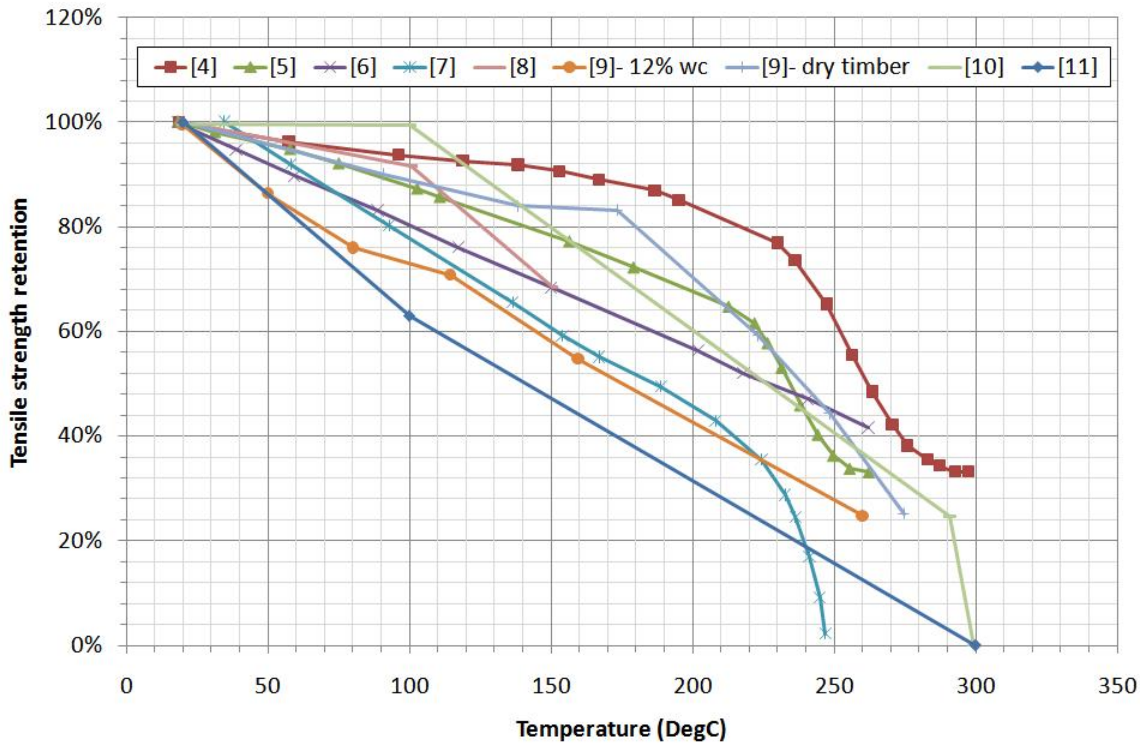



**Figure E.1. Tensile strength reduction in softwood exposed to elevated temperature (Parallel to the grain)**

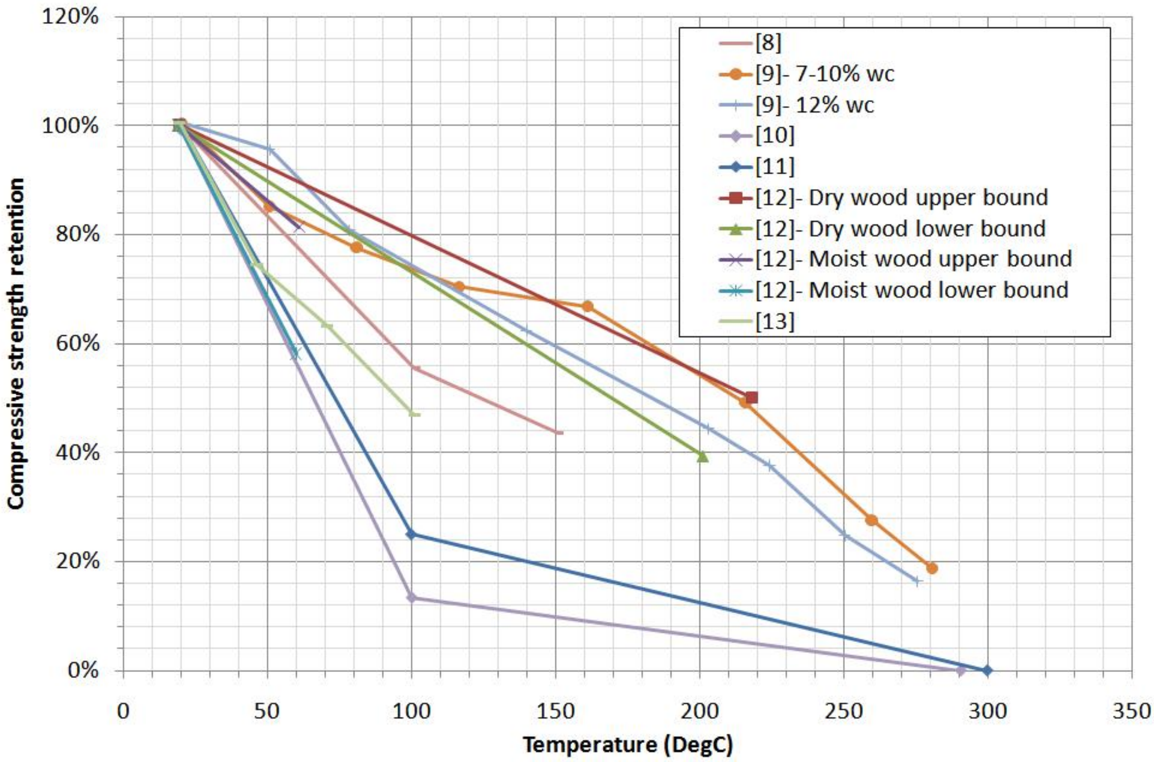



**Figure E.2. Compressive strength reduction in softwood exposed to elevated temperature (Parallel to the grain)**

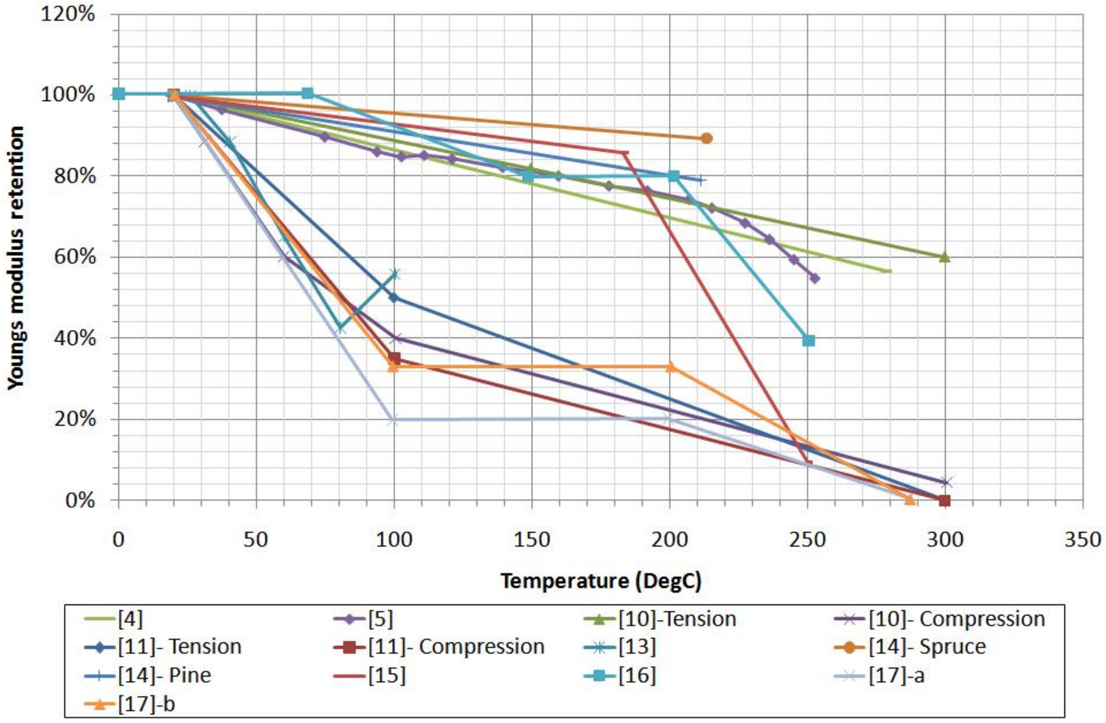



**Figure E.3. Elastic modulus reduction in softwood exposed to elevated temperature (Parallel to the grain)**

3. Overview of the modified conductivity model for timber

The design of timber structures for non-standard fire conditions is not simple since only limited test data exists on temperature development within timber structural elements under these conditions. In addition, as timber is an organic material, the thermo-physical properties are complex and depend on a number of factors. The complex phenomena present in heated timber elements are difficult to model explicitly and, hence, to date 'effective properties' are often defined. Such properties implicitly account for the effects of complex behaviour, such as the flow of pyrolosis gases and water vapour, through calibration against known temperatures, in limited experimental configurations. Konig [11] has been instrumental in initiating such a process for timber and has calibrated 'effective' thermal properties for standard fire exposure conditions.

