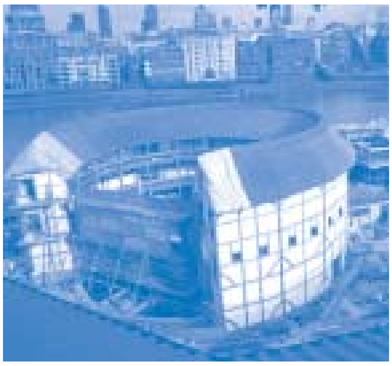
IStructE

Introduction to the fire safety engineering of structures

September 2003



The Institution of Structural Engineers

September 2003

Introduction to the fire safety engineering of structures

Constitution of Task Group

M Green BE CEng MIStructE MICE (Buro Happold) *Chairman*
C G Bailey BEng PhD CEng MIStructE (Manchester Centre for Civil & Construction Engineering)
A Burd BSc(Hons) MIFireE MBEng FRICS (Office of the Deputy Prime Minister)
P Bussey* BA Dip.Arch MIFireE MaPS RIBA (RHWL)
G Faller MSc CEng MIStructE MIFireE (Ove Arup & Partners Ltd.)
K Fisher** BSc MSc Tech PhD (Consultant)
G Harding DIC CEng FIStructE MICE (Office of the Deputy Prime Minister)
B R Kirby BSc PhD FIFireE CEng (Corus Fire Engineering)
C J Mettem MTech, CEng, MIMechE, FIWSc (TRADA Technology Ltd.)
G Newman BSc(Eng) CEng MIStructE MIFireE (The Steel Construction Institute)
R Plank BSc(Eng) PhD CEng FIStructE MICE (Sheffield University)
J Purkiss BSc(Eng) PhD CEng MIStructE MICE MIFireE (Aston University)

Corresponding members
D Charters*** BSc(Eng) PhD CEng MIMechE FIFireE (Ove Arup & Partners Ltd.)
A Ferguson MSc BArch MIFireE ARIAS (Ove Arup & Partners Ltd.)

Technical Editor
S Lamont BEng PhD (Ove Arup & Partners)

Secretary to the Task Group
B Chan BSc(Hons) AMIMechE (Institution of Structural Engineers)

* representing the Royal Institute of British Architects
** representing the Brick Development Association
*** representing the Institution of Fire Engineers

Published by The Institution of Structural Engineers,
11 Upper Belgrave Street, London SW1X 8BH, United Kingdom

ISBN 0 901297 29 1

The Institution of Structural Engineers and the members who served on the Task Group that produced this report have endeavoured to ensure the accuracy of its contents. However, the guidance and recommendations given should always be reviewed by those using the report in the light of the facts of their particular case and any specialist advice. No liability for negligence or otherwise in relation to this report and its contents is accepted by the Institution, the members of the Task Group, its servants or agents.

Contents

Contents

Glossary

Class 0: is the highest product performance classification for wall and ceiling linings. It is not identified in any British Standard Test. Class 0 is met if any material or surface of a composite product are composed throughout of materials of limited combustibility; or a Class 1 material has a fire propagation index (I) of not more than 12 and sub-index (I_1) of not more than 6. The rating Class 0 limits fire spread over and energy released from linings.

Compartmentation: separation of an area, a whole floor or building by enclosure within fire resisting construction.

Conduction: heat transfer through solids.

Convection: heat transfer through gases (and liquids).

Emissivity: a measure of the efficiency of a surface as a radiator. A black body has an emissivity of 1.

Fire compartment: a space within a building enclosed by separating members (e.g. wall, floor) tested to the required fire resistance. The space may extend over one or more storeys.

Fire load: the energy released by combustion of materials in a space.

Flashover: relatively rapid transition between the fire which is essentially localised around the items first ignited and the general conflagration when all surfaces within the compartment are burning.

Fully developed fire: fire stage after flashover. All combustibles within the compartment are burning.

Heat transfer: movement of heat energy from areas of high temperature to areas of lower temperature by three means: conduction, convection and radiation.

Nominal temperature-time curve: a well-defined fire exposure curve used for verification of fire resistance e.g. the standard fire curve.

Opening factor: is the ratio of the ventilation factor to the total surface area (A_t) of the enclosure $\dfrac{A_w\sqrt{H}}{A_t}$ ($m^{1/2}$)

Radiation: heat transfer by electromagnetic waves.

Spalling: is the loss of surface material from a concrete element and is dependent on aggregate, moisture content, stress level and temperature.

Standard fire curve: temperature-time relationship of the fire gases in a standard furnace test. The heating curve is achieved by programming the test furnace through control of the rate of fuel supply.

Temperature-time curve: gas temperature against time during a fire or in a furnace.

Thermal diffusivity: a measure of the rate of heat transport from the exposed surface to the inside of the material and of the temperature rise at a depth in the material.

Time equivalence: is defined as the exposure time in the fire resistance test which gives the same heating effect on a structure as a given compartment fire.

Ventilation factor: $A_w\sqrt{H}$, where A_w = area of openings (m^2), H = height of the openings (m).

Work size: defined by BS 3921 as the size of a brick (or unit) specified for its manufacture, to which its actual size should conform within specified permissible deviations.

Foreword

The compelling need to control the effects of fire has strongly influenced buildings since medieval times. The growing complexity from simple masonry chimneys to modern structures and active fire systems is symptomatic of the increasing sophistication of the built environment.

However complete control is not a realistic possibility even though there are advances in management, security, design and fire fighting. Recent disasters including the Kings Cross Fire in London and the World Trade Center in New York illustrate that we will never fully control fire but there is still progress to be made.

The increasing sophistication requires a well co-ordinated approach between the relevant disciplines to achieve the best results. Accordingly this report recognises the influence of the structural engineer, the architect, the fire safety engineer and many others in the design and the procurement process. It provides an introduction and a contribution to support the developments that are progressing world-wide to improve current practice from a structural engineering perspective. Some insight into the creative thinking by the structural engineer and the architect to help identify the technical and aesthetic requirements is also presented to enable the designers to better serve the needs of society and our clients.

However the essential breadth would not have been possible without the very broad support from government, industry and academia. Also considerable benefit was delivered when a draft report was made available on the Institution web site so that many others had a chance to contribute.

This report although detailed in some respects is still very much of an introduction. It is also clear that there is much research to do and knowledge to disseminate in the future. Much of this progress will come from the evaluation of the risks associated with real structures and natural fires that will lead to better risk assessments and better value.

The chairman would like to thank the Task Group and the correspondents who have contributed to the effort required to produce this challenging and topical report.

Mick Green
Task Group Chairman

1 Introduction

1.1 Scope

This guidance has been prepared to provide the engineer, the architect, the regulatory authorities and other construction industry professionals with the inspiration to develop safer and better value solutions for the performance of building structures during fire. There is a considerable opportunity for the engineer and the architect to work together to develop improved designs based on new and developing technology within a sensible regulatory framework. The guidance primarily addresses building structures and does not consider special structures such as tunnels.

The design of building structures during fire is developing at a significant pace in line with fire safety engineering as a whole. It is certainly starting to have a growing impact on the way structures are designed, procured and specified in many countries throughout the world. New procedures, advanced analytical methods and improved risk assessment techniques are now available to the experienced engineer to support performance-based design for the fire load case. However this knowledge tends to be in the hands of a few specialists and consequently the Institution of Structural Engineers has identified the need for guidance at a level that will be of value to a wide range of construction professionals. This document should be of benefit to the architect looking for better solutions, for the controlling authorities wishing to ask the right questions and for engineers seeking to develop new avenues and skills as technology develops in fire safety engineering. It should also enable contractors, manufacturers and suppliers to appreciate the broader approach being adopted and to adapt their products and future development. The document tends to reference UK and EU practice but much of the guidance is generic and applicable internationally.

In order to achieve the above aspirations there is a need for a wide range of skills. The structural engineer or the architect will be able to adopt some of these skills fairly easily. However other skills, which may involve the complexities of fire dynamics or the finite element analysis of hot structures, will require additional expertise. The guidance in this document will provide a comprehensive introduction to the design of the primary structure during fire for all the principal construction materials. Chapter 2 describes the concept of fire resistance and introduces the reader to fire. In chapter 3 fire resistance design is summarised whilst chapters 4-7 give specific guidance and information for the four main construction materials namely concrete, steel, timber and masonry respectively. Inevitably the level of research and knowledge of different construction materials is at varying stages; therefore the content of the corresponding chapters (chapters 4-7) is variable as is the number of associated references.

So that all potential users can benefit from the advances in fire safety engineering, guidance beyond this introductory document is outlined in the various chapters under the heading ‘Further engineering methods’.

In the future with the development of fire safety engineering and the spread of this knowledge there is an opportunity to reduce the fragmentation that is currently present in the procurement of some structures. This will come from the close co-operation of designers, suppliers, regulators and contractors.

The responsibility for fire safety design of a building needs to be clearly identified by the design team leader and the client at the outset.

1.2 Historical background

The development of prescriptive regulation has been established as a result of a series of prominent fire disasters and the recognition that there is a need to control regular losses.

The majority of life loss during fire is in dwellings, although there are occasionally major losses of life in public buildings including the Summerland fire on the Isle of Man, the Manchester Woolworth’s fire, the King's Cross fire and most recently the terrorist attacks on the World Trade Center, New York. The acceptance of life loss in public buildings is certainly much less than in dwellings, which is one of the factors that influence the content of regulations. This is primarily because people feel they are in control in their own homes, whereas in a public building others are responsible for their safety. The potential losses are also much greater in a public building than in a single dwelling. Public tolerance to loss of life on a larger scale is significantly less than it is to individual deaths in dwellings.

In medieval times attempts were made to control fire at source by the introduction of non-combustible chimneys. As long ago as 1189 the Assize of Henry

FitzAlwyn, prompted by the disastrous fires which regularly destroyed much of London, required stone walls to be built on the boundary between buildings to prevent the fires spreading. This legislation for non-combustible party walls has governed construction ever since. It was reiterated in the Act following the great fire, and was extended in the 19th century to include non-combustible, or 'fireproof', floors in certain parts of buildings such as corridors and staircases to facilitate escape. However much of the early regulation was influenced by property protection. The Fire of London was a good example of this, where life loss was very low but the economic losses were significant. In the 19th and early 20th century new materials and forms of construction were introduced e.g. cast iron, wrought iron, reinforced concrete, steel and many other proprietary systems. An understanding of their performance in fire was gained through testing.

Testing has always been a part of improving the understanding of the performance of buildings. Much of it took place in real buildings. There were the tests on the Hartley and Stanhope systems[1.1] in the late 18th century and the British Fire Prevention Committee tests for building components in the closing years of the 19th century. It was not until the early 20th century that the concept of the standard fire test was introduced.

One of the most significant developments in the 20th century was the Fire Grading of buildings, where the first real attempts to assess relative risks, the requirements for fire fighting, means of escape and limitations on compartment sizes were made[1.2].

Much of the early regulation on the control of buildings related to non-combustibility and fire separation. It was only as disciplines such as architecture and structural engineering developed into definable technical disciplines that fire was no longer the dominant factor in the design of buildings. Design for fire, other than by testing and prescription, was not supported sufficiently by science until relatively recent times. It is probably true to say that design for fire safety is where structural engineering was a 100 years ago except technology and computers are enabling it to progress much more quickly. Fire safety engineering is now rapidly closing the gap that has developed throughout the majority of the 20th century. This will enable the gradual transition from a prescriptive based approach to an engineered approach where goals can be set and achieved in the style of the more established engineering disciplines.

1.3 Background to fire safety

Fire resistance is one component of the set of fire protection measures that are required by a fire strategy to give adequately safe buildings. Most fire safety engineering is not the responsibility of the structural engineer but for clarity the scope of fire safety engineering and its relation to the specific problem of structural fire resistance is summarised.

The prime objective of the fire safety engineer is to prevent, delay or reduce the effects of flashover (flashover is defined as the rapid transition of a fire from its growth phase to its fully developed stage, see section 2.5) providing adequate time for the occupants of the building to escape. Consequently a considerable amount of fire safety engineering is concerned with the growth phase of a fire. This is achieved by designing active fire protection systems such as early detection, sprinkler systems and smoke control which may require the calculation of activation times of sprinklers and detectors. The rate of fire growth leading to flashover can also be predicted based on the amount, type and arrangement of fuel, the compartmentation geometry and the available ventilation. Other calculations may compare evacuation times with the growing depth and temperature of the smoke layer in a room.

Active fire suppression systems have many benefits some of which are related to the fire resistance of the structure. In certain occupancies the inclusion of sprinklers to a life safety standard will reduce the level of fire resistance required by the appropriate building regulations. It should be noted that sprinkler systems are designed to support activation of one or two sprinkler heads in the event of a fire. The sprinkler closest to the fire will be activated by the local heat source and all other sprinklers will remain inactive unless the fire grows to a size where a second sprinkler is affected. It is a common misconception that all sprinklers will operate on the floor of fire origin and flood the floor. One or two sprinkler heads have been shown to be effective in controlling and in some cases extinguishing fire in its early stages. The use of sprinklers also:

- allows a degree of certainty over the design fire size
- can perform the dual function of detection and suppression
- increases allowable compartment size
- increases the time for untenable conditions and thus increases available evacuation time
- reduces the fire size the fire service would be expected to tackle.

Properly designed smoke control vents heat and smoke, thereby:

- delaying the advance of untenable conditions, increasing the time for evacuation
- limiting the fire size and reducing the risk of flashover.

Fire resistance becomes important as a fire progresses to its fully developed stage. Where active systems are present, fire resistance is a second line of defence and may only become important if active systems fail. Fire resistance is described as a passive measure. The purpose of providing fire resistance depends on the circumstances and is influenced by one or more of the following:

- Life safety for means of escape and fire fighting. This tends to be more important in large and complex buildings where evacuation and fire fighting operations take longer.
- To protect business continuity, our heritage or special occupancies like hospitals, where evacuation and protection of property need special attention.
- To protect adjacent buildings from fire. This is achieved by compartmentation, which limits the amount of building on fire at a particular point in time. This in turn reduces the resultant radiation on adjacent buildings. This aspect is covered by normal prescriptive guidance where compartmentation and the combination of fire resisting walls and openings in the elevations are controlled.
- To reduce the chance of collateral damage resulting from the collapse of a structure. The normal prescriptive guidance covers this for the majority of buildings. However in major/complex and high rise buildings there may be a need to adopt a risk based approach. For example fire, explosions and other extreme events will need to be established as part of the scenario planning process when this approach is adopted and special events like terrorist attacks are considered[1.3].

There are two primary ways of achieving an adequate standard of fire safety in a building, including the fire performance of structures. One is the simple application of the building codes and standards, which requires limited engineering as the majority of solutions are prescribed. There is little flexibility in the approach. Alternatively, a fire safety engineering approach gives greater design flexibility to achieve a particular performance but requires greater skills involving analysis, risk assessment and engineering judgement. There is often an opportunity to improve value and/or performance by selecting the most appropriate combination of fire protection measures with each building requiring its own consideration and its own solutions. Engineered solutions can also be used to demonstrate an equivalent level of fire safety where there is a variation from prescribed guidance. One of the important objectives of this guidance is to enable the architect and the engineer to choose the most appropriate mix of engineering and prescription. It is also intended to provide assistance to the approving authorities to improve the efficiency of submission and approval process.

There are three ways of testing the ability of a structure to carry load during a fire:

- Time domain
- Strength domain
- Temperature domain

The time domain is commonly adopted. Time to failure in the fire resistance furnace (or equivalent time, see Chapter 2) must be greater than the fire duration as set down by the relevant code or standard. The other options of strength and temperature are more often associated with an engineering approach. In the strength domain the load capacity of an element of structure must exceed the load at the fire limit state. In the temperature domain the temperature of the structural element must not exceed the critical temperature of the material.

Generally higher stresses and loads are acceptable in the fire load case when compared with the normal ‘cold’ load case design and will be controlled by the partial load factors referenced in the relevant documents.

Some of the broader fire safety considerations that have an influence on the design of structures during fire, when adopting a more sophisticated approach are summarised below:

- Use of natural fires for design (see Chapter 2) can have varied impact e.g. in certain circumstances increased ventilation can cause reduced fire temperatures whereas in other cases temperatures can increase more quickly. For example the available ventilation through the glass façade at the Greater London Assembly Building, London resulted in a reduction of fire resistance rating (see Figure 1.1).

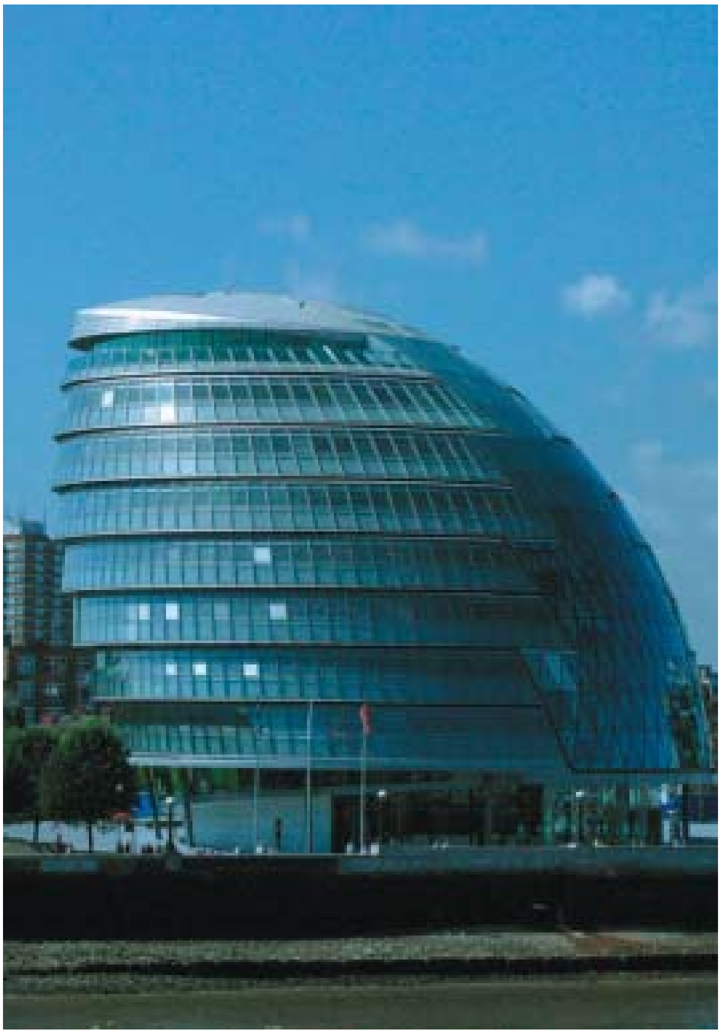


Fig 1.1 Greater London Assembly Building, London – This project used fire engineering principles to assess the fire resistance requirements for the structure as a whole, instead of relying entirely on the tabulated fire resistance values given in Approved Document B[1.4] © ARUP

- Beams and columns at the edge of a building are often cooler than internal beams and columns in the same fire.
- In very large buildings (high rise or with extended horizontal travel distances) with long times for total evacuation, additional consideration should be given to the performance of the structures where remote or progressive collapse could have an impact on safety. Fire fighting will also be an important consideration.
- Structural performance is not a direct measure of protection of life or protection of property and therefore broader considerations that involve means of escape, use of sprinklers and the quality of the operational management have an influence.