These properties form the basis of the advanced calculation models contained in Annex B of EN 1995-1-2. However, additional studies by Konig [18] proved (both experimentally and numerically) that those properties exhibited very conservative predictions of char depth when applied to non-standard (parametric) fire conditions with heating rates in excess of that for the standard-fire curve.

Similarly, the properties from the code were shown to result in un-conservative predictions of timber temperature and depth of char for heating rates lower than that for the standard-fire curve. As a result, EN 1995-1-2 explicitly states that the thermal properties present in Annex B should only be adopted for standard-fire exposure and not parametric fire exposure.

Konig [18] previously proposed that consistency between parametric charring measurements in experiments and computational predictions could be achieved via subtle modifications to the conductivity-temperature relationships proposed in Annex B of EN 1995-1-2, for standard-fire exposure. Konig [18] noted that only those properties in excess of 350°C should be modified as they represent the 'effective' properties of the char layer. It is phenomena in this area which appears to be influenced by heating rate, such as 'reverse cooling pyrolosis flows', cracking and ablation [18]. Although Konig [18] made the observation that the thermal properties present in Annex B of EC5-1-2 were not appropriate for parametric fire applications and that better agreement could be seen through adaptation of the char-layer conductivity, no follow-on research has been conducted to quantify necessary modification of the char-layer conductivity.

In recognition of the above limitations the authors proposed a modified-conductivity model for solid timber [19], which introduces its dependence on both heating rate ($\Gamma$) and fire-load density ($q_{td}$) via a modification factor $k_{\Gamma,mod}$, thus making it possible to simulate the temperature development in timber members exposed to natural fires. The modifications proposed are outlined in Table E.1 and the equation that follows describing $k_{\Gamma,mod}$. Additional information can be found elsewhere [19].

**Table E.1. Proposed conductivity model modification for softwood**

| Temperature (°C) | Conductivity (W/m.K) |
|---|---|
| 20 | 0.12 |
| 200 | 0.15 |
| 350 | 0.07 |
| 500 | $0.09k_{\lambda,\mathrm{mod}}$ |
| 800 | $0.35k_{\lambda,\mathrm{mod}}$ |
| 1200 | $1.50k_{\lambda,\mathrm{mod}}$ |

$$k_{l,\mathrm{mod}} = k_{\Gamma,\mathrm{mod}} k_{qtd,\mathrm{mod}}$$ **[Equation E.1]**

With $k_{\Gamma,\mathrm{mod}} = 1.5\Gamma^{-0.48}$ , $k_{qtd,\mathrm{mod}} = \sqrt{\frac{q_{td}}{210}}$ and $\Gamma = \frac{(O/b)^2}{(0.04/1160)^2}$

Where: O- Opening factor ($m^{0.5}$), b- Compartment thermal inertia ($J/m^2s^{0.5}K$) and $q_{td}$- Fire load density related to the total area of the enclosure ($MJ/m^2$).

4. Numerical study

Currently, the most common procedure for the fire design of timber structures is the residual cross section method, which is popular due to its simplicity. It accounts for reduction in load-bearing capacity, caused by a fire, through consideration of the depth of char ($d_{ch}$) and depth of pyrolysis layer (often referred to as a zero-strength layer, $d_0$) as a function of time. The depth of char is most commonly calculated for standard-fire exposure and is based on charring rates (often denoted as β).

The code assumes such charring rates are independent of both density and moisture content of timber. However, studies have shown [20] that they are consistent with a timber moisture content of 12% and an initial density of 450 kg/$m^3$. The zero-strength layer ($d_0$) is nominally 7 mm, as defined by the code. The concept of the zero-strength exists to simplify the effective strengths of an infinite number of timber layers at various temperatures located between the char line (typically defined by the 300°C isotherm), where timber has no apparent strength, and the ambient (typically 20°C) isotherm, where timber retains all of its cold-design strength.