1.4 Specific structural engineering issues

There are also a number of other considerations that the designer needs to be aware of when looking at the structural performance in more detail although these considerations can also be of value to the engineer during the concept and scheme design process.

- Different structural materials react very differently to heat. For example the performance of unprotected steel is dependent on the actual temperature reached with exposure time having relatively little effect on material characteristics. Steel loses strength and stiffness with increasing temperature.
- The temperature achieved by reinforced concrete structures during a fire is dependent on the duration of the applied heat. Concrete heats much more slowly than steel because of its thermal properties but like steel also loses strength and stiffness with heating.
- Timber sections burn resulting in an insulating layer of char forming on the exposed timber.
- Selection of the most appropriate structural material to suit the engineering and the architectural requirements for the normal and the fire load case together usually leads to an improved and better value design.
- Connections behave differently at high temperatures than at ambient, with additional imposed moments, rotations and axial forces. The behaviour of structures in fire can impose different conditions on connections e.g. simple connections may become moment resisting in fire.
- Structural members experience large deflections in a fire as a direct result of thermal expansion effects and loading on a weakened structure.
- Real structures rarely act in the same way as the simple elements that are commonly tested in a furnace. This is because structures often have inbuilt secondary load paths, which can be mobilised during fire when structural deflections become larger. This inbuilt redundancy can provide an enhanced performance during fire in some structures. However care should be taken as in some cases redundancy can give unexpected deflections, which can lead to other modes of failure.
- It is important to use appropriate acceptance criteria, when adopting a more sophisticated engineering approach, which can involve larger deflections than those that would be acceptable for serviceability requirements at ambient. Internal thermal forces can also be important.

1.5 Structural fire design and architectural solutions

The design of structures to perform in fire can have an important influence on architectural solutions in a variety of ways. It may involve more advanced engineering where the architectural solutions preclude the use of prescriptive rules or where good value is sought for highly repetitive forms of construction (see Figure 1.2). Some examples of where the fire performance of structures and the architectural solutions are closely related are given below:

- The need for sustainable environmental solutions often leads to the need for the exposed structure to provide the thermal mass that goes hand in hand with this type of design (see Figure 1.3). This leads to the need to consider fire safety solutions in greater depth or possibly a fire safety engineered solution.
- The increasing use of exposed structure as a fundamental part of the aesthetic is also important. In these situations the need for an integrated approach and an enhanced knowledge of fire safety are essential. A typical example of this is external unprotected steel (see Figure 3.1). Bare structural steel can be water-cooled to obtain the recommended fire resistance although this is an expensive solution. Examples are given in Figures 1.4 and 1.5.
- The desire to minimise the depth of the combined structure/services zone to reduce overall building heights is one of the challenges that the architect and the engineer continually face as spans increase and columns reduce in number. The potential solutions are numerous (see Figure 1.3). Fire performance of the structure plays a significant role in this process.

1.6 Codes, standards and relevant documents

At the time of writing this guidance there is considerable change and development taking place in fire engineering internationally. Methods are improving, research is turning into engineering practice and performance based standards are being devised. A careful approach is needed to balance what is developmental, what is viable for the fire safety engineer and what can be safely adopted. This guidance is part of that trend.

In the UK control of fire safety is based on a whole series of layered codes of practice and standards, which emanate from the functional

Fig 1.2 Glaxo Smith Kline building London – The GSK building involved repetitive large scale construction and fire safety engineering was used to design the edge beams to address constructability and improve safety. Offsite intumescent was more practical as a construction solution but the prescriptive fire resistance rating could not be economically achieved. An engineering approach looking at the natural fire effects on edge beams and considering the 3D behaviour of the structure enabled the use of offsite intumescent to achieve the required standard. © Hillier RHWL (view under construction), © Buro Happold (inset)

Fig 1.3 Wessex Water Headquarters – This project is an example of the integration of concrete and steel construction to support very low energy design solutions and the integration of services. Exposed structural steel was an integrated part of this solution. © Buro Happold/Mandy Reynolds

Fig 1.4 Bush Lane House – Water cooling of the external structure allowed the use of exposed steel without passive fire protection.

requirements of the Building Regulations. There are currently the high level documents like *Approved Document B (Fire Safety)*[1.4], BS 7974[1.5], BS 5588[1.6] etc. Below these high level documents there is a whole range of other standards that deal with structure, building services and architectural systems. At the lowest level there are whole sets of material performance standards that enable the delivery of the performance requirements defined in the above documents.

It is important to recognise that structural fire safety measures can make a contribution to the protection of property and, as a consequence, insurance premium reductions may apply if advice offered by their surveyors is followed. Guidance on standards that may be applied by insurers is available in a document published by the Loss Prevention Council (LPC), *Design Guide for the fire protection of buildings*[1.7].

Fig 1.5 HACTL Terminal (Hong Kong Cargo Terminals Ltd) – On this project the automatic sprinkler system was incorporated into the tubular roof truss members in such a way that the sprinkler water served a dual purpose; that of controlling the fire size by normal sprinkler action and to remove heat from the structure. The combined effect was used to demonstrate that no passive fire resistance protection was required.

Eurocodes[1.8] are gradually being produced for all aspects of design and will eventually replace British Standards in the UK. Eurocodes have considerable design content which has been derived directly from research. Reference to Eurocodes in this document have been based on draft for comment versions. The reader should refer to the most recent published Eurocode.

The research solutions are continuously feeding into design codes and guidance. Therefore practitioners should frequently check what the latest information is in their country.

References

1.1 *Resolutions of the Associated Architects with the Report of a Committee To consider the Causes of the Frequent Fires, and the best means of preventing the like in future*. London, 26th July, 1793.

1.2 *Fire Grading of Buildings – Part 1. General Principles and Structural Precautions*. Post-War Building Studies No.20. Ministry of Works, HMSO, London, 1946.

1.3 Standing Committee on Structural Safety. *Structural Safety 2000-2001*. 13th Report of SCOSS. The Institution of Structural Engineers, London 2001.

1.4 *Building Regulations 1991. Approved Document B: fire safety*. The Stationery Office, London, 2000.

1.5 *BS 7974: 2001. Application of fire safety engineering principles to the design of buildings*. British Standards Institution, London, 2001.

1.6 *BS 5588: Part 1-11. Fire precautions in the design, construction and use of buildings*. British Standards Institution, London, various dates.

1.7 Loss Prevention Council *Design Guide for the fire protection of buildings 2000: a code of practice for the protection of business*. Revised edn. Fire Protection Association, London, 1999.

1.8 *DD ENV 1991-2-2: 1996. Eurocode 1 Basis of design and action on structures. Part 2.2 Actions on structures exposed to fire*. British Standards Institution, London, 1996.

Summary

- Fire resistance design is traditionally based on fire resistance testing using a standard time temperature curve. The objective is to ensure that the performance in the test is greater than the prescribed requirement.
- The prescribed fire resistance requirements are related to building height and occupancy. Building height is generally used because fire fighters have to enter tall buildings and may have to remain there for a considerable period of time.
- Conventionally fire exposure has been based on the standard temperature-time curve followed by the furnace test in ISO 834[2.1], BS 476[2.2] or ASTM E119[2.3]. The standard furnace test has shortcomings but it is still the only universally accepted method of demonstrating fire resistance.
- Natural fires are very different in peak temperature and duration from the standard fire curve.
- The development of natural fires is dependent on fuel load, ventilation, compartment shape and the thermal properties of the boundary wall materials. Occupancy influences the type of fire load present in a particular building.

2.1 Introduction

The purpose of this section is to describe current prescriptive design in relation to the standard fire resistance test. Alternative approaches will be discussed and the relevant field of application for such alternative methods identified.

2.2 Prescriptive fire resistance

For most practitioners the term fire resistance is a period of time for which an element of construction (beam, column, floor, wall etc.) will survive in a standard fire test carried out in an approved furnace under specified conditions of temperature, imposed load and restraint.

The fire resistance provisions for buildings are normally specified in the various building regulations around the world. In the UK[2.4-2.6], for example, all buildings must meet certain functional requirements covering means of escape, internal fire spread, external fire spread, and access and facilities for the fire service.

Traditionally the prescribed fire resistance recommendations are set out in tables (see Table 2.1 below taken from the *Approved Document 'B', 2000 Edition*[2.4]) with the fire resistance being a function of building use and height above ground level (providing some measure of the number of storeys). Building height is generally used because fire fighters have to enter tall buildings and may have to remain there for a considerable period of time. Also the time for escape is longer. Occupancy relates to the type of fire load present in a particular building. If the building is sprinklered a reduction in fire resistance rating is possible in some occupancies.

Table 2.1 Minimum periods of fire resistance

Purpose Group		Minimum periods of fire resistance (minutes)					
		Depth of lowest basement		Height of top floor above ground			
		more than 10m	not more than 10m	not more than 5m	not more than 18m	not more than 30m	more than 30m
Residential flats and maisonettes	Unsprinklered	90	60	30	60	90	120
Office	Unsprinklered	90	60	30	60	90	Not permitted
	Sprinklered	60	60	30	30	60	120
Shops and Commercial	Unsprinklered	90	60	60	60	90	Not permitted
	Sprinklered	60	60	30	60	60	120
Assembly and Recreation	Unsprinklered	90	60	60	60	90	Not permitted
	Sprinklered	60	60	30	60	60	120
Industrial	Unsprinklered	120	90	60	90	120	Not permitted
	Sprinklered	90	60	30	60	90	120

Internationally there can be distinct differences in fire resistance ratings. Table 2.2 lists the fire resistance ratings applicable to multi-storey office blocks in various countries. In Hong Kong the fire resistance ratings are much lower because the provisions for means of escape are much more onerous than in the UK or the USA. In Hong Kong allowable travel distances are shorter and evacuation is simultaneous i.e. the whole building is evacuated together. In the UK the concept of phased evacuation is generally adopted in taller buildings where occupants on the fire floor and the floor above are evacuated first. A decision is then taken about whether to evacuate other floors. As the escape stairs are sized for phased evacuation they are generally fewer or narrower.

Table 2.2 Comparison of typical fire resistance ratings in various countries

Country	Fire resistance rating for a multi-storey office block (minutes)
Australia	120
UK	120
Hong Kong	60
Sweden	90
Singapore	90
US	120

It is important to note that building regulations are only intended to ensure reasonable standards of health and safety for persons in or about the building, including fire-fighting personnel. They are not designed to prevent structural damage and they are not designed to minimize financial losses arising from a fire although clearly some of the provisions do provide this protection. This has important implications for the fire engineering design of buildings where the requirements of the regulations may not be sufficient to meet the needs of the client.

The most important requirement that deals with the effects of a fire on the structural elements is addressed in the Building Regulations[2.4] as follows:

> *'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 *Approved Document B*[2.4] provides detailed guidance on ways to demonstrate compliance with the functional requirements. In general the most common route to demonstrating compliance has been to follow the guidance in the document (see Table 2.1). However, it is important to emphasise that it is the requirement which is in effect mandatory and not the guidance on how it can be achieved. This allows for alternative approaches to meeting the requirements and these can be developed in collaboration with approving authorities.

The fire resistance provisions contained in the guidance to the approved document relate directly to elements of structure that have been tested in the standard furnace test.

2.3 The standard fire resistance test

The idea of a standardised approach to fire testing dates back to the International Fire Prevention Congress held in London in 1903. In 1917 ASTM-C19 (later altered to E119[2.3]) was issued. This document included a specification for a standard heating curve. The first edition of BS 476 on fire resistance testing was published in 1932. Subsequent revisions have attempted to harmonise both the heating curve on an international basis through the adoption of the international standard, ISO 834[2.1] and the control of furnaces within the European Community.

The standard fire curve has been adopted for a number of reasons:

- to provide evidence to the regulatory bodies of compliance
- to assist in product development
- to provide a common basis for research into the effect of variables other than temperature.

As such it has proved to be remarkably successful over a long period of time. It has the advantage of familiarity for both designers and regulators and there is a large body of experimental data available. It is simple to use and clearly defined and allows for a direct comparison between the performance of products tested under nominally identical conditions.

In the UK the technique used to establish fire resistance is to expose an element of construction to a fire exposure corresponding to the formula below:

$$T - T_0 = 345 \log_{10}(8t+1)$$

where t = time from the start of test (minutes)
T = furnace temperature at time t (°C)
T_0 = initial furnace temperature (°C)

The standard curve in ASTM E119[2.3] is prescribed by a series of points rather than an equation but is almost identical to the British standard curve.

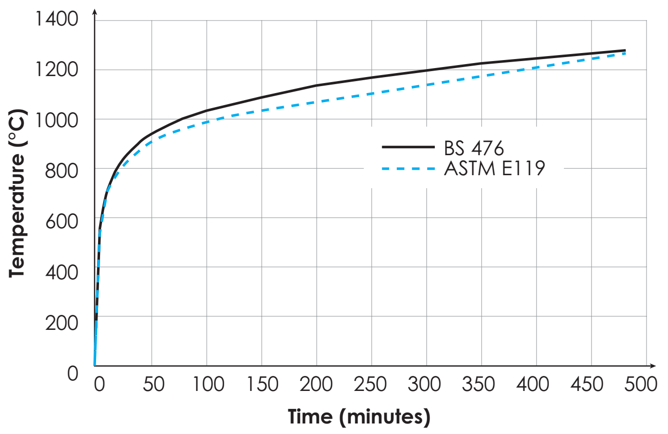


Fig 2.1 Standard temperature time curves

The relationships for the BS 476 and the ASTM E119 standard fire curves are shown in Figure 2.1 for comparison.

The results of the standard test are described in terms of a period of resistance in minutes against each of the appropriate performance criteria (stability, insulation and integrity) described in Chapter 3. In Eurocode terminology such a beam would be classed as R60.

The normal limiting conditions for the stability performance criterion in BS 476[2.7] are given in Table 2.3.

Table 2.3 Failure criteria for load bearing capacity in terms of deflection, δ, in the standard furnace test BS 476: Part 21[2.7].

Structural element	Failure criteria in accordance with BS 476: Part 21
Beam	$\delta \geq$ Span/20 or rate of $\delta > \dfrac{span^2}{(9000 \times d)}$ when $\delta >$ span/30
Column	Rapid rate of vertical deflection or Lateral $\delta >$ 120mm
Note:	*d*=depth of beam section

2.4 Limitations of the standard furnace test

It is often incorrectly assumed that there is a one to one relationship between survival of single elements in a standard fire resistance test and survival of actual buildings in a fire. This is certainly not the case, since a real fire is not likely to follow the time-temperature profile used in a standard fire test and the building will not behave as a collection of individual elements. In reality the element of construction may perform satisfactorily for a longer or shorter period depending on the characteristics of the particular fire and structural configuration. The prescriptive nature of specifying fire resistance for elements of structure has hindered the development of a more rational approach to the design of buildings for fire.

The temperatures in a standard furnace are relatively uniform when compared with those in a real fire compartment. In a real fire temperature variations may develop in structural members which are not present during a furnace test. Lack of furnace harmonization is another drawback to the standard fire test. Although the relevant test codes specify the same control temperature the heat flux experienced by the test specimen is dependent on the form of construction of the furnace, the location of the burners relative to the specimen and the type of fuel used. Additional problems exist since private companies commission most tests and the results are therefore commercially sensitive and are not, in general, available to be scrutinized by researchers. Furnace testing is expensive and the staged grading system in periods of thirty minutes fire resistance does not allow for an optimization of the results from the test.

The physical limitations of standard furnaces mean that it is not possible to simulate complicated three-dimensional structural behaviour. The influence, beneficial or detrimental, of restraint provided by the surrounding structure is thereby ignored. The effect of restraint in standard tests has been discussed extensively in the USA where restrained tests are frequently conducted[2.8]. BS 476[2.2] recommends that restraint conditions should represent those met in practice. This is difficult to achieve in test conditions as restraint is difficult to measure and is likely to change throughout the test. Very often elements are tested unrestrained.

The nature of the test means that only idealised end conditions can be used and only idealised load levels and distributions adopted. During a fire some degree of load shedding will take place from the areas affected by fire to the unheated parts of the building. In the standard test no allowance can be made for alternative load carrying mechanisms or for alternative modes of failure.

The choice of arbitrary failure criteria adopted so as to restrict damage to the test furnace is a further drawback in terms of predicting behaviour within a real building.

Although the shortcomings of the fire resistance test are significant, standard fire resistance tests are the only universally recognised method of determining the fire resistance of elements of construction.

2.5 Characteristics of natural fires in a building compartment

The standard temperature-time curve bears little resemblance to a real fire temperature-time history. It has no decay phase and as such does not represent any real fire. It was designed to typify temperatures experienced during the post-flashover phase of most fires. Figure 2.2[2.8] illustrates the temperature-time histories of 'real' fires, of varying fire load and ventilation, together with the standard curve. This shows that the standard curve generally is a poor representation of real fires in the post-flashover phase.

The standard fire takes no account of the different thermal exposures which result from different compartment geometries, ventilation conditions, fire loads and compartment boundary materials[2.9]. The behaviour of a compartment fire is strongly dependent on available ventilation. If there is insufficient air in the room for all combustibles to burn the rate of burning is dependent on the air supply. The duration of a fire is dependent on the total fire load. If the supply of air to a room is large the burning rate is dependent on the surface area and burning characteristics of the fuel. High ventilation can also have a cooling effect on the fire.

The compartment fire process can be described by three distinct phases, the pre-flashover fire, the fully-developed fire (or post-flashover fire) and the cooling phase. There is a rapid transition stage called flashover between the pre-flashover and fully developed fire. This is shown in Figure 2.3[2.8] which illustrates the whole process in terms of heat released against time. While still small (during the growth phase) the compartment fire will behave as it would in the open. As it grows the confinement of the compartment begins to influence its behaviour. If there is sufficient fuel and ventilation the fire will develop to flashover and its maximum intensity, when all combustible surfaces are burning. If the fire is extinguished before flashover or if the fuel or ventilation is insufficient there will only be localised damage. The fire will remain small around the items first ignited (represented by the broken line in Figure 2.3). Post-flashover the whole enclosure and its contents will be devastated. Structural damage and fire spread beyond the room of origin are also likely unless the fire is in a fire rated enclosure (compartment). Structural fire engineers are concerned with elements of structure subjected to high temperatures. Post-flashover fires provide the worst case scenario. However localised heating of key elements of structure without flashover may also need to be considered.