The EN 1995-1-2 residual cross-section method has its benefits, namely that it is simple and can be applied to both standard and parametric fire exposure. However, the method is limited to isolated members without consideration of the beneficial aspects of behaviors present when entire structures are subject to fire. The adoption of coupled thermo-mechanical whole-timber building models is potentially a much more efficient way to design fire-exposed timber buildings, especially when natural fires are considered. For this to be realised, the aforementioned modified-conductivity model must be coupled with mechanical behavior (with associated strength and stiffness reduction with increasing temperature). Before more complex behaviors associated with bending, biaxial bending and buckling are considered, it is first appropriate to observe how uniaxially loaded members behave when exposed to fires, both standard and natural. The ability of the MCM to be adopted in performance-based fire designs is measured by comparing the predicted failure times of uniaxially loaded timber members using both FEA simulations and simple empirical calculation models (reduced cross-section method) contained in BS EN 1995-1-2. Once this step has been taken it will be possible in future research to explore the potential for whole-building design using coupled thermo-mechanical modeling and the authors proposed MCM [19].

### 4.1. Modelling approach

To assess the applicability of the proposed MCM in the design of timber structures exposed to natural fires, it is necessary to confirm that simulations give results consistent with simple examples, for which solutions are well defined. A simple case of a 200 mm wide vertical wall member, of unit length, heated on both sides by a nominal fire, is considered. The fire follows a time-temperature response corresponding to either the standard fire curve [2] or a defined parametric fire [3]. As noted previously the 'standard fire curve' refers to the heating regime a furnace would typically follow in a fire resistance test. This fire curve describes an ever increasing gas temperature with time. In reality fires have a finite life governed by available fuel. One simple representation of 'real fires' are parametric curves which include a cooling phase.

Using the residual cross-section method in section 4.2.2 of EN 1995-1-2 it is possible to determine when a uniaxially loaded timber member will fail under well-defined fire conditions. Conversely, it is also possible to determine the applied load required for a timber member to fail at a pre-determined time. The following is a simple example of this process for a timber member loaded in compression and exposed to the standard fire curve. The target failure time ($t_f$) is 30 minutes:

Step 1: Determine depth of char after 30 minutes- From EN 1995-1-2 table 3.1:

$$b_0 = 0.65 \text{ mm/min} \quad d_{ch} = 30 \text{ min} \times 0.65 \text{ mm/min} = 19.5 \text{ mm}$$

Step 2: Determine reduced cross-section width:

$$b = 200 - 2\left(d_{ch} + d_0\right) = 200 - 2\ \left(19.5 + 7 \text{ mm}\right) = 147 \text{ mm}$$

Step 3: Determine limiting load:

$$P_{\max} = As_c = bls_c = \left(147 \times 1000\right) \times 21.25\ = 3123750 \text{ N}$$

This principal of determining a limiting load for a fire resistance can also be applied to timber tension members using the grade strength [21] of the given timber element. Similarly, the standard-fire charring rate can be substituted with that of the EN 1995-1-2 Annex A to determine failure times/loads for timber members exposed to natural fires. These limiting loads form the applied load in an FEA simulation and the apparent failure time is noted, thus giving a means of benchmarking empirical and simulated results. This modeling process and concept is discussed in detail below.

4.1.1. Geometry and meshing

As noted previously, the simulated specimen represents a wall of width 200 mm, heated from both sides. Utilising a plane-strain formulation and symmetry, it was possible to simplify the model geometry extensively. The wall is purposely short in height to ensure failure is a result of squashing and not instability/buckling. Simulations were conducted on walls (of unit length) loaded both in uniaxial tension and compression. The density of mesh adopted was governed by the heat transfer analysis conducted prior to the mechanical analysis. As such a graded mesh was adopted which is denser at heated boundaries relative to the centre of the timber specimens. More dense meshes were trialled and were shown to result in nominal differences relative to the mesh adopted.

4.1.2. Thermal material properties

The thermal properties adopted fall into two categories, namely boundary/interface properties and thermo-physical material properties. The boundary properties determine the net heat flow into a structure and describe convective and radiative heat transfer. Convection coefficients of 25 and 35 W/m$^2$ K were adopted for standard and parametric fire exposure. A constant emissivity of 0.7 was assumed throughout. The thermo-physical properties of solid timber are taken from Annex B of EN 1995-1-2 for standard fire exposure (Figure E.4), with modifications for natural fires, as set out in section 3, where appropriate.

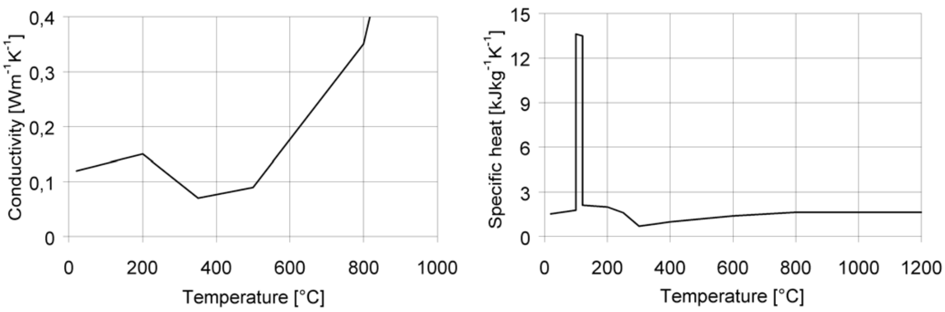



**Figure E.4. Conductivity and specific heat of softwood from EN 1995-1-2 [1]**

### 4.1.3. Mechanical material properties and models

Given the findings of the literature review, it is apparent that timber is ductile in compression and brittle in tension. As a result, plasticity models should be defined for simulations of compression. Similarly, for elements loaded in tension smeared cracking models were implemented, with tension softening for improved numerical stability. Class 16 timber is assumed throughout, giving 80% fractile compressive and tensile strengths of 21.25 N/mm$^2$ and 12.5 N/mm$^2$ respectively. Tension softening is assumed to occur over a maximum crack strain of 0.5%. Crack initiation is assumed to occur when the principal tensile stress exceeds the defined tensile strength. The onset of plasticity is determined using the Von Mises yield criteria [22].

### 4.1.4. Constraints and boundary conditions

The creation of a perfectly uniaxial loaded member heated from one side is challenging. To achieve only pure vertical displacements the concept of ‘tyings’ was adopted in DIANA, whereby degrees of freedom were tied together with corresponded resulting displacements. For the purpose of the simulations presented in this paper all degrees of freedom were tied to corresponding horizontally adjacent symmetry-line nodes. This ensured that edge degrees of freedom displace by the same amount as nodes located on the axis of symmetry. In addition, vertical constraints were introduced at the base of the wall, and the horizontal displacement of all nodes was restrained.

#### 4.1.5. Thermal and mechanical loading

Mechanical loading was derived on a case-by-case basis using the concept shown previously in the three-stage example. Applied loadings and target failure times are summarised in the Table E.2. Initial calibration trials were conducted for standard-fire exposure (Runs 1 to 8 C/T), where the thermal and mechanical aspects of timber behaviour are well defined in BS EN 1995-1-2. Those simulations were conducted to establish the robustness of the proposed geometry and material model arrangement before simulations of timber specimens exposed to parametric fires were conducted. The construction of a parametric fire curve is not included herein, however guidance can be found in EN 1991-1-2 [3] using the parameters outlined in Table E.2. In Table E.2 $q_{td}$ refers to the surface averaged fire load density in $MJ/m^2$.

**Table E.2. Overview of numerical study**

| Run number | Fire | $\Gamma$ | $q_{td}$ | $t_f$ (min) | $P_C$ (kN/m) | $P_T$ (kN/m) |
|---|---|---|---|---|---|---|
| 1 C/T | | | | 15 | 3613 | 2125 |
| 2 C/T | | | | 30 | 3123 | 1837 |
| 3 C/T | | | | 45 | 2709 | 1594 |
| 4 C/T | Standard | N/A | | 60 | 2295 | 1350 |
| 5 C/T | | | | 75 | 1880 | 1106 |
| 6 C/T | | | | 90 | 1466 | 862 |
| 7 C/T | | | | 105 | 1052 | 619 |
| 8 C/T | | | | 120 | 637 | 375 |
| 9 C/T | Parametric | 6.83 | 210 | 2338 | 1375 | 6877 |
| 10 C/T | Parametric | 8.93 | 210 | 2511 | 1477 | 7387 |
| 11 C/T | Parametric | 12.15 | 210 | 2671 | 1571 | 7857 |
| 12 C/T | Parametric | 17.49 | 210 | 2855 | 1680 | 8398 |
| 13 C/T | Parametric | 27.34 | 210 | 3038 | 1787 | 8937 |
| 14 C/T | Parametric | 12.34 | 280 | 2233 | 1314 | 6569 |
| 15 C/T | Parametric | 16.12 | 280 | 2416 | 1421 | 7106 |
| 16 C/T | Parametric | 21.94 | 280 | 2608 | 1534 | 7671 |
| 17 C/T | Parametric | 31.59 | 280 | 2796 | 1645 | 8225 |
| 18 C/T | Parametric | 49.34 | 280 | 3008 | 1769 | 8847 |
| 19 C/T | Parametric | 3.54 | 150 | 2482 | 1460 | 7300 |
| 20 C/T | Parametric | 4.63 | 150 | 2614 | 1538 | 7690 |
| 21 C/T | Parametric | 6.30 | 150 | 2760 | 1624 | 8118 |
| 22 C/T | Parametric | 9.07 | 150 | 2907 | 1710 | 8551 |
| 23 C/T | Parametric | 14.17 | 150 | 3082 | 1813 | 9066 |