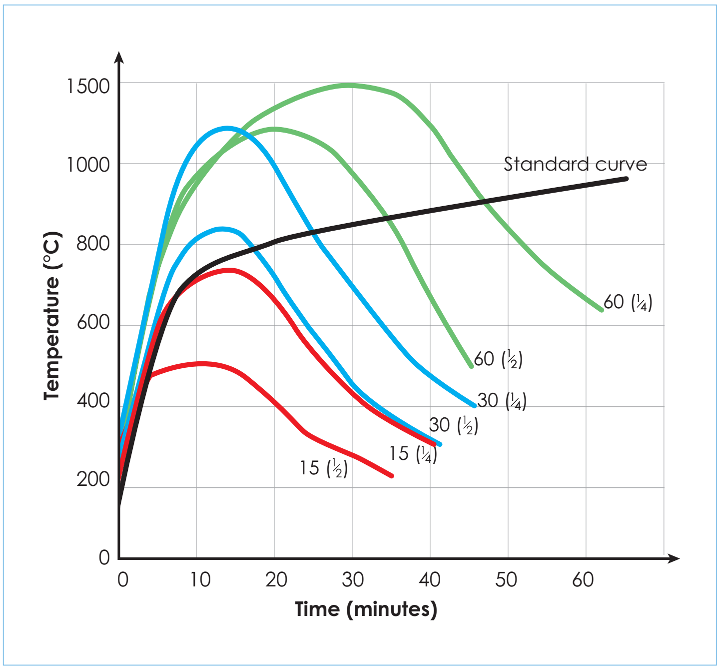


Fig 2.2 Comparison of the Standard Temperature-Time curve with the Temperatures measured during Compartment Fires. (Note: 60(½) implies fire load density equals 60kg/m^2 and ventilation is 50% of one wall etc). © John Wiley & Sons Ltd

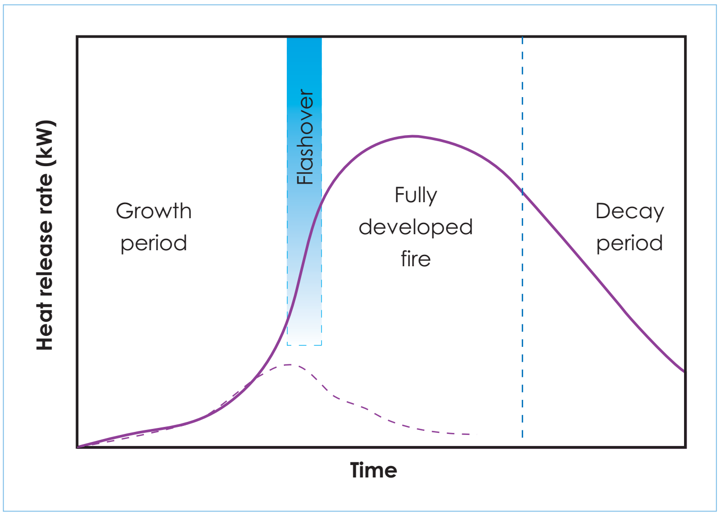


Fig 2.3 History of a well-ventilated natural fire against rate of heat release. The dotted line represents diminution of fuel before flashover has occurred. © John Wiley & Sons Ltd

2.5.1 The pre-flashover fire

During the growth phase of any fire the flames form a buoyant plume above the items first ignited. In a compartment, if the fire grows to a size where the plume impinges upon the ceiling, a ceiling jet will develop, radiating outwards from the central axis of the plume. When the flow of hot gases meet the walls of the enclosure, a hot smokey layer builds up under the ceiling, radiating heat back down towards the lower compartment and the fuel below. The development of the smoke layer is important for flashover. The radiative heat feedback from the dense, hot smoke results in the ignition of many more items in the room, which in turn increases the level of hot gases near the ceiling.

An understanding of pre-flashover fires is very important for life safety in terms of time for evacuation. It can also be important when considering a localised fire adjacent to a critical piece of structure.

2.5.2 Flashover

Flashover is defined as the 'relatively rapid transition between the primary fire which is essentially localised around the item first ignited, and the general conflagration when all surfaces within the compartment are burning.'[2.10] It is a transition period as a result of several mechanisms, each one contributing to the growth of the fire to a size at which flashover becomes inevitable. If there is insufficient fuel, ventilation or propensity for fire spread then a compartment fire may not achieve a rate of heat release sufficient for flashover to occur. The relative duration of the three phases may have a significant impact on the performance of elements of structure.

2.5.3 The post-flashover fire

After flashover high temperatures are sustained until the fuel is almost completely consumed. The compartment is engulfed with hot gases and products of combustion. Rate of heat release is at its highest. External flaming through the windows will also occur as the unburnt products of combustion in the fuel rich atmosphere flow out of the window and burn in the presence of air. Intensive research into compartment fires has centred around the post-flashover fire because the fire is most severe in this period but also because the compartment can be treated as one volume of uniform temperature and composition, simplifying modelling. Many compartment fire models only consider this stage[2.11-2.16].

2.6 Natural fire tests

The development of the fire design rules in Eurocode 1[2.17] (EC1) is based on a number of natural fire tests which have investigated the effect of the location of ventilation openings, type of fire load and the thermal properties of the compartment linings. The developed design approach allows the designer to predict the three-phase description of post-flashover fires comprising growth phase, steady burning phase and a decay phase, based on the actual parameters which define the fire's behaviour. This provides a more realistic design approach compared with the standard fire curve with an ever-increasing growth phase.

2.7 Further methods

The guidance so far relates mainly to current practice for the majority of designers at this time. However, alternative methods which are not explicitly dealt with in this guidance, are available for designing structures and compartmentation, which take into account real fires and real structures. These include:

- The time equivalent concept that makes use of the fire load, ventilation data and thermal properties of the boundary walls in a real compartment fire to produce a value, which would be 'equivalent' to the exposure time in the standard test. The formulation of equivalent fire exposures has traditionally been achieved by gathering data from room-burn experiments where protected steel temperatures were recorded and variables relating to the fire severity were systematically changed (e.g. ventilation, fire load, compartment shape). Time-equivalence calculations such as those developed by Law[2.18, 2.19] and Pettersson[2.12] and set out in EC1[2.17] calculate the fire resistance requirements of the structural elements assuming a total burnout of the fire compartment.
- Parametric fires where the atmosphere temperatures within a fire compartment are calculated on the basis of the compartment construction, geometry, ventilation and fuel load. The parametric equations such as that in EC1[2.17] are based on empirical data. The development of a temperature-time curve to be able to describe the temperature history of a compartment fire has been researched for decades. The earliest significant work was carried out by Kawagoe and Sekine[2.11] in the 1960s. Magnusson and

Thelandersson[2.13] and others[2.14-2.16,2.20] have also contributed significantly. EC1[2.17] describes a method of calculating post-flashover temperature-time curves.

- The energy and mass balance equations for the fire compartment that can be used to determine the actual thermal exposure and fire duration. This is known as the natural fire method. This method allows the combustion characteristics of the fire load, the ventilation effects and the thermal properties of the compartment enclosure to be considered. It is the most rigorous means of determining fire duration.
- Computational modelling of compartment fires using zone models and computational fluid dynamics techniques (CFD).

References

2.1 International Organization for Standardization. *ISO 834 Fire resistance tests. Elements of Building Construction*. ISO, Geneva, 1975.

2.2 *BS 476: Part 20: 1987 Fire tests on building materials and structures. Part 20: Method for determination of the fire resistance of elements of construction (general principles)*. British Standards Institution, London, 1987.

2.3 *ASTM E119-95a Standard Test Methods for Fire Tests of Building Construction and Materials*, 34th edn Volume 4. American Society for Testing and Materials. West Conshohocken, Pa., 1995.

2.4 *Building Regulations 1991. Approved Document B: fire safety*. The Stationery Office, London, 2000.

2.5 *Building Standards (Scotland) Regulations 1990: Technical Standards*. The Stationery Office, London, 1990.

2.6 Statutory Rules of Northern Ireland 2000, no. 389. Building Regulations 2000: *Technical Booklet E, Fire safety*, The Stationery Office, London, 2000.

2.7 *BS 476 Part 21:1987. Fire tests on building materials and structures. Part 21: Method for the determination of the fire resistance of loadbearing elements of construction*. British Standards Institution, London, 1987.

2.8 Drysdale, D. *An introduction to fire dynamics*, 2nd ed., Wiley, London, 1999.

2.9 Karlsson, B and Quintiere, J G. *Enclosure Fire Dynamics*. CRC Press, Boca Raton, Fl., 1999.

2.10 Drysdale, D D. The flashover phenomenon. *Fire Engineers Journal*, pp18-23, 1996.

2.11 Kawagoe, K and Sekine, T. *Estimation of Fire Temperature-Time curve in Rooms*. Technical Report No.11, Building Research Institute, Ministry of Construction, Tokyo, 1963.

2.12 Pettersson, O, Magnusson, S E and Thor, J. *Fire engineering design of steel structures*. Swedish Institute of Steel Construction, Stockholm, 1976.

2.13 Magnusson, S E and Thelandersson, S. *Temperature-time curves of complete process of fire development*, ACTA Polytechnica Scandinavica, Civil Engineering and Building Construction Series No.65, Stockholm, 1970.

2.14 Babrauskas, V and Williamson, R B. Post flashover compartment fires: basis of a theoretical model. *Fire and Materials* **2**, 1978, p39-53.

2.15 Babrauskas, V and Williamson, R B. Post flashover compartment fires: application of a theoretical model. *Fire and Materials* **3**, 1979, p1-7.

2.16 Lie, T T. Fire temperature-time relationships. In: Di Nenno, P J et al, eds. *SFPE handbook of fire protection engineering*, 2nd ed., pp 4.167-4.173. Society of Fire Protection Engineers, Boston, 1995.

2.17 *DD ENV 1991-2-2: 1996. Eurocode 1: Basis of design and actions on structures. Part 2.2: Actions on structures – Actions on structures exposed to fire*. British Standards Institution, London, 1996.

2.18 Law, M. *A relationship between fire grading and building design and contents*. Fire Research Note No. 877. Fire Research Station, Borehamwood, 1971.

2.19 Law, M. A review of formulae for time-equivalent. In: *Fire Safety Science*. Proceedings of the Fifth International Symposium, International Association of Fire Safety Science, London, 1997.

2.20 Profil ARBED Recherches. *Natural fire safety concept, full-scale tests, implementation in the Eurocodes and development of a user-friendly design tool*. Profil ARBED Recherches Technical Report 5. Profil ARBED, Centre de Recherches, Esch-sur-Alzette, Luxembourg, March 2000.

Summary

- Standard fire resistance testing involves testing construction for stability and/or insulation and/or integrity.
- Design methodologies range from basic prescriptive guidance to complex modelling of whole frame structures using finite element analysis, natural fires, heat transfer and structural behaviour.
- Open sided car parks and external structures are often exposed to less severe fires with relatively high levels of ventilation reducing the level of fire resistance required.
- Risk assessment can play an important role in fire design when there are departures from prescriptive fire safety guidance.
- Recent large scale testing of whole frame structures in fire has created a new understanding of structural behaviour in fire.

3.1 Performance criteria for design

The requirements for the design of structures for fire are influenced by means of escape, compartmentation and access for fire fighting. The requirements are wide ranging and recommendations can be found in prescriptive guidance, or alternatively a fire safety engineering approach can be adopted which involves calculations and engineering judgement to determine the impact of a real fire on a building and the occupants. The engineered approach is not covered in detail in this document although some of the concepts are introduced in this chapter.

The following are used to describe the performance of an element of structure in a fire resistance test:

- Stability – the ability of a structure to carry the applied loads, while being acted on by fire (introduced in Chapter 2). Traditional approaches to the design and analysis of structures in fire are based on the behaviour of isolated elements – beams, slabs and columns with idealized support conditions. This is convenient for 'proof testing' and is consistent with the approaches traditionally used for normal design at ambient temperature.
- Insulation – the ability of a structure to limit the transfer of heat within defined limits so that fire does not spread and adjacent spaces do not become untenable. For insulation, the failure criteria are a temperature rise on the unexposed face of 140°C (average) or 180°C (at any single point).
- Integrity – the ability of the structure to prevent the development of significant sized holes to limit the transmission of hot gases. Failure is related to openings forming in a member and defined by ignition of a cotton pad held close to an opening. This reduces the chances of spread of fire and increases the chances of adjacent spaces remaining tenable. Voids in the construction for services or other purposes need to be carefully detailed, usually by the architect, in accordance with the appropriate building code to prevent spread of fire around or through the structure.

The performance criteria which may need to be met depend on the nature of the structural element. For example:

- Stability is required for all structural elements including slabs, beams, columns and load-bearing walls. In addition insulation and integrity may be required to meet the recommendations of the building codes.
- Slabs require insulation and integrity to protect compartments above and below the fire.
- Compartment walls require insulation, and integrity to protect the adjacent space.
- Columns only require stability unless they are built into an element which requires insulation and integrity, such as a compartment wall.
- Beams only require stability unless they are built into a slab or a wall which requires insulation and integrity.

Whenever an approach is being adopted that does not follow the normal prescriptive route and the referenced fire tests it is important that the performance of structural elements in terms of acceptable deflections are reviewed taking into account the full fire strategy for the building. In many cases there will be no need to consider this because escape happens at a relatively early stage in the fire development. However there are cases where there may be a need to control deflection. Examples include:

- Where there is a need to maintain business continuity in a separate compartment or floor.
- Where escape routes may be affected in the later

stages of a fire for example in phased evacuation of tall structures.

- Deflections should not affect compartmentation. Slabs or beams may deflect onto compartment walls but the interaction of the wall and slab should not compromise compartmentation.
- In cases where added fire protection has only been tested up to a certain deflection during the fire test. The risk is that high deflections could result in damage to the fire protection unless additional evidence to the contrary can be provided.

Other issues assessed as part of the fire strategy, that may affect the specification of the performance of structures are as follows:

- Repairability, which influences business continuity.
- Insurance requirements.
- Resistance to explosive damage.
- The impact of earthquakes where active systems may be damaged.
- Design against fire initiating progressive collapse for large or complex buildings[3.1]. Considerations of collateral damage will be relevant in this context.

Deflection criteria may also be important in assessing some of the above performances.

3.2 Influences on the design

The approach taken to achieve suitable fire safe solutions is influenced by the designer's own knowledge and the circumstances for a particular building. A holistic design approach to fire safety should consider a number of issues relating to design and construction. Examples of the issues that are likely to have the greatest influence are discussed below.

- Robustness – Minimise potential construction and long term damage of applied fire protection by selecting a system that is suitably robust for the given situation. Damage caused by maintenance and upgrading of building services is always a risk for most applied fire protection systems.
- Integration and design co-ordination – A structural solution that optimises the normal and fire load case whilst giving sufficient flexibility for services distribution i.e. minimum depth of structure and service zone.
- Use of active fire protection systems, such as the introduction of sprinklers, as a trade off against passive systems (see Section 1.3).
- Selection of passive fire protection minimising waste and time on site and exploring offsite application.

3.3 Design methodology

A range of different approaches can be used to ensure satisfactory structural performance during fire. These vary from simple prescription (e.g. the required thickness of applied fire protection for steelwork, or the cover to reinforcement in concrete) to more complex analytical calculations. The approaches vary for different structural materials, and are described in the appropriate chapters of this guide.

The treatment of the structure as a series of independent elements is common to all. However as the more advanced guidance is developed and further research work is disseminated the designer will have a greater range of potential approaches available both in terms of whole frame action and use of real fires instead of the standard fire. The degree of sophistication adopted by the designer will depend greatly on the particular circumstances and design objectives. The various levels of sophistication can be categorised in increasing order of complexity as shown in Table 3.1.

Table 3.1 Complexity of design process

Level	Procedure	Description
1	Use simple prescribed rules and simple structural elements.	This is relatively straightforward and usually requires reference to National Standards and manufacturer's standard literature.
2	Account for reduced stress levels in the structure to demonstrate improved fire performance in conjunction with the standard fire. Usually involves simple structural elements.	This is still a relatively straightforward process with simple calculations and requires reference to documents such as BS 5950 Part 8[3.2] for steelwork.
3	Introduce parametric fires into level 2. Parametric fires provide a reasonable representation of the time temperature profile of a real fire.	Requires additional skills and knowledge of ventilation conditions and fire loads.
4	Advanced finite element performance of structures in combination with parametric fires. Often involves whole frame structural action.	Requires considerable analytical skill in combination with engineering judgement and represents the current state of the art.

The combinations and approaches listed in Table 3.1 are not exhaustive but do represent a reasonable gradation in design complexity. In some cases particular approaches have been adapted for a number of very common structural types for example:

- Open sided car parks with high ventilation and low fire potential.
- External structures where fire temperatures are often less severe.

3.3.1 Open sided car parks

Open sided car parks have characteristics which have enabled special design approaches to be developed, based on the above principles. In particular they have very high levels of ventilation combined with a low fire load. This results in relatively low temperatures, which are recognized as being equivalent to 15 minutes exposure in the standard fire. Accordingly, *Approved Document B*[3.3] states that in open sided car parks less than 30m high, 15 minutes fire resistance is normally sufficient (higher fire resistance provisions may be needed in relation to lines of compartmentation and elements protecting escape routes).

Fig 3.1 DSS Longbenton – external unprotected steelwork which is an integrated part of the architectural solution for the elevations

The term 'open sided' means each car parking level is naturally ventilated by permanent openings having an aggregate area of at least 5% of the floor area at that level, with at least half the minimum area in opposing walls.

Further guidance can be found in *Design recommendations for multi-storey and underground car parks*.[3.4]

3.3.2 External structures

Structure that is located on the outside of a building (an example of which is given in Figure 3.1) is not subject to the same intensity of fire as the structure inside a building. However the external structure needs to be protected from the intense radiation of the compartment fire and checked for heat transfer from external flaming. This type of design is most commonly adopted with external steelwork. Heat transfer calculations are carried out to check that the steel remains below its critical temperature for the compartment fire and flame projection considered. If the steelwork exceeds its critical temperature the steel can be shielded with suitable construction so that the impact of flames and radiation is reduced. This approach has been accepted in the UK[3.5] and the USA[3.6] for some time but has only recently been introduced into the Eurocodes[3.7].

3.4 Risk assessment as part of advanced engineering approach

Where there is a departure from the simplest prescribed methods there is often a need for the designer and the regulator to examine the robustness of a solution and the consequences of failure as part of the risk assessment process. Although not comprehensive, the types of consideration that the engineer should review in the context of the fire strategy for a building include:

- Consideration of the robustness in terms of performance and maintenance. A fire protection measure with a managed certified maintenance routine is much more reliable. Some solutions require little maintenance since they are well able to withstand the rigours of day to day use to which buildings are subjected.
- Ensure adequate margin of safety where a high degree of reliance is placed on a single fire protection measure.
- Accept lower factors of safety on the individual fire protection measures where they all combine together to achieve the required standard since the chances of every fire protection measure

going wrong at the same time is fairly remote. In a diverse solution the likely outcome of a single fire protection measure (active or passive) failing is a reduction in the factor of safety but not a complete failure to provide the required performance (e.g. means of escape, compartmentation etc).

- Other fire protection measures that could be 'traded off' with fire resistance or used to improve performance include sprinklers and management capability with related audit procedures.

This type of approach is outside the scope of this document. It would normally form part of a fire safety engineering approach and is likely to include more advanced engineering methods.