5. Results

The simulations outlined in Table E.2 are split into two categories: standard-fire and parametric fire simulations. In each case the assumed failure time is compared with the target failure time according to calculations conducted using section 4.2.2 of BS EN 1995-1-2. Failure in the simulation of the timber members is determined via either a deflection rate tending towards infinity or a lack of convergence for ductile and brittle failure modes respectively. In the latter case the stresses immediately before numerical instability were inspected to confirm the simulation was approaching failure. In addition to a comparison of the failure times of various cases, the concept of the zero-strength layer (adopted in the reduced cross-section method of EN 1995-1-2) was also investigated. In the simulation results presented the apparent zero-strength layer is determined as follows:

Step 1: Determine area required to resist the applied load using ambient temperature strengths, i.e.

$$\frac{P_c}{s_{c,20°c}} = A_{req} \qquad \text{similarly} \qquad \frac{P_t}{s_{t,20°c}} = A_{req}\,.$$

Step 2: Determine depth of char ($d_{ch}$) from simulations by approximating the position of the 300°C isotherm as the position of the char line.

Step 3: Determine residual cross section area ($A_{fail}$) at failure:

$$A_{fail} = (b - (2d_{ch})) \times l \text{, where } l = 1000 \text{ mm}.$$

Step 4: Determine zero-strength layer depth ($d_0$) simply as:

$$d_0 = \frac{1}{2}\left(\frac{A_{reg} - A_{fail}}{l}\right).$$

The use of the above simple formulae allowed the authors to determine the validity of the zero-strength layers published in EN 1995-1-2 for standard fire exposure, whilst also allowing the assessment of their suitability for natural (parametric) fire exposure conditions.

The results of the numerical simulations outlined in Table E.2 for standard-fire exposure (cases 1-8) are shown in Figures E.5a and b. The first of these figures plots computed failure times (from FEA simulations) against target failure times, for which corresponding loads were derived with the reduced cross-section method. The second figure plots the apparent zero-strength layer depth versus failure time for members loaded in compression (C) and tension (T). Additionally, the size of zero-strength layer given in EN 1995-1-2 is also included for comparison.  Similarly, Figures E.6a and b give the same results for parametric fire exposure (cases 9-23).