3.5 Form of construction

Each form of construction and construction material has its own advantages and disadvantages. An understanding of these will enable the designer to plan for any shortcomings in the performance of a particular form of construction. Each of the materials is dealt with in detail in the relevant section. However some typical characteristics that influence the designer's thinking process are simply illustrated here:

- Where steel structures are used in conjunction with precast concrete slab elements, the concrete can be located in such a way as to provide partial protection of the steel elements which will enhance the performance of the structure. A range of proprietary and design solutions are available to effectively shield the steel in this way (see Chapter 5).
- Composite steel and concrete structures perform well in fire when the full 3D behaviour of the frame is considered, taking account of the end restraint conditions provided and alternative load paths available to the individual members (see section 3.6). This benefit is not realised in standard testing.
- Reinforced concrete structures are rarely analysed for fire conditions, but continuity in complete buildings is similar to steel frames. In an in-situ reinforced concrete structure with suitable reinforcement detailing, the connections are generally close to rigid. The continuity is often accounted for in ambient temperature design so there is potentially less additional benefit during the fire case. One issue is the difficulty in predicting spalling (see section 4.1.2), which may expose reinforcement, reducing strength. However, the fire design rules for concrete are well prescribed and covered in design codes.
- Timber construction performs in a substantially different way from concrete or steel and is either protected with fire resisting boarding, particularly for small section sizes, or designed to rely on sacrificial charring of the outer part of the timber. There are circumstances where the performance of real structures can be very different to the prescribed solution. This has become apparent when systems of joists and boards, taken from an existing building, performed to a higher standard than expected from simple charring calculations.
- Load bearing masonry tends to be very robust in its fire performance if it is tied to the structural floors. However care needs to be taken to assess the potential for unprotected steel roof beams to expand and push out the top storey putting the fire fighters and others at risk. Design of walls is based upon prescriptive guidance.

These are just a few of the many considerations. Recent research has demonstrated that, when exposed to fire, whole structures can behave quite differently from the way individual members might respond to the same conditions. This may be for a number of reasons, but especially:

- Structural 'continuity' (through tensile membrane effects or moment continuity in composite floor slabs and connections respectively), which facilitates the exploitation of alternative load paths inherent in the highly redundant structural frame.
- 'Free' thermal expansion, which can cause deformations and additional (second order) stresses.
- Restraint to free expansion, which can lead to induced forces (axial and bending).

The effects of continuity are generally beneficial, whilst those of expansion (free and restrained) are generally detrimental. The effects of continuity and expansions can be very significant and it may be necessary to consider whole building behaviour in relation not only to structural performance but also to the integrity of other building components (e.g. non-load bearing walls). A more advanced engineering approach will be required to deal with structures in this way.

Fig 3.2 Large scale fire tests at Cardington on timber, steel and concrete buildings © Building Research Establishment Ltd

3.6 Large-scale testing at Building Research Establishment (BRE), Cardington (UK) and in Europe

The difference between the behaviour of isolated members and the behaviour of the entire building can have a beneficial or detrimental effect on the overall fire resistance of the building. Similarly, the behaviour of the fire, which is governed by the amount of fuel load (combustible material), ventilation conditions, shape of the enclosed space and the thermal characteristics of the linings, can result in a condition which may be more or less severe than the standard furnace time temperature relationship.

In order to explore these behaviours a number of large-scale tests have been conducted to provide test data which will allow the development of design guidance based on more realistic structural and fire behaviour. However, due to the cost of carrying out large-scale tests they are limited in number and do not cover all possible types of structural forms or fire scenarios.

In the UK, recent large-scale fire tests (see Figure 3.2) to investigate whole building behaviour were carried out at the BRE test facility in Cardington. Compartment fire tests were conducted on the full-scale steel-composite, timber and concrete buildings. The primary aim of these large-scale tests was to identify modes of structural behaviour that could not be identified from the standard fire test. In the tests where natural fires were used, valuable data was also obtained of the effect of the compartment and fuel load characteristics on the behaviour of the fire. The large-scale structural tests are briefly discussed in sections 3.6.1 to 3.6.3.

3.6.1 Fire tests on the steel-composite building

A total of six fire tests[3.8, 3.9], of varying compartment sizes, were carried out on the eight-storey steel-composite building between 1995 and 1996. Four tests were conducted by British Steel (now Corus) and two by BRE.

The building comprised conventional ‘H’ section steel columns, ‘I’ section downstand steel beams and a trapezoidal composite lightweight concrete slab. In almost all the tests, the beams were left exposed to the fire and the heated columns were protected. The steel deck to the composite slab was also left unprotected, as is typical of current practice.

A number of research institutions have modelled the structural behaviour during the fire tests using finite element codes[3.9-3.13]. The prime objective of the modelling was to understand the structural behaviour in terms of the internal forces which could not be measured in the tests.

Conclusions of the experimental program and subsequent analysis were:

- The Cardington composite steel framed building exhibited very stable behaviour under the various fire scenarios tested because of the nature of its highly redundant structural form.
- Composite framed structures possess great reserves of strength by adopting large deflection configurations.

This research has led to published design guidance[3.14, 3.15] that allows a significant number of steel beams to be left unprotected. The approach of using unprotected beams is possible due to the mechanism of membrane action, which could not be identified from the standard fire test carried out on single members, but was identified from the large-scale fire tests.

3.6.2 Fire test on the timber framed building

In 1999 a large-scale compartment fire test was carried out on a six-storey timber framed building as part of a larger programme of research work, known as TF2000. The fire test generated valuable data, which is being used to develop further recommendations on construction methods and standards for medium rise, lightweight timber frame buildings. The primary objective of the compartment fire test was to evaluate the resistance of such a structure to severe natural fire exposure. The compartment used for the test was a single apartment, located at level 3, on one corner of the test building. Ignition took place in the living room, and the test was arranged so that ventilation into the whole apartment created a worst case scenario in terms of fire severity.

The test demonstrated that this form of timber frame construction is able to meet the functional requirements of the Building Regulations in the UK. This involved proving limited internal fire spread and the maintenance of structural integrity.

The results showed that adequate standards of workmanship are crucial in providing the necessary fire resistance performance. Attention needs to be given especially to the correct fixing and finishing of gypsum plasterboards, and other similar protection; also to the correct location of cavity barriers and fire stopping, which is extremely important in maintaining integrity.

3.6.3 Fire test on the concrete building

A full scale concrete fire test, funded by British Cement Association (BCA), Febelcem, Cembureau, CONSTRUCT, Reinforced Concrete Council (RCC) and BRE, was carried out on a 7-storey concrete building at BRE Cardington on 26 September 2001. The overall objective was to investigate the behaviour of a full-scale concrete building subjected to a realistic compartment fire together with realistic applied static load.

The test was carried out on a ground floor compartment 15m x 15m in plan, which included high strength concrete columns. The design fire load was 40kg/m^2 representing typical office fire loading. The performance of the structure was recorded through extensive instrumentation, including thermocouples, displacement transducers, photographs and video recording.

The main results[3.16] and observations from the test are summarised here:

- The building satisfied the performance criteria of load bearing function (R), Insulation (I) and Integrity (E), when subjected to a realistic fire. The imposed load during the test represented a typical office load, with a partial load factor of 0.5 applied to the live load of 2.5kN/m^2.
- The maximum atmosphere temperature recorded was 950°C, before malfunction of some of the instrumentation. It is possible that higher temperatures may have occurred during the test.
- Spalling of the soffit to the first floor was observed; however this did not compromise the structural integrity of the floor under imposed loads.
- The horizontal displacements of the floor slab due to thermal expansion were significant, with a maximum horizontal residual displacement of 67mm being recorded. The lateral displacements of external columns, due to the thermal expansion of the floor slab, will induce an additional moment due to the P-δ effect.
- The high strength concrete columns (103N/m^2) which contained polypropylene fibres, performed very well during the fire. Some corner spalling was evident, but this is considered to be insignificant to the column's stability. Corner spalling can easily be repaired following the fire.
- The slab was able to carry the loads with very low residual displacement (maximum 78mm). Observations from the test suggest that the floor slab was acting in compressive membrane action.

3.7 Appraisal of existing structures

The appraisal of existing structures uses much of the information that is defined in this document. However there are particular considerations that make the appraisal of existing structures more involved. These are described in detail in a guide prepared by the Institution[3.17]. However some principal features of the approach are repeated here. There are two very different requirements:

- There is often a need to try and predict the future performance of an existing structure because of changes in occupancy, changes in ownership or the structure is old and there is a desire to comply with current standards. Knowledge of the materials used and the assumptions that were made at the time are useful. However there is great benefit in being able to reinterpret the structure using a modern engineering approach to explore if there is any redundancy that can be accounted for[3.18] or weak points.
- After a fire there is always a chance that some or all of a structure can be reused and therefore determining the degradation of materials is most important. Reinterpreting the structural performance is also important in this respect. All structures that are exposed to a substantial fire are likely to deflect substantially and/or suffer damage. Compliance with tests does not mean there will be no need for repairs. However it is important to put the cost of structural repairs into context. In a highly serviced building with lifts and expensive cladding which have been affected by fire, it is likely that the structural repair costs will be relatively small as long as the structure has complied with the design performance criteria.

References

3.1 International Working Group, convened by The Institution of Structural Engineers. *Safety in tall buildings and other buildings of large occupancy*. IStructE, London, 2002.

3.2 *BS 5950: Structural use of steelwork in building Part 8: Code of practice for fire resistant design*. British Standards Institution, London, 1990.

3.3 *Building Regulations 1991. Approved Document B: fire safety*. The Stationery Office, London, 2000.

3.4 Institution of Structural Engineers. *Design recommendations for multi-storey and underground car parks*, 3rd ed. IStructE, London, 2002.

3.5 Law, M and O'Brien, T. *Fire safety of bare external steel*. The Steel Construction Institute, Ascot, 1986.

3.6 American Iron and Steel Institute. *Fire safe structural steel. A design guide*. AISI, Washington DC, 1977.

3.7 *DD ENV 1993-1-2: 2001. Eurocode 3: Design of steel structures. Part 1.2: Structural fire design* British Standards Institution, London, 2001.

3.8 Kirby, B R. *British Steel data on the Cardington fire tests*. Technical report, British Steel, 2000.

3.9 Bailey, C G, Lennon, T and Moore, D B. The behaviour of full-scale steel framed buildings subjected to compartment fires, *The Structural Engineer*. **77**, (8), 20th April 1999. p15-21.

3.10 The University of Edinburgh. *Behaviour of steel framed structures under fire conditions*. Final report of the DETR-PIT project. The University of Edinburgh, 2000. (www.civ.ed.ac.uk /research/fire/project/main.html).

3.11 Huang, Z, Burgess, I W and Plank, R J. Non-linear modelling of three full scale structural fire tests. In: *First International Workshop on Structures in Fire*, Copenhagen, June 2000, p53-70.

3.12 Huang, Z, Burgess, I W and Plank, R J. Non-linear analysis of reinforced concrete slabs subjected to fire. *ACI Structural Journal*, Vol. 96, No. 1, 1999, p127-135.

3.13 Usmani, A S, Rotter, J M, Lamont, S, Sanad, A M and Gillie, M. Fundamental principles of structural behaviour under thermal effects. *Fire Safety Journal* **36** (8) 2001.

3.14 Newman, G M, Robinson, J T and Bailey, C G. *Fire safe design: a new approach to multi-storey steel-framed buildings*. SCI Pubn 288. The Steel Construction Institute, Ascot, 2000.

3.15 Bailey, C G. *Steel structures supporting composite floor slabs: design for fire*. BRE Digest 462. Building Research Establishment. 2001.

3.16 Bailey, C G. Holistic behaviour of concrete buildings in fire, *Structures and Buildings, Proc ICE*, vol 152, issue 3, 2002, p 199-212.

3.17 Institution of Structural Engineers. *Appraisal of existing structures*. 2nd edn. SETO, London, 1996.

3.18 CIRIA *Structural renovation of traditional buildings*. CIRIA Report 111. CIRIA, London, 1986.

Summary

- The thermal and mechanical properties of concrete depend on aggregate type, moisture content and mix.
- Concrete has a low thermal conductivity and thus controls the rise in temperature of embedded reinforcement and readily reduces heat transfer through floor slabs and walls.
- Reinforcing steel and concrete lose strength and stiffness at high temperatures.
- Pre-stressing steel suffers a more severe strength loss than reinforcing steel due to its metallurgical properties.
- Cover or axis distance to the reinforcing steel is the most important aspect of reinforced concrete elements for fire design.
- Prescriptive design is essentially based on tabulated data for minimum dimensions and cover or axis distance.
- Spalling – the loss of material from a concrete surface – is dependent on aggregate, moisture content, stress level and temperature, and is very often a localised effect.
- Anchorage of reinforcement is important for continuity at the fire limit state.
- Eurocode 2 allows fire resistance to be calculated using simple analytical techniques incorporating loss of material properties at high temperatures.
- Research based solutions are likely to centre on high strength concrete and an understanding of the 3D behaviour of whole frame structures in fire.

4.1 Reaction of reinforced concrete to fire

Concrete covers a vast array of different materials all of which are formed by the hydration of Portland cement. The hydrated cement paste accounts for only about 24-43% by volume of the materials present so that the aggregate used has a significant effect on the properties[4.1]. Three common aggregates are siliceous aggregates (gravel, granite and flint), calcareous aggregates (limestones) and lightweight aggregates made from sintered fuel ash.

4.1.1 Thermal properties

Concrete has excellent fire resisting properties (see Figure 4.1). Compared with steel it has a very low conductivity, thus low thermal diffusivity. A major disadvantage of concrete is spalling, the loss of surface material as a result of high temperatures.

Lightweight concretes (LWC) have the best thermal properties with half the thermal conductivity of normal weight concrete (NWC). Typical densities of LWC range 1200-1900kg/m^3 but they can be as low as 1000kg/m^3. NWC is in the range 2000-2900kg/m^3. Densities show only slight temperature dependence, mostly due to moisture losses. Limestone concretes are an exception. They show a significant drop in density at about 800°C due to the decomposition of the aggregate[4.2].

Specific heat of NWC increases with temperature whereas specific heat of LWC is almost constant. All concretes with free water experience a sudden rise in specific heat as water evaporates around 100°C.

Fig 4.1 Fire in office building in Kuwait during construction. Post fire photograph showing damage to green concrete prior to carrying out repairs, and the distorted props that were used to support the timber formwork. © Buro Happold

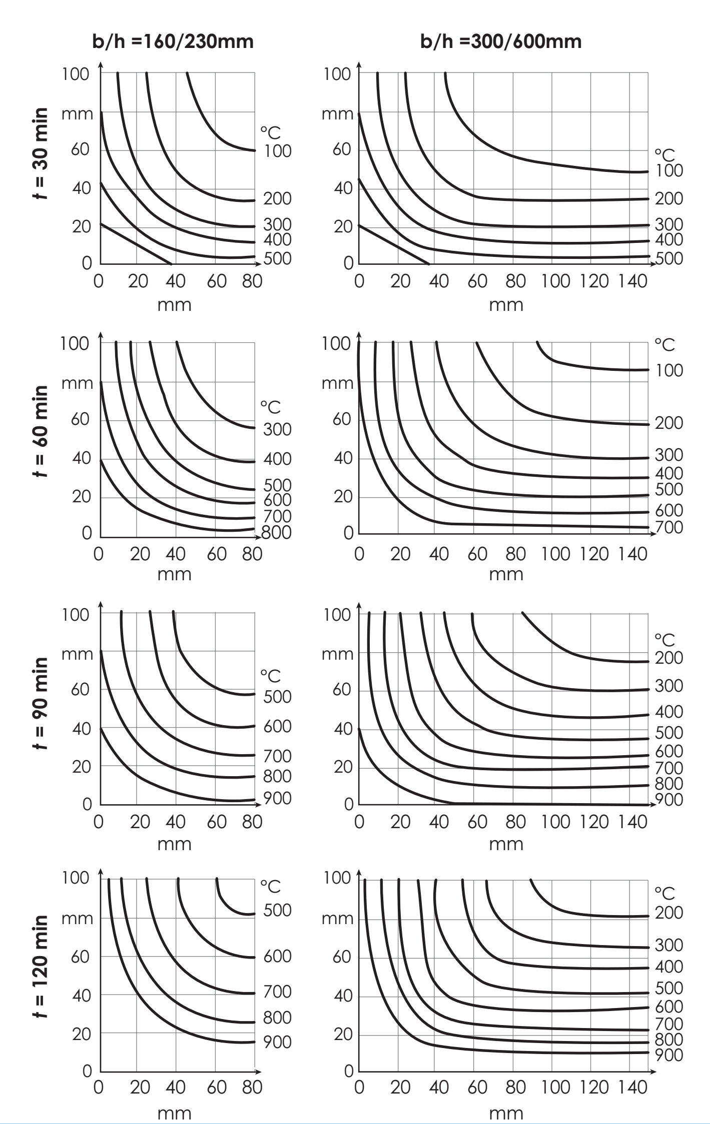


Fig 4.2 Temperature profiles for beams

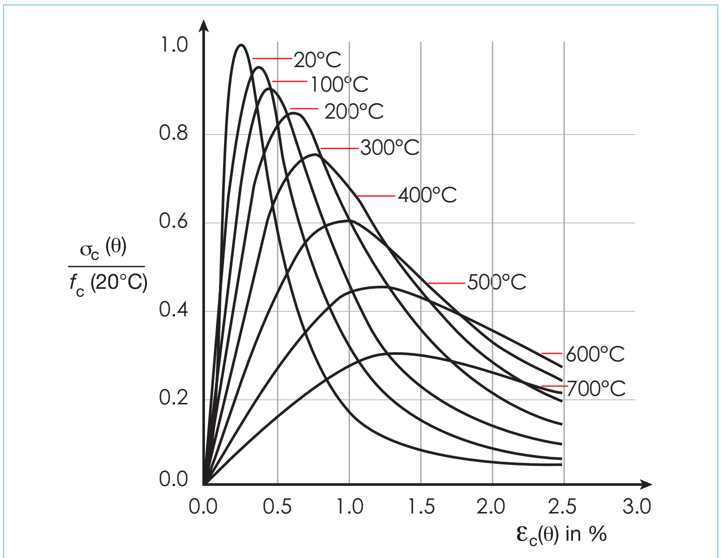


Fig 4.3 Stress-strain curves for siliceous concrete at high temperatures

Moisture in concrete

Free water evaporates from concrete at 100-150°C in the absence of pore pressures whilst chemically bound water remains until temperatures of 450°C[4.2]. Moisture absorbed by the concrete significantly increases its thermal conductivity because the conductivity of air is lower than that of water. In lightweight concrete an increase in moisture content of 10% increases the conductivity by 50%. However the conductivity of the water is less than half that of the hydrated cement paste so the lower the water content of the mix the higher the conductivity of the hardened concrete[4.2].

Modelling heat transfer in concrete

Heat transfer to concrete is complicated by moisture evaporation, water migration, reinforcing steel and heat transfer by radiation and convection in the pores. Nomograms based on the standard fire test have been produced for concrete sections (see Figure 4.2). To calculate the heating effect of natural fires, numerical heat transfer analysis using computer software is very often necessary.

4.1.2 Mechanical properties

Thermal expansion

Thermal expansion, like all other properties of concrete, is complicated by the complex nature of the composite material. It is dependent upon stress level, type of aggregate, % volume of cement paste and rate of heating[4.2-4.4]. Cement paste expands up to 150°C but contracts between 150-400°C. This is associated with water evaporation and chemical changes. However the aggregates may still expand[4.2].

Transient creep

Creep is significant in concrete above temperatures of 400°C. Concrete creep consists of the creep of the cement paste and the creep of the aggregate. The reason concrete does not disintegrate at high temperatures as a result of differential thermal expansion between the cement paste and the aggregate is the ability of the paste to creep[4.4].

Stress-strain relationships

Stress-strain behaviour of concrete is radically different in compression and tension. In tension concrete is often assumed to have zero tensile strength. Strength is affected by type and size of aggregate, % cement paste, preload and water-to-cement ratio at ambient and at elevated temperatures.

The stress-strain-temperature behaviour of concrete with no preload is described by a set of equations in Eurocode 2[4.5] (EC2). Figure 4.3 shows compressive stress-strain data for concrete at elevated temperatures. High temperature creep above 400°C is included implicitly. Both steel and concrete lose strength and elasticity when heated. For concrete, loss in compressive strength starts to occur at around 350°C for siliceous aggregates and at slightly higher temperatures for calcareous and lightweight concrete (see Figure 4.4). Most of the strength is lost by around 800°C although the loss is less severe for non-siliceous aggregate concrete. Loss in tensile strength and elasticity is more severe with loss starting almost immediately with no obvious threshold temperature.

The deterioration of steel depends on the type, with prestressing steel suffering a more severe strength loss than reinforcing steel. This is entirely due to the metallurgical changes induced by the manner of production of the steel[4.6, 4.7]. The concrete cover to the reinforcement insulates the steel from the extreme temperatures of the fire and therefore the thickness of the cover affects the rate of temperature increase of the steel. However it is the reduction in the strength and the stiffness of the steel as it increases in temperature that has the largest impact on the loading capability and deflection of the reinforced concrete section.

Concrete is non-combustible and, because of their typical size, individual elements have a high thermal inertia which results in relatively slow rates of temperature increase through the cross-section (see Figure 4.2 and Figure 4.5). This also helps to keep the steel reinforcement temperatures sufficiently low to avoid significant softening and weakening.

There is, consequently, rarely a major problem with in-situ reinforced concrete structures in fire. This is also in part due to the monolithic nature of the construction and in part due to the existence of alternative load paths should part of the structure lose strength and stiffness. The actual continuity in a structure also allows a far higher redistribution of moments from the ambient condition to the fire condition than the marginal difference between tabular data for simply supported and continuous elements would suggest. Equally there will be a three-dimensional dispersion of loading which may not have been completely utilised in the ambient condition, thus enhancing load-carrying capacity.

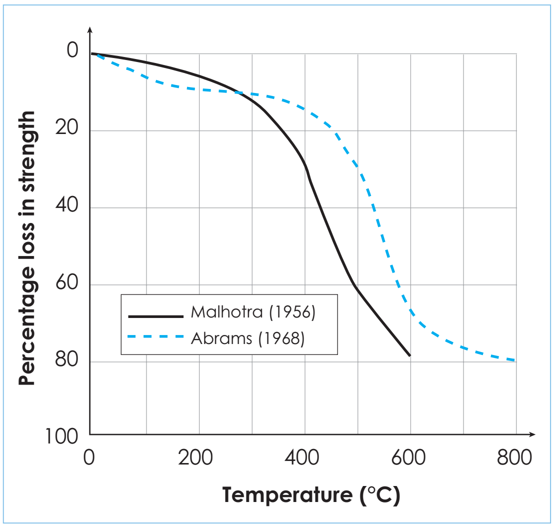


Fig 4.4 Variation of concrete strength with temperature

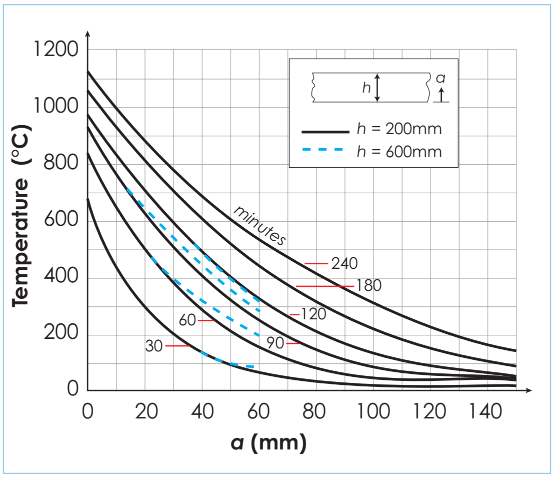


Fig 4.5 Temperature distribution in slabs exposed to a standard fire on one side

Spalling

There may be damage during a fire due to various types of spalling and thermal expansion but these rarely cause more than local failure.

Spalling is the loss of surface material from a concrete element and is dependent on aggregate, moisture content, stress level and temperature. Aggregate splitting is the splitting and bursting of silica containing aggregates due to physical changes in the crystalline structure at high temperatures. This is a surface effect and as such has little effect on the structural performance. Explosive spalling is characterised by large or small pieces of concrete being violently expelled from the surface often exposing reinforcement thus reducing the load bearing capacity of the structure. Normal weight concrete is much more susceptible to spalling than light weight concrete. Spalling is associated with differential expansion and thus can occur under heating or cooling. The onset and amount of spalling will be influenced by the intensity of the fire, the type of aggregate, size and shape of element and stresses to which the element is subjected.

The requirements for cover given in codes such as BS 8110 Part 2[4.8], have generally proved to be adequate in practice for normal strength concretes to prevent failure of concrete elements due to excessive spalling.

4.2 Established practice

For reinforced and prestressed concrete elements within a structure there are two main considerations: insulation and stability. The integrity of the concrete element is considered to be acceptable if the insulation and stability requirements are met.

Knowledge of the fire resistance of concrete elements is generally based on standard testing. In a large number of furnace tests on fully loaded reinforced concrete members, failure occurred when the temperature in the reinforcement reached around 500-550°C and for prestressed members 400-450°C. From these tests the cover to the main reinforcement was considered to be the controlling factor. Only with the advent of calculation methods was it realised that the controlling factor was the axis distance, i.e. the distance from the fire exposed face to the centroid of the reinforcing. This has been reflected in the approach now used in EC2[4.5]. Noting that EC2[4.5] specifies axis distances to the main reinforcing bars, for low standard fire periods (up to 90 minutes) durability will be the design criterion for cover (or axis distance)[4.5, 4.8]. A method for calculating average axis distance is given in Section 4.2.5.

The fire resistance of concrete elements is met by concrete width (in the case of beams, ribs, columns and walls), thickness (in the case of slabs) and cover to the reinforcement. This is typically set out in tabular data in design codes and some typical examples are given below comparing values for BS 8110 Part 2[4.8] and EC2[4.5]. Those from EC2 are taken form the main code with no consideration of the National Application Document.

The use of the simplified tables for Fire Performance in BS 8110[4.8] is conservative as they make assumptions on the size of links or other reinforcement by specifying cover to any reinforcement rather than cover to the main flexural reinforcement given by the tables in Part 2. Some economy may, therefore, be achieved if Part 2 of BS 8110[4.8] is used.

With regard to tabular data two points should be noted:

- The reliability of tabular data has not been proved for high strength (performance) concretes (f_{cc} > 60 N/m^2), and structures with this type of concrete need to be assessed by calculation with allowance for the potentially higher probability of spalling.
- Tabular data for minimum dimensions for beams and slabs do not account for the loading and make only small adjustments for continuity. For beams, whether isolated or part of beam and slab construction, there is some flexibility on member sizing by trading off increased cover against reduced web widths.

4.2.1 Provisions for floors

A comparison of the prescriptive provision for thickness and cover to normal weight (dense) concrete slabs in BS 8110 Part 2[4.8] and EC2[4.5] is given in Table 4.1. The minimum required thickness and cover both increase for longer fire resistance requirements, and whilst there are some differences between the values stated in BS 8110 Part 2 and EC2, these are generally small.

Table 4.1 Minimum dimensions for RC slabs

Time (min)	Simply supported				Continuous			
	BS 8110 Part 2[4.8]		EC2 1992-1-2[4.5]		BS 8110 Part 2[4.8]		EC2 1992-1-2[4.5]	
	Thickness (mm)	Cover (mm)	Thickness (mm)	Axis distance (mm)	Thickness (mm)	Cover (mm)	Thickness (mm)	Axis distance (mm)
30	75	15	60	10	75	15	80	10
60	95	20	80	20	95	20	80	10
90	110	25	100	30	110	20	100	15
120	125	35	120	40	125	25	120	20
180	150	45	150	55	150	35	150	30
240	170	55	175	65	170	45	175	40

Note: The data for continuous slabs is taken from the table for ribbed slabs.

For precast pre-stressed concrete floor slab units, there is potentially an additional problem not identified in 'deemed to satisfy' clauses in design codes, namely loss of bond strength or shear capacity at the support. This is entirely due to the prestressing tendons (or wires) being placed close to the soffit of the unit, therefore heating quickly and losing strength fairly rapidly. In the absence of shear links the prestress contributes to the shear resistance, which decreases due to loss in both prestress and the tensile strength of the concrete. The problem is only likely to be of importance for longer fire resistance periods and short spans.

4.2.2 Provisions for beams

For simply supported beams EC2[4.5] provides a marginally more economic design as it will give slightly lower covers than BS 8110[4.8] when the steel diameter exceeds 20mm. For continuous beams, EC2[4.5] has smaller minimum axis distances but higher values for minimum widths than BS 8110[4.8].

A comparison of the prescriptive provision for width and cover to normal weight (dense) beams in BS 8110 Part 2[4.8] and EC2[4.5] is given in Table 4.2. In both Codes trade off is allowed by increasing the width and decreasing the cover or axis distance.

4.2.3 Provisions for walls

There are no significant differences between BS 8110[4.8] and EC2[4.5] for reinforced concrete walls heated on one side except that at fire periods higher than 180 mins, EC2 gives much higher effective covers for similar thickness. For plain walls acting as partitions EC2 specifies a much lower minimum thickness, although in practical terms these minimum thicknesses are likely to be exceeded at lower fire periods.

4.2.4 Provisions for columns (fully exposed on four sides)

Traditionally the most important factor has been column section size with cover being less important. EC2[4.5] recognises that the load level should also be taken into account, as this has a significant impact on fire resistance.

EC2 gives consistently smaller minimum section dimensions than BS 8110 Part 2[4.8] with the difference increasing as the fire resistance increases. There is little difference between the covers and axis distances given the usual size of reinforcing bars in columns.

For this type of member spalling can be important and thus test data[4.9] need to be assessed with some degree of caution.

A recent development has been to encase concrete columns in permanent formwork, which has the effect of providing containment to the concrete and thereby enhancing its design compressive strength. The effect of this containment has not been investigated under the effects of fire, but the casing will rapidly lose strength and any resultant enhancement to the concrete strength will be lost. Although the level of conservatism in the standard fire performance rules has not been quantified, it is probably necessary to base the load level on such columns using the unenhanced concrete strength to determine the base load level.

4.2.5 Determination of average axis distance

A method of calculating the average axis distance is given in a joint publication by the IStructE and the Concrete Society [4.10]. The average axis distance (a_m) is determined by summing the product of the cross sectional area of each reinforcing bar and the minimum axis distance of the bar and dividing this, by

Table 4.2 Minimum dimensions for RC beams

Time (min)	Simply supported				Continuous			
	BS 8110 Part 2[4.8]		EC2 1992-1-2[4.5]		BS 8110 Part 2[4.8]		EC2 1992-1-2[4.5]	
	Thickness (mm)	Cover (mm)	Thickness (mm)	Axis distance (mm)	Thickness (mm)	Cover (mm)	Thickness (mm)	Axis distance (mm)
30	80	20	80	25	80	20	80	12
60	120	30	120	40	80	20	120	25
90	150	40	150	55	120	35	150	35
120	200	50	200	65	150	50	220	45
180	240	70	240	80	200	60	380	60
240	280	80	280	90	240	70	480	70

the total area of the reinforcing bar.

$$a_m = \frac{\sum A_s a}{\sum A_s}$$

where,

A_s = cross sectional area of tensile reinforcement, wire or tendon

a = axis distance, the distance between the exposed surface of the member and the axis of the tensile reinforcement wire or tendon.

Figure 4.6 illustrates the design method.

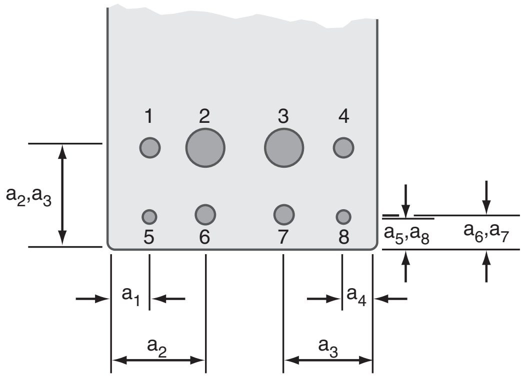


Fig 4.6 Determination of average axis distance

4.3 Construction and Detailing

There are generally two issues that need raising: the first concerns anchorage (and laps) and the second is spalling.

4.3.1 Anchorage

For flexural members where continuity is required in the fire limit state, then laps and anchorage must be detailed to allow for the resultant shifts in the points of contraflexure towards the supports. This has most impact on the top (or hogging) reinforcement.

4.3.2 Spalling

Spalling is a complex process and is aggravated by excessive cover. The use of supplementary mesh to control spalling may be recommended in design codes, but in practice it can be difficult to fix at the mid-depth of the cover where it is most effective.

It should also be noted that for beams and slabs, severe spalling can still occur without unduly affecting fire performance[4.11] provided there is adequate continuity with properly detailed reinforcement to ensure such continuity can be mobilised[4.12].

Polypropylene fibres can be included in the concrete mix to mitigate spalling and is essential in high performance (strength) concrete.

4.4 Eurocode Approach

The Eurocode describes a simple calculation method to justify reducing the axis distance ‘a’ for beams and slabs. It compares the maximum fire design moment with the moment resistance accounting for the reduction of strength in the reinforcement with increasing temperature.

There is also a step by step procedure for assessing the structural response of reinforced concrete in fire by simple statics. The procedure enables profiles of thermal strain to be calculated for a particular cross-section and so describe the behaviour under fire conditions.

EC2[4.5] also gives general guidance on the appropriate approach to adopt when conducting advanced heat transfer and mechanical response calculations. The methods should be based on the acknowledged principles and assumptions of heat transfer and structural mechanics. These recommendations typically relate to finite element modelling.

4.5 Further engineering methods

For any structure where the fire load is low the move to calculating equivalent fire resistance is likely to be beneficial by demonstrating that axis distance (or cover) or member thickness may be reduced below the values required for normal ambient design.

For structures requiring a standard fire resistance of less than 90 minutes, there is little to be gained by going outside guidance in this document. For 120 minutes where there is continuity there may be a case for evaluating beam or slab response. For cases where greater resistance is required then calculations for beams and slabs may provide a more economic solution for flexural reinforcement requirements. The case with columns is less clear as the analysis is more complex and must allow for redistribution of forces within the structural frame induced by the effects of a fire. Calculation methods may be of benefit where there is change of use (and fire resistance) on an older structure in order to reassess its performance.

References

4.1 Harmathy, T Z. Properties of building materials. In: *The SFPE Handbook of Fire Protection Engineering*, 2nd edn. Section 1, Chapter 10, p141-155. Society of Fire Protection Engineers, Boston, 1995.

4.2 Schneider, U. Concrete at high temperatures a general review. *Fire Safety Journal* **13**, 1988, p55-68.

4.3 Malhotra, H L. *Design of fire resisting structures*, Surrey University Press, Glasgow, 1982.

4.4 Khoury, G A, Grainger, B N and Sullivan, P J E. Strain of concrete during first heating to 600°C under load. *Magazine of Concrete Research*. **37** (133), 1985.

4.5 DD ENV 1992-1-2:1996. *Eurocode 2: Design of concrete structures. Part 1-2: General rules – structural fire design*. British Standards Institution, London, 1996.

4.6 Anderberg, Y. *Properties of Materials at High Temperatures – Steel*, RILEM Report. University of Lund, Lund, Sweden, 1983.

4.7 Schneider, U. *Properties of Materials at High Temperatures – Concrete*, 2nd ed., RILEM Report, Gesamthochschule Kassel, Germany, 1986.

4.8 *BS 8110: Part 2: 1985. Structural use of concrete. Part 2: Code of practice for special circumstances*. British Standards Institution, London, 1989.

4.9 Purkiss, J A, Morris, A W and Connolly, R J. Fire resistance of reinforced concrete columns-correlation of analytical methods with observed experimental behaviour. In: Franks, C A and Grayson, S *Interflam 96*, p531-541. Interscience Publications Ltd, London, 1996.

4.10 The Institution of Structural Engineers and The Concrete Society. *Design and detailing of concrete structures for fire resistance*. IStructE, London, 1978.

4.11 Lawson, R M. *Fire resistance of ribbed concrete floors*. CIRIA Report 107, CIRIA, London, 1985.

4.12 Purkiss, J A. *Fire safety engineering design of structures*. Butterworth-Heinemann, Oxford, 1996.

Summary

- Steel loses strength and stiffness at high temperatures and often needs to be protected to reduce the rate of heating.
- Steelwork can be protected with lightweight materials such as intumescent paint, boards, sprays and blankets, or with traditional materials like concrete, or by partially shielding it with other construction elements.
- Relatively simple structural calculations for single elements enable fire resistance to be calculated, often allowing reduced fire protection.
- There are various levels of design guidance given in the Eurocodes[5.1-5.3] which recognise simple and advanced engineering methods.
- There have been significant advances in design guidance for the three-dimensional behaviour of whole frame structures in fire.

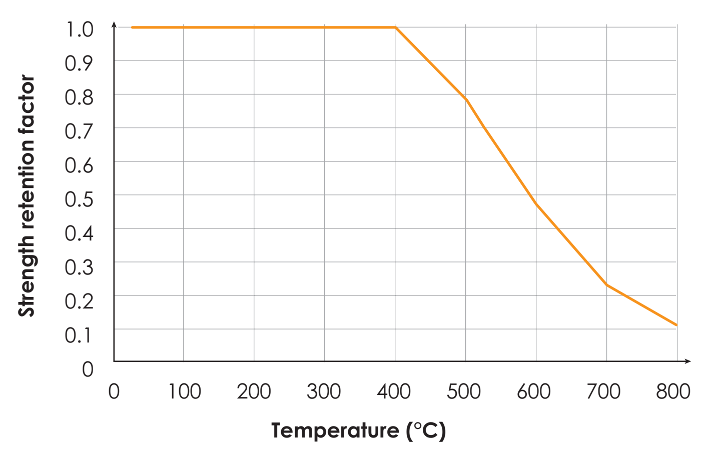


Fig 5.1 Variation of effective steel strength with temperature

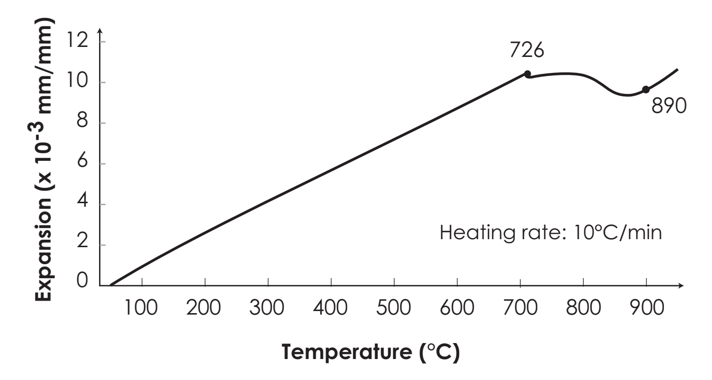


Fig 5.2 Thermal expansion of steel with increasing temperature

5.1 Reaction of steel to fire

Steel loses strength when heated in a fire so, for many years, steel which is required to have fire resistance has been protected by insulation. At typically 500°C – 600°C (higher if stresses are low) steel has reduced in strength to the point where the reserve of strength, which provides the factor of safety assumed for normal (cold) design, has been lost (see Figure 5.1). As bare steel can quickly reach these temperatures in a fire, the need to provide some form of fire protection or allow for the high temperatures in some other way, is clear.