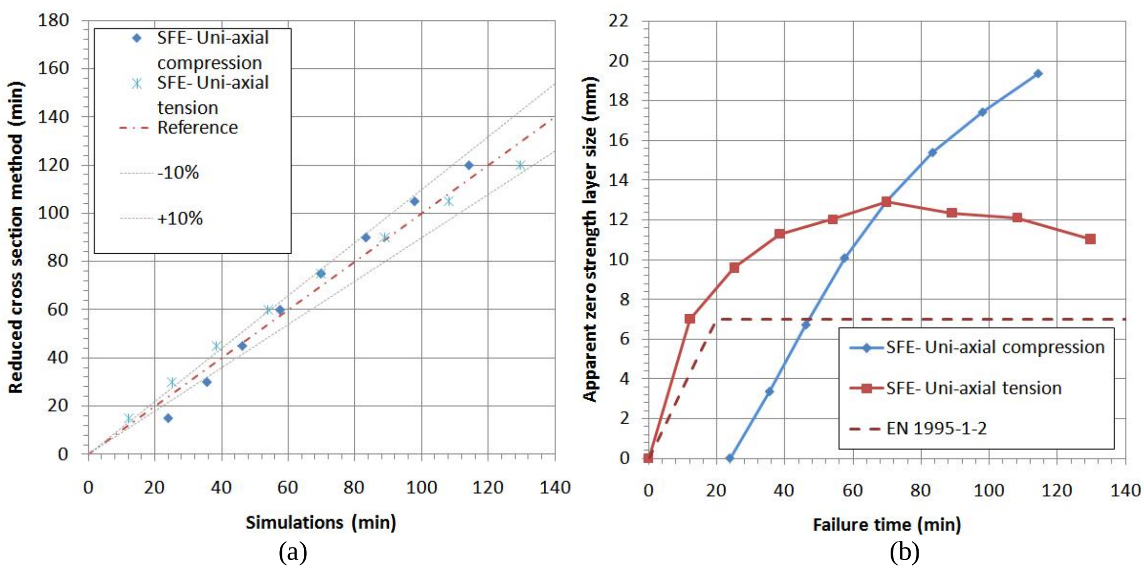
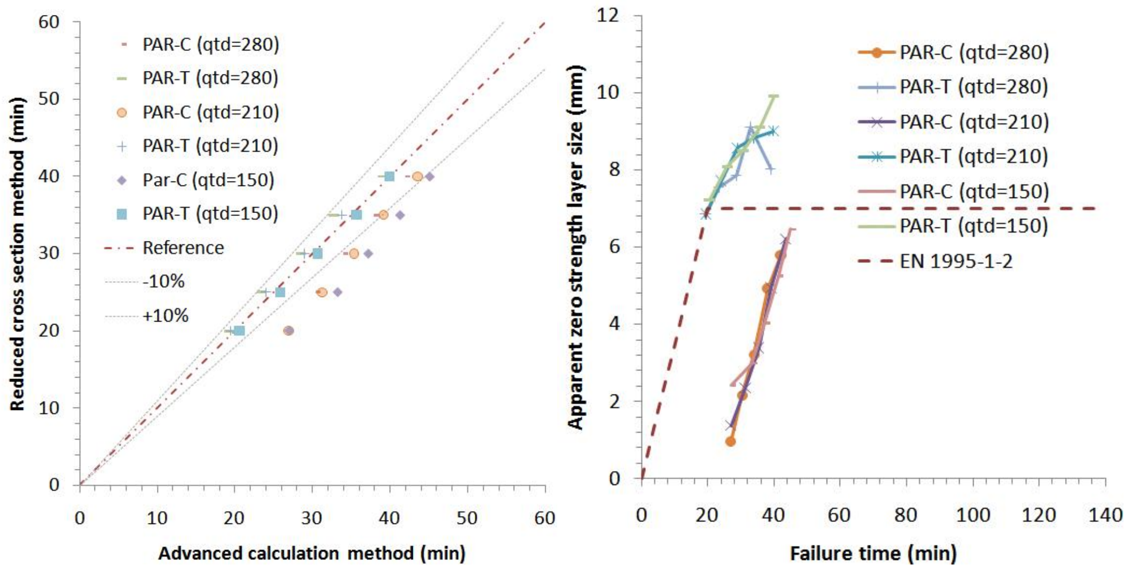



**Figure E.5. Results for standard fire exposure (SFE): (a) failure time; (b) $d_0$ dimension**




(a) (b)

**Figure E.6. Results for parametric fire exposure (a) Failure time (b) $d_0$ dimension**

The results presented in Section 5.1 highlight that both the reduced cross-section method and the advanced calculation properties (included in Annex B of EN 1995-1-2) give consistent predictions of failure time for simple tension and compression members exposed to standard fires. Generally, apart from very low failure times, the variations fall within 10%.

The deviations apparent at low failure times are explained by the significant difference in depth of char when simplified charring rate concepts and advanced heat transfer models are compared. The former case assumes charring starts instantaneously, whilst the latter case is more realistic and simulates a short time delay in the onset of charring.

It is apparent from observing Figure E.5b that significant differences exist between the codified zero-strength layer and that determined by the authors using numerical simulations. For both compression and tension simulations, the zero-strength layer exceeds that proposed in EN 1995-1-2 after approximately 45 and 15 minutes, respectively. Much of EN 1995-1-2 is empirical; however the concept of the zero strength layer was derived numerically based upon simulations of flexural members [23]. Hence, the applicability of the EN 1995-1-2 concept of a 7-mm zero-strength layer for uniaxially loaded members is questionable as noted in other studies [23].

The results in Section 5.2 indicate that the MCM proposed by the authors can be implemented in coupled thermo-mechanical analyses of timber exposed to the heating phase of parametric fires. Comparison of predicted failure times using FEA simulations are benchmarked against target failure times using the reduced cross-section method, indicating very favorable correlation. This is particularly apparent for uniaxial tension members. Much like for standard-fire exposure, the determined depths of a zero-strength layer are inconsistent with those proposed in EN 1995-1-2. As a result it is clear that the equations for the zero-strength layer in EN 1995-1-2 should be modified not only for parametric fires, but also for cases where members are not loaded in flexure. It is, however, apparent that there appears, particularly for parametric fires, to be a linear relationship between the zero-strength layer depth and failure time.