Traditionally the most common fire protection material was concrete. This has now been largely replaced with lightweight proprietary materials. It is not always necessary to use applied fire protection to ensure the necessary fire resistance. Simple to use methods have been developed and these are described in later sections.

5.1.1 Mechanical properties of steel at high temperature

The mechanical properties of steel, typically thermal expansion, stress and strain all vary with increasing temperature.

Thermal expansion

Thermal expansion is a measure of a material's ability to expand (or contract) in response to temperature changes. The coefficient of thermal expansion, α is defined as the expansion of a unit length of material when its temperature is raised by 1°C. Figure 5.2 shows the influence of temperature on the coefficient of thermal expansion for steel. The rate of thermal expansion remains constant up to 700°C when there is a temporary sudden change in behaviour. This is caused by the phase transformation from pearlite to austentite and a rearrangement of the crystal structure. The shrinkage is about 15% of the expansion from 20-700°C. For design purposes an average thermal expansion is assumed. For example, in the UK, BS 5950 Part 8[5.4] assumes 14 x 10^{-6}.

Stress-strain relationships

Figure 5.3 shows stress-strain relationships for hot rolled steel with increasing temperature. At ambient temperature there is a well defined yield between the elastic and plastic portions of the curve. With increasing temperature this is lost, and the stress-strain behaviour becomes highly non-linear, with both strength and stiffness decreasing.

At higher temperatures the concept of proof stress is typically adopted (see Figure 5.4). When calculating the structural performance of a steel member to BS 5950 Part 8[5.4] proof stresses for specific conditions are considered. The level depends on whether the beam is acting compositely with a slab or whether it has any applied protection.

The total strains induced in a structural element during a fire are a combination of thermal and mechanical strains. Thermal strains depend on the temperature and thermal expansion of the material. Mechanical strains are a result of applied loading or restrained thermal movement.

A bilinear representation of the stress-strain behaviour is used for design at ambient. The steel behaves in a linear-elastic manner up to yield at which point it is allowed to strain infinitely with constant stress. In general a bi-linear model of steel does not adequately represent the highly non-linear relationship at higher temperatures.

Eurocode 3[5.2] (EC3) presents non-linear relationships for steel stress-strain curves based on reduction factors for steel stress-strain behaviour at high temperatures. The curves include strain hardening below 400°C.

5.1.2 Thermal properties of steel at high temperature

The rate of heat transfer into a particular material is dependent on three properties: thermal conductivity (k), specific heat (c) and density (ρ). Steel is an exceptionally good conductor. At ambient temperature, steel has a thermal conductivity of 54W/mK which decreases to half this value by 800°C. Beyond 800°C it remains constant (see Figure 5.5). EC3[5.2] assumes a constant value of 45W/mK.

At 20°C the specific heat of steel is about 450J/kgK increasing to 700J/kgK at around 600°C (EC3[5.2] assumes a constant of 600J/kgK). At 730°C steel undergoes a chemical transformation from ferrite-pearlite to austentite. This is associated with a huge increase in specific heat, as shown in Figure 5.6.

The density of steel is approximately 7850kg/m^3 decreasing slightly with increasing temperature.

5.1.3 The heating of steel

In order to better understand how fire protection materials work and are specified some knowledge of why different sizes and weights of steel section heat up at different rates is important (see Figure 5.7). The increase in temperature of a steel beam or column

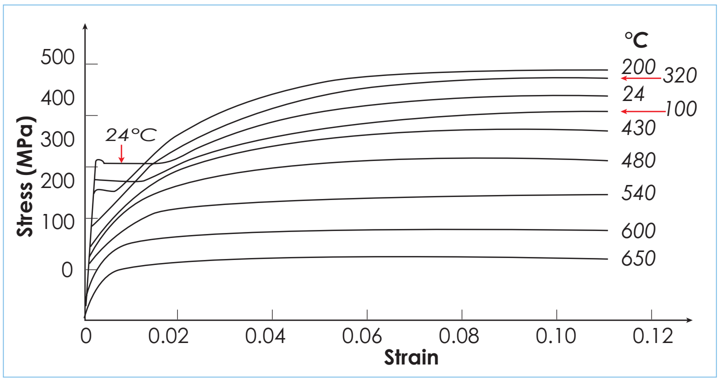


Fig 5.3 Stress-strain curves for typical hot rolled steel at elevated temperatures

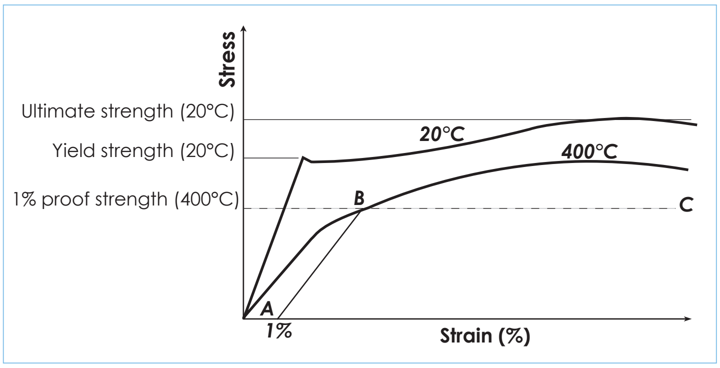


Fig 5.4 Stress strain curves for steel illustrating yield strength and proof strength

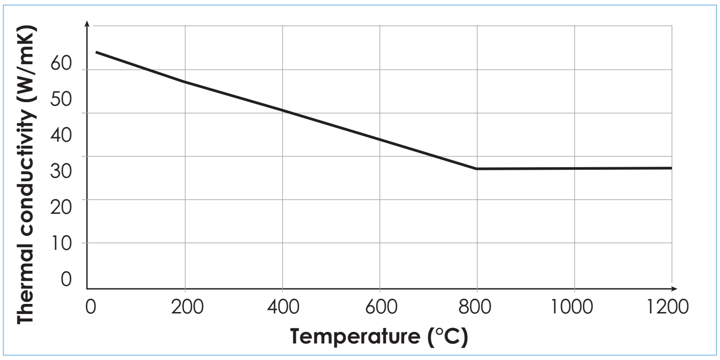


Fig 5.5 Thermal conductivity of steel with increasing temperature

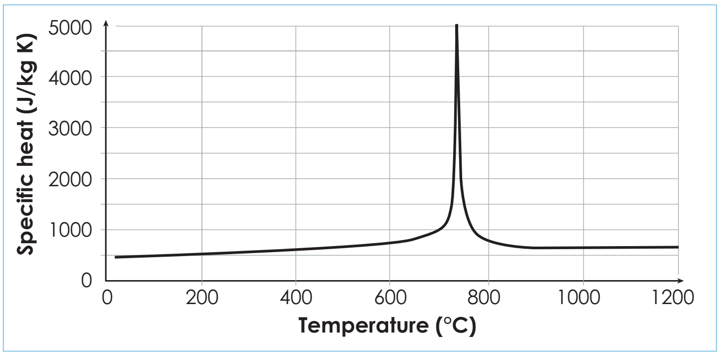


Fig 5.6 Specific heat of steel with increasing temperature

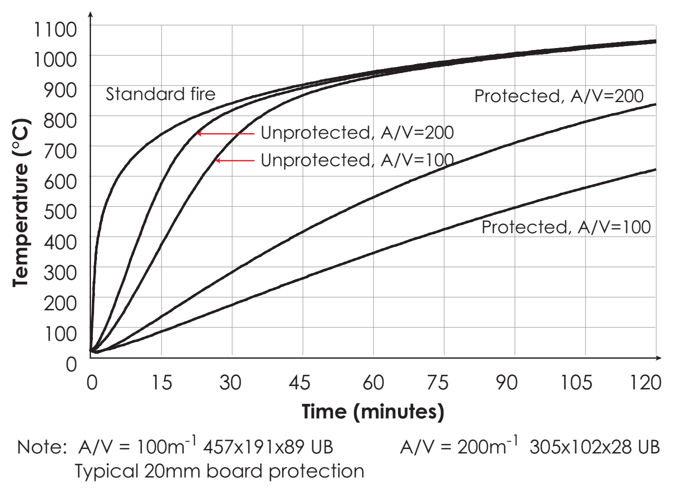
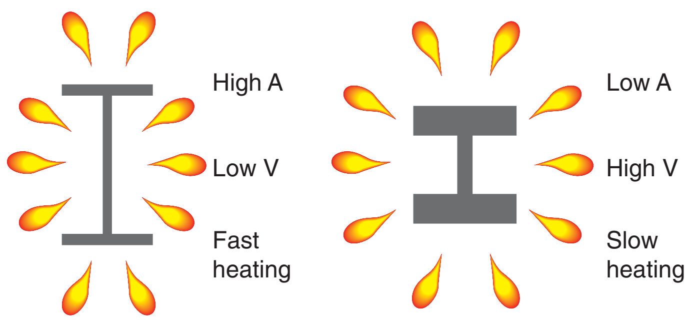


Fig 5.7 Heating rates of protected and unprotected steel beams



Section factor = A/V where;

A = surface area of steel exposed to fire per unit length

V = volume of the section per unit length

Fig 5.8 Concept of the Section Factor

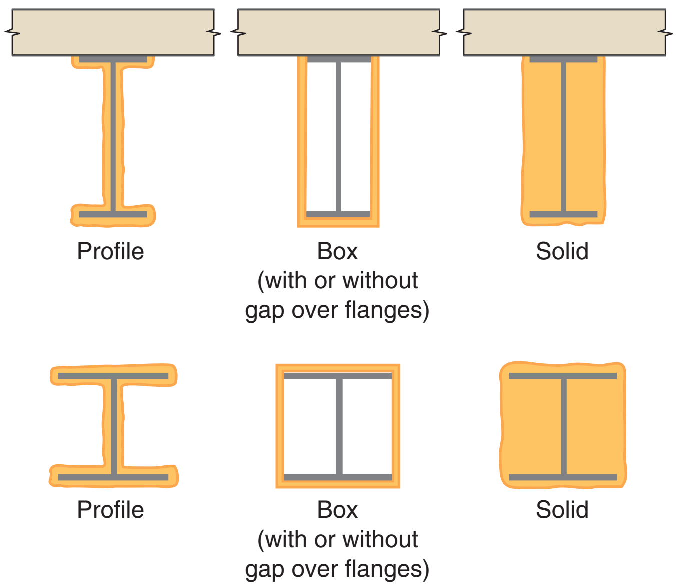


Fig 5.9 Techniques for three and four sided protection

depends on its section factor, defined as the ratio of the exposed surface area to the volume of the member per unit length, A/V (see Figure 5.8). In the UK the section factor is defined as the ratio of the heated perimeter to the cross-sectional area, Hp/A. However, this is exactly equivalent to the Eurocode definition, A/V, which will become the standard definition in the future.

A steel section with a large surface area (A) will receive more heat than one with a smaller surface area. Also, the greater the volume (V) of the section, the greater is the heat sink. It follows therefore, that a small, thick section will be slower to increase in temperature than a large, thin one. The section factor (A/V) thus provides a measure of the rate at which a section will heat up in a fire – the higher the factor, the faster the section will heat up.

The required thickness of fire protection will depend on the section factor of the steel section and the required fire resistance period – the higher the section factor, the greater the required thickness. Fire protection manufacturers present information on their products in terms of the necessary thickness to protect a steel section of a given section factor to provide a given fire resistance. In the UK the 'Yellow Book'[5.5] provides this advice. This publication, produced by the Association for Specialist Fire Protection, is based on fire resistance testing to a critical steel temperature of 550°C. Some manufacturers now test materials to steel temperatures of 620°C.

5.2 Established practice

Fire resistance ratings of steel elements of construction are derived from testing or calculation. The international standard for fire resistance testing is ISO 834[5.6] and was discussed in Chapter 2.

5.2.1 Protected steel

Various generic and proprietary fire protection systems are used to protect structural steelwork (see Figure 5.9). Manufacturers and/or specialist contractors offer comprehensive information on characteristics of materials, test results, advice about suitability for particular applications and installation procedures. This is also discussed in Appendix A.

5.2.2 Partially protected steel

Some types of steel beam or column, notably those where the steel is partially encased in concrete or masonry, may be used without applied fire protection and achieve up to 60 minutes fire resistance. Fully exposed I or H steel sections can only achieve 30 minutes fire resistance in certain limited cases where the loading is low or the section is very large. Their use is generally impractical.

The SCI publication, *The design of steel framed buildings without applied fire protection*[5.7] describes the engineering aspects of the use of unprotected steel members in building frameworks to achieve up to 60 minutes fire resistance. In order to use the published design information it will often be necessary for the user to be familiar with simple calculation methods. The guidance is normally based either on the load ratio/limiting temperature method or on the calculation of moment resistance.

Composite construction

Unprotected downstand steel beams, acting compositely with the supported floor slab (see Figure 5.10), will generally achieve 15 minutes fire resistance. It is possible for the fire resistance of unprotected downstand beams to be increased to 30 minutes, but this typically involves 'over-designing' the section at other limit states so that it is stressed to a lower level at the fire limit state.

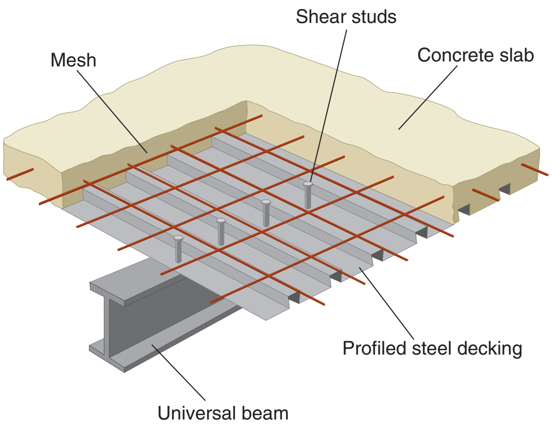
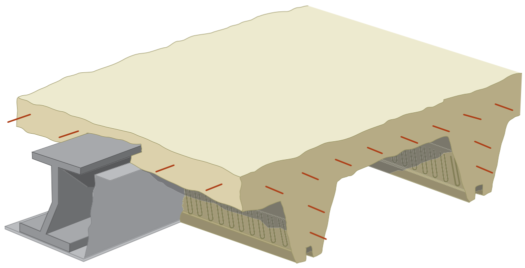


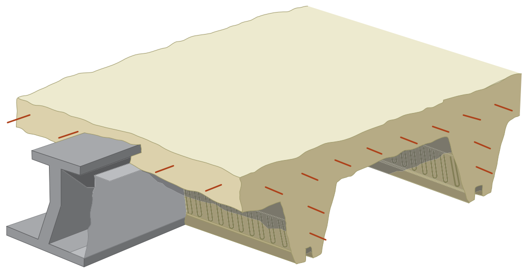
Fig 5.10 Typical composite floor slab

One method of increasing the fire resistance of steel beams consists of incorporating the beams within the depth of the concrete floor slab. Slimfloor beams adopt this approach effectively and act compositely with the supporting floor slab. Some examples of slimfloor beams are shown in Figure 5.11, which can readily achieve 60 minutes fire resistance.

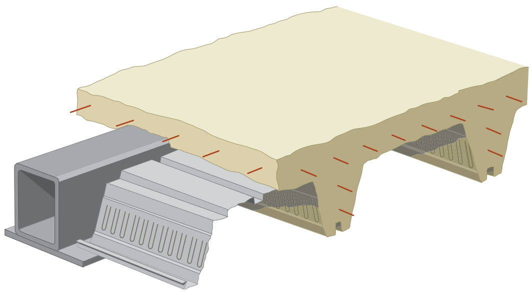
Another form of composite beam, which is commonly used in continental Europe, consists of infilling between the webs of the beam with reinforced concrete (Figure 5.12). The fire design of this form of construction is given in Eurocode 4[5.3] (EC4) and up to 2 hours fire resistance can be obtained. For fire



a) Universal Column with plate welded to beam



b) Asymmetric steel beam



c) Rectangular hollow section with plate welded to bottom flange

Fig 5.11 Types of slim floor beams

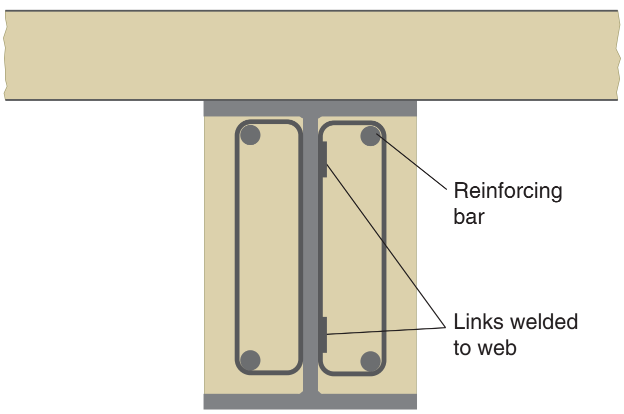


Fig 5.12 Partially encased composite beam

resistance periods of 30 and 60 minutes it may be possible, depending on the load level, to use unreinforced concrete. However, a light mesh is recommended to control any risk of concrete spalling. The concrete is usually placed before erection, with connection areas left exposed and covered following erection. If reinforcement is required, the links need to be welded to the steel beam or bars passed through the web to ensure composite action between the concrete infill and steel beam.

In-filled steel sections

A practical form of a composite column consists of a circular, square or rectangular steel hollow section filled with unreinforced or reinforced concrete (see Figure 5.13). These types of columns have a good inherent fire resistance, with the steel shell being left unprotected for up to 2 hours fire resistance. In a fire the load carried by the outer steel shell is redistributed to the inner concrete core that remains cooler and loses strength and stiffness at a lower rate. Considering this behaviour, the most economical form of a composite column for fire resistance consists of a column where the load carrying capacity of the steel shell is minimised and the load carrying capacity of the concrete core is increased in the cold design. This approach typically results in larger cross-sections of columns compared with those designed for ultimate and serviceability limit states where the steel shell is protected with some passive material to achieve the required fire resistance.

Another form of a composite column consists of infilling between the webs of the column with unreinforced or reinforced concrete (see Figure 5.13). The design of the column with unreinforced concrete is covered by a SCI design guide[5.8], where in a fire it is assumed that the load is transferred from the steel section to the concrete by shear connectors and welded plates. The SCI type of composite column can achieve 60 minutes fire resistance and it is often used in car parks because it also offers good impact resistance. The design of the steel column with reinforced concrete infill is covered by EC4-1-2[5.3] and, like the infill composite beams, is popular in continental Europe.

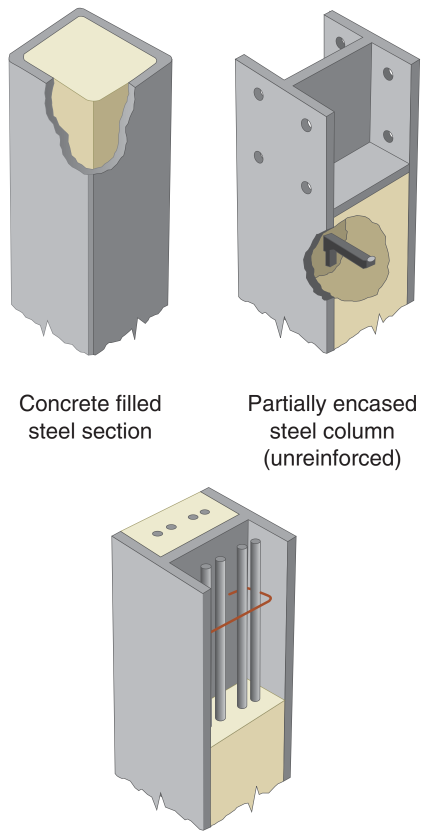


Fig 5.13 Forms of composite columns

5.2.3 Unprotected steel

There are many occasions when it is desirable to use structural elements without applied fire protection. For example:

- An exposed structure is part of the design objective.
- It may be cost effective to design elements in such a way that they are not fully exposed to fire and require no additional protection.
- It may be more reliable to design members so they do not need fire protection as an alternative to an applied fire protection system, because an applied system can be damaged or removed during the life of a building.

Design codes such as BS 5950 Part 8[5.4] and EC3-1-2[5.2] contain simple calculation methods which are similar to those which engineers are used to using at normal temperature. The only difference is that lower partial factors are used at 'the fire limit state' and the strength of materials are reduced. Some methods involve little more than the use of standard tabular data.

The simplest method is the load ratio and limiting temperature method. A technique that is commonly used for beams and floor slabs is to calculate the bending resistance at an elevated temperature. The calculation of bending resistance is the basis for published design tables for many types of proprietary beam and composite floor system.

Partial factors

Partial factors on loads at the fire limit state are lower than for ambient temperature design; for example BS 5950 Part 8[5.4] assumes the values given in Table 5.1.

Table 5.1 Partial factors on loads at ambient and the fire limit state

Load Type	Partial safety factor at ambient	Partial factor in the fire limit state
Permanent dead loads	1.4	1.0
Non-permanent imposed loads	1.6	0.8
Permanent imposed loads	1.6	1.0

Load ratio and limiting temperature

Because the strength of steel reduces with increasing temperature it follows that the load which a member can support will also reduce with increasing temperature. For a member requiring fire protection, the amount of protection required will reduce as the applied load reduces and the failure temperature increases. BS 5950 Part 8[5.4] uses the concept of load ratio as a measure of the applied load that a member can resist at the time of a fire. For different types of member (beam, column etc) the load ratio will correspond to a failure, or limiting temperature. The load ratio is defined as:

$$\textit{Load ratio} = \frac{\textit{Load or moment at time of fire}}{\textit{Member strength at } 20^{\circ}C}$$

The load ratio is a useful concept because it allows different size elements to be considered in the same way. A 200mm deep beam will fail at approximately the same temperature as a 400mm deep beam if they are both working at the same load ratio. In practical designs the load ratio will vary from 0.45 to 0.55. Load ratios much higher than 0.6 are very rare although the maximum value could be as high as 0.7 for an element carrying purely the dead weight of the structure. In most practical situations, the load ratio will not exceed 0.6 and, if the thicknesses of fire protection is being taken from the 'Yellow Book'[5.5], no check of load ratio and limiting temperature are required.

Table 5.2 is an extract from BS 5950 Part 8[5.4]. It lists the limiting temperature for various types of steel construction against load ratio.

Composite floor slabs

Composite floor slabs (see Figure 5.10) comprising profiled steel deck, concrete and mesh reinforcement have a good inherent fire resistance, without the need to protect the steel deck. In a fire the reduced design loads are assumed to be mainly resisted by the mesh or any additional reinforcement, placed in the concrete slab, with the exposed steel deck being largely sacrificial.

In the UK, most composite floor slabs are designed for fire using either the 'Simplified Method', or the 'Fire Engineering Method' as explained in the SCI Publication 056[5.9]. The 'Simplified Method', which can be used for up to 2 hours fire resistance, is predominately based on test results with the mesh reinforcement carrying the load during the fire. The method specifies limits on the

Table 5.2 Limiting temperatures for various forms of construction in terms of load ratio from BS 5950 Part 8[5.4]

Description of member		Limiting temperature (°C) at a load ratio of:					
Loading type	Case	0.7	0.6	0.5	0.4	0.3	0.2
Members in compression, uniform heating on all faces; braced members in simple construction	Slenderness ratio ≤ 70	510	540	580	615	655	710
	Slenderness ratio > 70 but ≤ 180	460	510	545	590	635	635
Members in bending, supporting concrete or composite deck floors	Unprotected members, or protected members complying with clause 2.3 (a) or (b)	590	620	650	680	725	780
	Other protected members	540	585	625	655	700	745
Members in bending, not supporting concrete floors	Unprotected members, or protected members complying with clause 2.3 (a) or (b)	520	555	585	620	660	715
	Other protected members	460	510	545	590	635	690
Members in tension	All cases	460	510	545	590	635	690

Note: clause 2.3 (a) or (b) refers to BS 5950 Part 8 and relates to the maximum strain level of the protection material verified in a standard furnace test.

overall thickness of the slab, thickness of the steel deck, size and position of mesh reinforcement, and maximum allowable design load. In addition the method is only valid for slabs that are continuous over at least two spans.

If the composite slab is outside the limitations imposed by the 'Simplified Method', the 'Fire Engineering Method' can be used which consists of placing additional reinforcement bars in the ribs of the slab and over supports. The method involves calculating the temperatures through the cross-section of the slab, followed by a plastic analysis that allows for the reduced strength of the reinforcing bars and concrete due to elevated temperatures. Any contribution from the steel deck is ignored in the design, due to its high temperature and observed behaviour in fire where the deck debonds from the concrete.

Deep decks, with the steel deck typically 210mm to 225mm deep, are generally used with slim floor beams (see Figure 5.11). The resulting composite floor, which is assumed to be simply-supported between beams, is designed in fire using the 'Fire Engineering Method'. Reinforcing bars of 16, 20 or 25mm are placed in the ribs of the slab and any contribution from the steel deck is ignored in the design.

5.3 Eurocode approach

Eurocode 3[5.2] (EC3) recognises standard fire resistance testing as a means of assessing the structural behaviour of a steel element in fire design but also allows consideration of natural design fires and three levels of structural analysis:

- Single member analysis
- Analysis of portions of the structure
- Global structural analysis

Structural analysis is based on load bearing function. The effect of actions on the structure should be less than the design resistance of the member. The design resistance of a steel section is calculated at time, t, taking account of the temperature distribution in the cross-section and the appropriate reduction in yield strength with temperature.

In terms of fire protection EC3[5.2] recognises the benefits of shielding and states that any method which limits the temperature rise of the steel can be used e.g. water filling or partial protection.

5.3.1 Simple calculation models

EC3[5.2] allows design to a limiting temperature based on a utilisation factor being the ratio of the 'design effect of actions' to 'design resistance at ambient temperature'. This is equivalent to the load ratio concept in BS 5950 Part 8[5.4].

The heat transfer to unprotected and protected steel can be calculated using the equations in EC3. These are simple lumped mass heat transfer models. Input data includes the thermal properties discussed in Section 5.1.2 and the appropriate 'section factor' introduced in Section 5.1.3. The position of the steel member is taken into account by calculating a resultant emissivity value which has values in the range 0.3-0.7. An emissivity of 0.7 is associated with a member totally engulfed in flames and 0.3 relates to a member which is remote from direct fire exposure.

5.3.2 Heat transfer to external steelwork

Where steelwork is placed on the outside of buildings, EC3[5.2] allows heat transfer calculations developed by Law and O' Brien[5.10] to assess the need for fire protection. The calculations are based on steady state fire conditions and consider:

- Radiative heat flux from the fire compartment
- Radiative and convective heat flux from the external flames through the windows
- Radiative and convective heat loss from the steelwork to the ambient surroundings
- Size and location of the structural steelwork
- Through draught conditions.

In the USA the same calculations are adopted but they do not consider through draught conditions[5.11]. Through draught conditions change the shape of the external flame assumed in the design. With no through draught the flames will hug the façade above the window, whilst with a through draught the flames will project diagonally straight out of the window.

5.4 Further engineering methods

Relatively simple further engineering methods are available to determine the fire resistance of steel. The simplest of these is the moment capacity method for beams. If, at the fire limit state, the temperature distribution across a beam is known, the moment resistance of the beam can be calculated. However, most engineers will not have access to such temperature data and cannot therefore use the method. Design tables and software, published by organisations such as The Steel Construction Institute, are often based on the method. In these cases the temperature distribution is normally obtained from fire tests supplemented by thermal analysis. For some types of composite beam, EC4-1-2[5.3], gives rules to obtain the temperature distribution and guidance on calculating the moment resistance.

Further steps in using steel in interesting ways in fire are based on more detailed use of the available design codes, fire safety engineering codes and design guides published by organisations such as The Steel Construction Institute, BRE (Building Research Establishment) and Corus. Generally two approaches are possible – structural design, and fire design. In the former the structure is designed at the fire limit state to resist applied loads and heating, whilst a fire design approach uses fire safety techniques and codes such as EC1-2-2[5.1], to determine the design fire to which the structure is subjected. The design fire will normally be less severe than the standard fire, and it may be possible to reduce the fire protection provisions. The two approaches, structural design and fire design are often used in tandem. Analytical calculations which allow designers to combine structural design with fire design can be found in a number of references[5.12-5.17].

As a direct result of the BRE Cardington tests a new design guide has been produced by SCI entitled *Fire safe design – a new approach to multi-storey framed buildings (P288)*[5.18]. It enables secondary steel beams to be left unprotected by capturing the enhanced strength of slabs at large deflections as a result of tensile membrane action. The basic method is presented as a series of tables and is applicable to structures of similar construction to Cardington i.e. non-sway frames with composite steel decks. The method relies on composite action between any edge beams and the slab to carry tensile membrane action. This method does not apply to fabricated beams, pre-cast concrete slabs or slim floor construction.

More advanced calculation methods are available for steel fire design. These vary in scope and complexity up to a fully fire engineered modelling of the fire and structural response based on finite element analysis.

References

5.1 *DD ENV 1991-2-2. Eurocode 1: Basis of design and actions on structures Part 1: Basis of design, and Part 2.2: Actions on structures exposed to fire*. British Standards Institution, London, 1996.

5.2 *DD ENV 1993-1-2: 2001. Eurocode 3: Design of steel structures Part 1.2: Structural fire design*. British Standards Institution, London, 2001.

5.3 *DD ENV 1994-1-2: 2002. Eurocode 4: Design of composite steel and concrete structures Part 1.2: Structural fire design*. British Standards Institution, London, 2002.

5.4 *BS 5950: Structural use of steelwork in building Part 8: Code of practice for fire resistant design*. British Standards Institution, London, 1990.

5.5 Association for Specialist Fire Protection. *Fire protection for structural steel in buildings*, 2nd edn. SCI Pubn 13. Steel Construction Institute, Ascot, 1992.

5.6 International Organization for Standardization. *ISO 834 Fire resistance tests. Elements of Building Construction*. ISO, Geneva, 1975.

5.7 Bailey, C G, Newman, G M, and Simms, W I. *The design of steel framed buildings without applied fire protection*. SCI Pubn 186. Steel Construction Institute, Ascot, 1999.

5.8 Newman, G M. *The Fire resistance of web infilled steel columns*. SCI Pubn 124. The Steel Construction Institute, Ascot. 1992.

5.9 Newman, G M. *The fire resistance of composite floors with steel decking*, 2nd Edition. SCI Pubn 56. The Steel Construction Institute, Ascot. 1991.

5.10 Law, M and O'Brien, T. *Fire safety of bare external steel*. The Steel Construction Institute, 1986.

5.11 American Iron and Steel Institute *Fire safe structural steel. A design guide*. AISI, Washington DC, 1977.

5.12 *Competitive steel buildings through the natural fire safety concept*. Final report No. 32. CEC agreement 7210-SA/125, 126, 213, 214, 323, 423, 522, 623, 839, 937. Profil ARBED, Centre de Recherches, Esch-sur-Alzette, Luxembourg, 1999.

5.13 Buchanan, A H. *Structural design for fire safety.* Wiley, 2001.

5.14 Lie, T T. *Structural fire protection*. ASCE Manuals and Reports on Engineering Practice No. 78, 1992.

5.15 Milke, J A. Analytical methods for determining fire resistance of steel members. Section 4, Chapter 9, *The SFPE handbook of fire protection engineering*, 2nd Edition, 1995.

5.16 Malhotra, H L. *Design of fire resisting structures*. Surrey University Press, Glasgow, 1982.

5.17 Pettersson, O, Magnusson, S E and Thor, J. *Fire engineering design of steel structures*. Swedish Institute of Steel Construction, Stockholm, 1976.

5.18 The Steel Construction Institute. *Fire safe design: new approach to multi-storey steel framed buildings*. SCI Pubn 288. Steel Construction Institute, Ascot, 2000.

Summary

- Timber is a combustible material – the rate of combustion depending on the ratio of surface area to volume, timber density and moisture content.
- Density varies greatly from species to species and as a result each species responds differently to fire.
- The presence of fissures accelerates fire growth because they increase the surface area.
- The natural chemical process of charring provides an insulating layer and protects the unexposed cross-section of timber below the char layer.
- Fire retardants are available to reduce the surface spread of flame but they are not fire resisting.
- There are simple design solutions for timber in fire, treating uncharred timber as ‘cold’.
- More sophisticated methods of design also exist providing greater accuracy and more economic solutions.
- Careful consideration has to be given to connection and detailing to achieve fire resistance especially with modern factory fitted interlocking connections.
- Further engineering methods for fire resistance consider temperature and moisture content at any point in the cross section taking into account material degradation with increasing temperature.

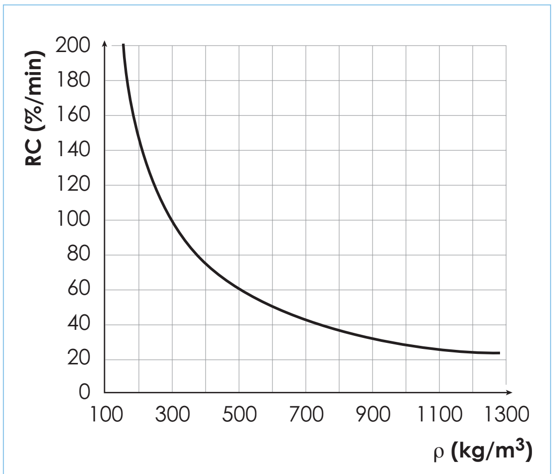


Fig 6.1 Relationship between density (ρ) and rate of combustion (RC)

6.1 Reaction of timber to fire

Timber and wood-based materials consist mainly of cellulose and lignin, which themselves are formed from carbon, hydrogen and oxygen[6.1]. They are therefore combustible, and it is almost impossible to make them totally non-combustible. However, complete incombustibility is only necessary in rare cases. In general, timber and wood-based panel products behave in a very predictable manner in fire. Procedures, codes and standards are well established for their design and specification under fire loading conditions.

6.1.1 Effect of density on fire performance

The time taken for wood to ignite and for combustion to spread is primarily dependent upon its density (see Figure 6.1). Density is one of the most fundamental characteristics of wood, varying significantly according to the botanical order, class, family, genus and even species of the tree from which the timber is produced[6.2]. Thus, different species, and the various wood composite types that are produced from them, vary considerably in their behaviour under the influence of fire[6.3,6.4]. Broadly, the higher the density, the slower the combustion rate. This effect has been well known, at least empirically, for centuries. In the early days of underground railway development, for example, it was soon realised that dense hardwoods such as Jarrah (*Eucalyptus marginata*) were ideal for use as sleepers in tunnels because of their slow charring response to hot tinders.

It was also known empirically that there is a varying propensity of different species of an approximately similar weight to ‘catch fire’ and subsequently burn. This is due to the varying natural chemical contents of the wood (volatiles, extractives etc.) according to species, but is masked by the over-riding significance of density itself.

6.1.2 Influence of hygroscopicity and moisture content on fire performance

Hygroscopicity – the affinity of wood to water, both at the molecular and at higher levels – is another fundamental property of wood and of the timber and other products that are derived from it. Moisture content has a profound influence on all aspects of physical and mechanical performance, including behaviour in fire.

6.1.3 Effect of shape and fabrication of timber on fire performance

Combustibility is dependent not only upon the inherent material, but also upon the surface-to-volume ratio of the specimen or element. The greater the combustibility rating, the more easily ignition starts, and the faster flames spread. This is easily imagined by considering the rate of combustion of a sheet of paper, compared with that of a match, of approximately equal mass and moisture content.

In timber structural elements, sharp corners and coarse surfaces enlarge this surface-to-volume ratio, and result in less favourable fire behaviour. Fissures (the generic term for all forms of crack, shake and split in timber) also exacerbate the effects of fire, by causing larger surface-to-volume ratios. Hence, for example, the fire performance of glulam, Laminated Veneer Lumber (LVL) and other structural timber composites[6.5], tends to be better than that of solid sawn timber, which is more liable to fissuring. LVL, which is a laminated structural composite made from bonded veneers, and other modern composite timber elements of a similar type, are described and discussed in reference 6.5. Urea, resorcinol and melamine adhesives tend to ensure good performance, having very thin bond lines that are generally treated as equivalent to the parent material. Epoxy based adhesives behave less well, but are generally installed in situations where there is considerable surrounding sacrificial timber.

6.1.4 Charring depths

Charring is a favourable phenomenon that is fundamental to understanding the behaviour of timber structures in fire[6.3]. The effective remaining depths (beneath the insulating charred layer) for members exposed to fire, are calculated by means of the charring rate. Simple calculations assume a linear relationship between the charring advance and time, with the latter calculated directly from the time of exposure to the fire.

Referring to Figure 6.2, central sections are insulated from the heat by the outer, exposed layers, which convert to charcoal and are gradually consumed. Central, or 'residual' sections are relatively unaffected by fire. In simple methods of design, they are treated as effectively the same as the 'cold' material. In Figure 6.2, fire barrier materials e.g. gypsum plasterboard, are shown. Provided these are satisfactorily constructed, they alter the pattern of exposure and consequent charring, as suggested in Figure 6.2.