## 6. Conclusion

This paper has presented a demonstration of the implementation of a MCM proposed by the authors in coupled thermo-mechanical analyses. Encouragingly, although the cases studied are relatively simple, the failure times noted in simulations are in agreement with those determined using the empirical equations in EN 1995-1-2. Interestingly, it has been determined that, for uniaxially loaded members, the zero-strength layer presented in EN 1995-1-2 may not be sufficient for both standard and parametric fire exposure and is derived from simulations of flexural members [23] using the advanced calculation properties outlined in Annex B.

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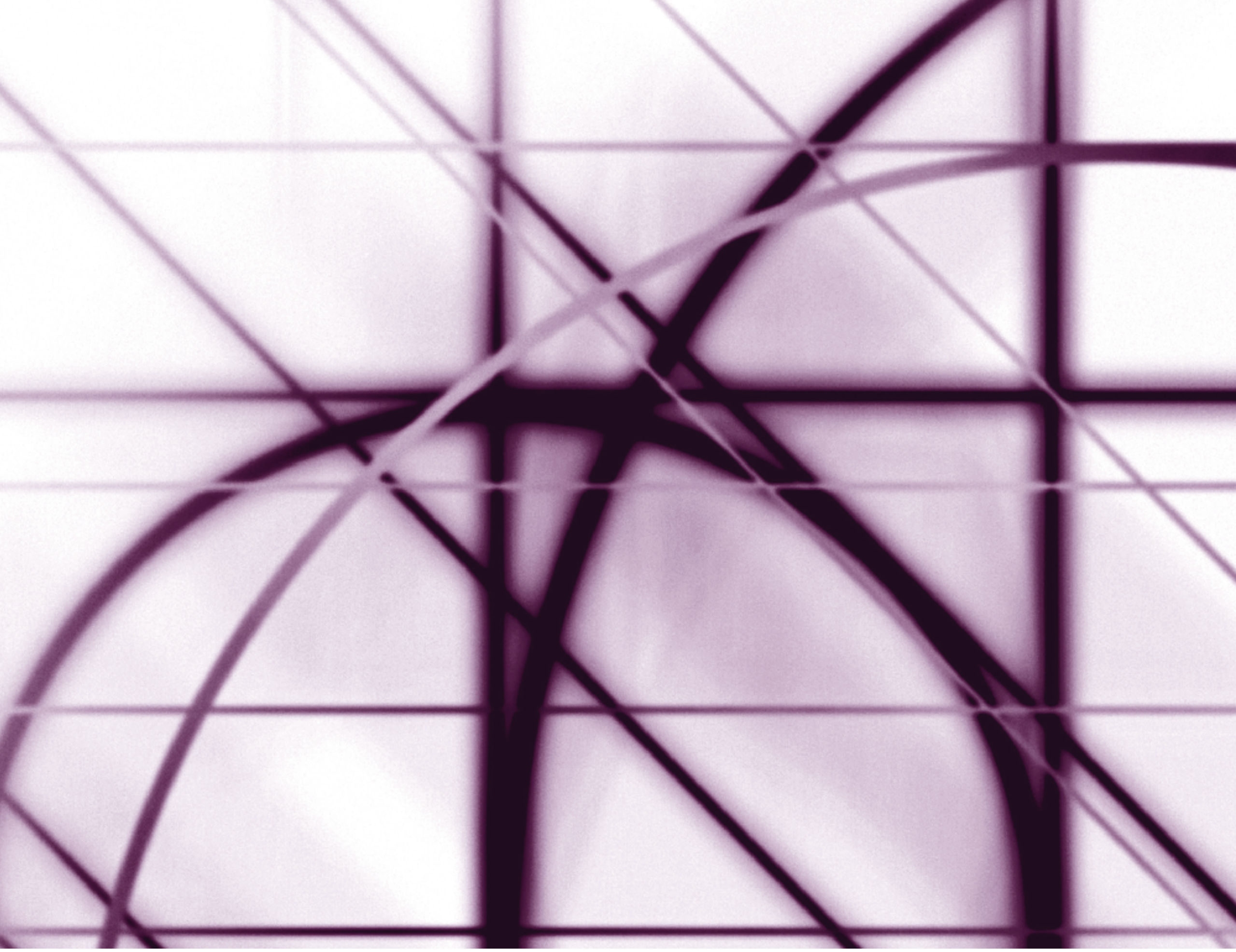

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