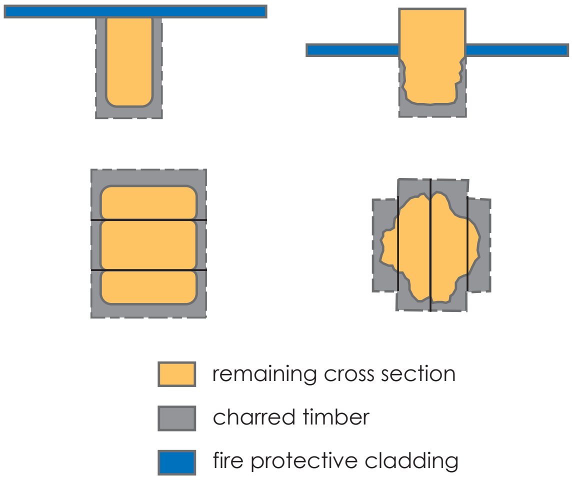


Fig 6.2 Beams and columns before and after the exposure to fire on three or four sides © Centrum Hout Almere Netherlands

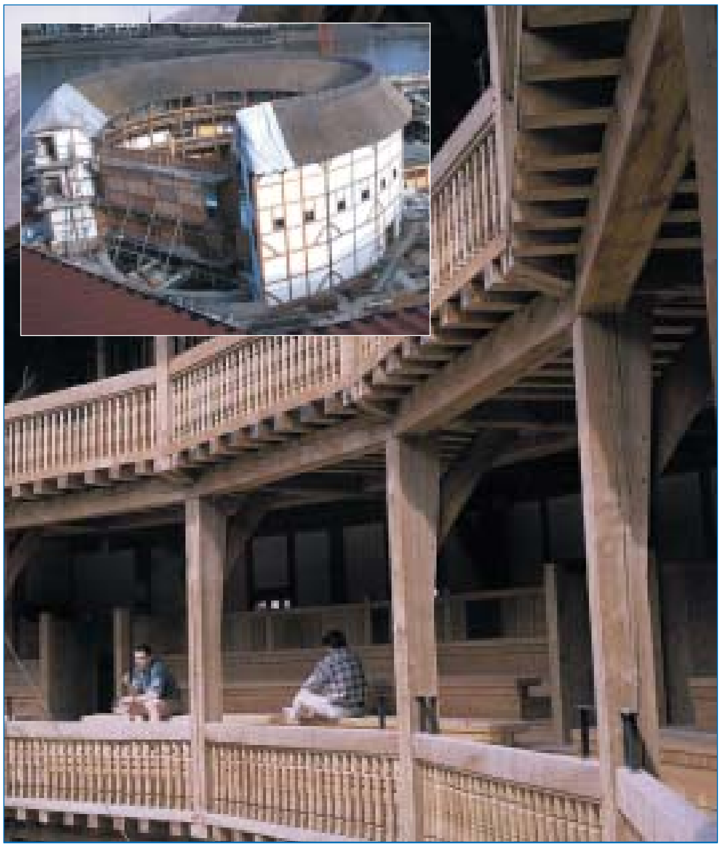


Fig 6.3 Globe Theatre, London, UK – an example of a timber building where the large timbers provided the necessary fire resistance without any additional fire protection. © Buro Happold

The Globe Theatre is an example of a timber building where the large dense timbers provided the necessary fire resistance without any additional fire protection (see Figure 6.3).

6.1.5 Use of fire retardant

Fire retardant can be used to improve the surface spread of flame characteristics, with impregnation methods being preferable unless maintenance of surface treatments can be provided. The retardant does not improve the fire resistance, because even though the timber is relatively non-combustible the timber will still char in the event of a fully developed fire. Some retardant processes may have an adverse effect on normal cold strength properties however responsible manufacturers will advise on this.

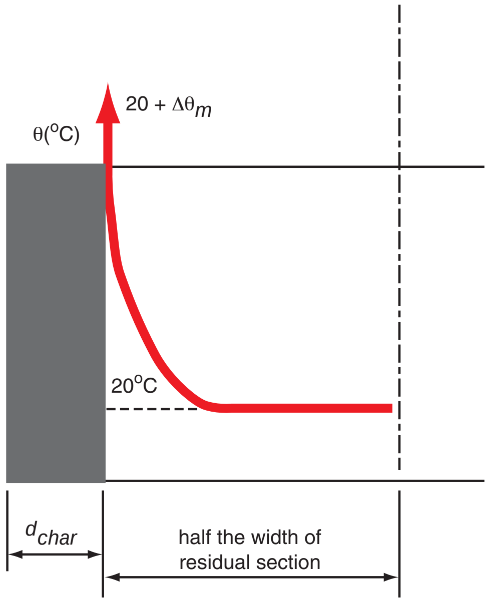


Fig 6.4 Temperature profile for heating on one side where the width of residual section is greater than the distance required for the temperature to fall to ambient.

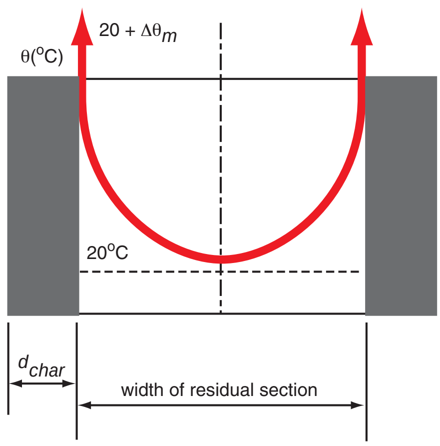


Fig 6.5 Temperature profile for heating on one side where the width of residual section is less than the distance required for the temperatures to fall to ambient.

6.2 Established practice

Three levels of calculation method are available, and are recognised by both Eurocode 5[6.6] (EC5) and BS 5268[6.7]. Detailed application rules are given for the first two. In increasing order of complexity, these methods are:

1 Effective cross section method
2 Reduced strength and stiffness method
3 General calculation methods

The effective cross-section method and the reduced strength and stiffness method are useful to obtain approximate results for fire resistance. They may well be sufficient to avoid costly full-scale testing. Neither method is adequate if the fire resistance period needs to be calculated very precisely, or if second order effects, such as reductions in element stiffness leading to significant short-term local instability risks, are not negligible.

For timber elements covered by combustible panel or diaphragm materials, such as laminated decking, that are to be included in calculations, more advanced design procedures, possibly supported by testing, are likely to be called for [6.8].

6.2.1 Temperature profiles

The temperature at an actual char-line in softwood timber is typically about 300ºC. The char-line temperature relating to the elementary expression:

$$d_{char} = \beta_0 t$$

is approximately 200ºC where:

d_{char} = depth of char (mm)
β_0 = charring rate (subscript 0 indicates base case)
t = time (min)

For a fire exposure of more than 20 minutes, ambient temperatures are reached at a distance below the char-line which remain constant for the remaining exposure time (see Figure 6.4). This distance is about 30mm beneath the actual char-line, and for the char-line computed with

$$d_{char} = \beta_0 t$$

it is about 25mm. Gradients are modified when thin cross-sections are used, and when fire exposure occurs on two opposing faces of the element (see Figure 6.5).

6.2.2 Effective cross section calculation method

With the effective cross section method, the time of fire resistance depends upon the load bearing capacity of the un-charred remaining cross-section. It is necessary to calculate the effective cross-section (see Figure 6.6) using an elementary expression of the form:

$$d_{eff} = d_{char} + k_0\, d_0$$

where the constant d_0 is obtained from test calibrations and experience. A simple explanation of its basis is given by Hartl[6.3]. The coefficient k_0 is obtained from tables in EC5[6.6]. It depends upon factors such as the protection or otherwise of the surfaces affected by the fire, and the duration of resistance required. Representative values of variable mechanical actions, accounting for the accidental design situation of fire exposure, should be introduced in accordance with Eurocode 0[6.9].

6.2.3 Reduced strength and stiffness method

This method is also derived from research and testing of temperature profiles in timber. The approach resulting in the application rules was to carry out integration of temperatures within the profiles, giving averages that can be used in calculations. The load carrying capacity is calculated for the residual cross-section in a similar manner to the Effective cross-section method. The method can therefore be regarded simply as a slightly more precise version of the above.

Due to an almost linear relationship between temperature, strength and stiffness properties, an expression was found in which the reduction factor can be calculated as a function of the perimeter of the fire-exposed cross section (p) and the area of the residual cross section (Ar).

6.3 Construction and detailing

In lightweight timber frame construction such as that of the platform frame type[6.10], structural connections (using typically nails, bolts or timber connectors such as split rings and shear plates) tend to be fully protected by the normal lining materials and frame assembly systems. Full sized building tests employing real fires in lightweight multi-occupancy medium rise construction[6.11] have confirmed that fire protection does indeed exist at all of the conventional panel junctions which are normally used in this type of building.

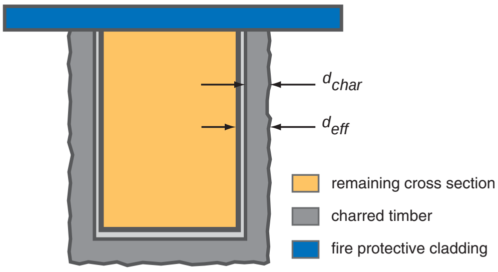


Fig 6.6 Effective cross section when heated on three sides © Centrum Hout Almere Netherlands

To improve productivity and quality, factory-fitted interlocking connections are becoming more popular. In designing and assessing these, and other innovations that are intended to speed up fabrication and assembly, attention should be paid to the almost fortuitous protection offered by the better established methods. Additional fire protection may be required, where the 'traditional' overlapping of protective layers and linings such as gypsum board may not occur.

The fire resistance of exposed connections may need to be calculated. There are well-established design procedures and details, such as steel connecting plates buried within the timber elements themselves (see Figure 6.7 and 6.8). These are held by tightly fitted steel dowels, whose heads may, if necessary, be protected by plugs of wood or wood-based materials. These can be designed to provide effective cover, using the standard timber engineering fire design principles. An introduction to ways of achieving fire resistance of such connections is described by Hartl[6.12].

Fig 6.7 Sibelius Hall, Lahti Finland – junction detail of roof parasol beams to column tops in balcony area of atrium – connection steel work is entirely embedded within glulam to provide fire protection. © C.J.Mettem, TRADA Technology

The fire resistance of the unprotected connection, determined through a table of values that depend upon fastener type and geometry, is modified (improved) using an expression that depends upon the charring rate of the plugs or cover material. Guidance is provided on the length of protective plugs needed, other fixings, and on geometry. As mentioned above, connections with inserted steel plates are a common technique in timber engineering for larger structures. Protected connection design methods for these are also provided in the Eurocode and its supporting documents. These methods generally rely on bonded-in metalwork or adhesively fastened edge strips or other fire-resistant or sacrificial covers, placed over the slits that are provided for the inserted plates.

Fig 6.8 Sibelius Hall, Lahti Finland – view of atrium, ticketing and enquiries desks from gallery – note integrated restored brick building to rear and vertical wind girders supporting glazing to right – these incorporate embedded fire resistant flitched steelwork connections.

Another recent technology being applied by timber engineers for very large structures, including Olympic stadia and bridges, is bonded-in steel rod connections[6.13]. Because of the deep embedment of the rod-like fastening devices within the structure, these have excellent fire resistance, although demonstration testing of this capability is at present somewhat limited[6.14]. Restoration and upgrading applications are another application of these bonded-in rod techniques, and some full-sized fire resistance testing has also been undertaken in this context[6.15].

It should also be pointed out that not all of the problems related to fire resistance are calculable. Fire testing remains an important approach in relation to structural floor and wall design for domestic and medium-scale structures.

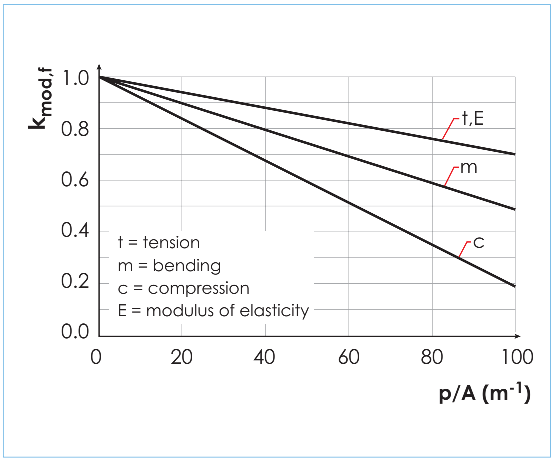


Fig 6.9 Variation of $K_{mod,f}$ with p/A

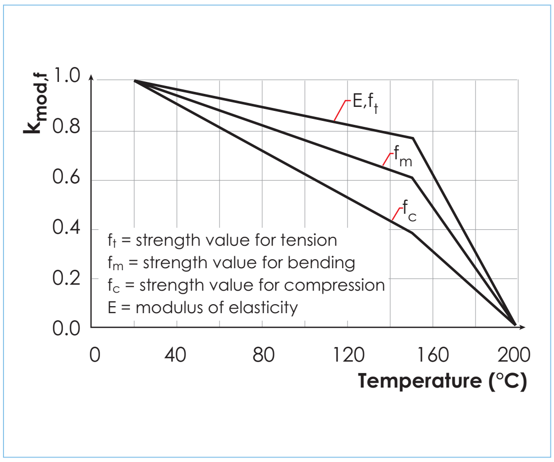


Fig 6.10 Temperature dependent strength and stiffness properties

6.4 Further engineering methods

Only a brief introduction to general calculation methods is provided here. However more advanced approaches take into account the temperature and moisture content at any point in the cross section. Also, the relationship between strength and stiffness properties and temperature and moisture content are employed. Therefore, an increase in the amount of design work is inevitable. However, such complex methods lead to more economical forms of construction. They are also required, alongside techniques such as fire modelling, to derive design principles and application rules for innovative elements, connections and components.

For load-carrying (strength) verification, the design strength and stiffness values are determined using $k_{mod,f}$ coefficients, in a very similar format to standard design, where comparable expressions are applied for load duration, for example. Another k coefficient is included to adjust cold material properties to equivalent mean values, because lesser reliability is demanded during what is regarded broadly as an accident emergency period for the structure. The reducing factor $k_{mod,f}$ takes a value reflecting the influence of temperature and moisture content on strength and stiffness in the case of fire (see Figures 6.9, 6.10).

For deflections (serviceability limit states) the stiffness values are derived from the mean moduli given in standard design properties tables, using a similar approach to the above. Criteria for deformation limitations are far more liberal than with standard cold design, again reflecting the accidental and relatively transient nature of fire safety assurance in general.

References

6.1 Desh, H E, Rev. and Dinwoodie, J M. *Timber its structure, properties and utilisation*, 6th edn. Macmillan, London, 1981.

6.2 Anon. *Wood – decorative and practical*. Wood Information Sheet 2/3-6, TRADA. High Wycombe, 1999.

6.3 Hartl, H. Behaviour of timber and wood-based materials in fire: STEP lecture A13. In: Blass, H J et al. *Timber engineering STEP 1*, Centrum Hout, Almere, The Netherlands, 1995.

6.4 Kollmann, F F P and Coté, W A. *Principles of wood science and technology. Volume 1. Solid Wood*. Springer, Berlin, 1968.

6.5 Mettem, C J, Gordon, J A and Bedding, B. *Structural timber composites*. Design Guide 1. TRADA, High Wycombe, 1996.

6.6 *DD ENV 1995-1-2: Eurocode 5: Design of timber structures. Part 1.2 General rules for structural fire design*. British Standards Institution, London, 2000.

6.7 *BS 5268: Part 4: Fire resistance of timber structures, recommendations for calculating fire resistance of timber members*. British Standards Institution, London, 1978.

6.8 Hartl, H. Timber frame houses – Fire resistance. In Blass, H J et al. *Timber engineering STEP 2*, STEP lecture E12. Centrum Hout, Almere, The Netherlands, 1995.

6.9 *BS EN 1990: 2002. Eurocode. Basis of structural design*. British Standards Institution, London, 2002.

6.10 Hartl, H. Fire resistance of timber members. In Blass, H J et al. *Timber engineering STEP 1*. STEP lecture B17. Centrum Hout, Almere, The Netherlands, 1995.

6.11 Mettem, C J, Bainbridge, R J, Pitts, G C and Enjily, V. Timber frame construction for medium-rise buildings. *Progress in Structural Engineering and Materials*, **1** (3) 1998 p253-262.

6.12 Hartl, H. Fire resistance of joints. In Blass, H J et al. *Timber Engineering STEP 1*. Centrum Hout, Almere, The Netherlands, 1995.

6.13 Bainbridge, R J and Mettem, C J. Bonded-in rods for timber structures: a versatile method for achievement of structural connections. *The Structural Engineer*, **77** (15), 1999, p24-27.

6.14 Kangas, J. *Capacity, fire resistance and gluing pattern of the rods in v-connections*. CIB-W 18/33-7-10 International Council for Research and Innovation in Building and Construction, Working Commission W18 – Timber Structures, Meeting Thirty-Three, Delft, Netherlands, August 2000.

6.15 Mettem, C J, Page, A V and Robinson, G C. *Repair of structural timbers – Part 1 Fire resistant repairs*. Research Report RR 1/93 Pt2, TRADA, 1993.

Summary

- Historically, substantial masonry walls have been shown to have excellent performance in fire and they were one of the original codified measures.
- The thermal and mechanical properties of a particular type of masonry are dependent on the material of the masonry units and the mortar.
- Thermal gradients through the depth of a masonry wall can result in deflection of the wall towards the fire, giving rise to eccentric loading and additional stresses.
- If the extent of the wall deflection is less than the wall thickness the resulting eccentricity is unlikely to promote failure.
- There is very little research into the effect of fire on masonry; therefore design is based on testing and real fire experience (historical performance) and information is very often in tabular form.

Fig 7.1 Historically, substantial masonry walls have provided a good performance in fire

7.1 Reaction of masonry to fire

Masonry is defined as an assemblage of masonry units laid in a specified bond pattern and jointed with mortar; predominantly of single-leaf or cavity construction. The masonry units covered are bricks or blocks of fired brickearth, clay or shale, calcium silicate, dense and lightweight aggregate concrete, autoclaved aerated concrete or manufactured stone. Natural stone units are not included.

Masonry units and the mortar used to build masonry walls are non-combustible (see Figure 7.1). As passive materials they are Class 0 spread of flame. Following initial drying out after construction walls will generally contain moisture with the amount varying with ambient conditions. Some of this water is chemically bonded to the constituent materials, whilst the remainder is located in pores of the material, as liquid or vapour, known as 'free water'. Both components of the moisture content move in response to applied high temperatures, and affect heat transfer through the material and thus the insulation performance of walls.

The behaviour of walls in fire is affected by the temperature distribution and the thermal stresses in the wall. In a fire, masonry walls are predominantly subjected to heating on one face, giving rise to a thermal gradient through the thickness of the wall. For low thermal conductivity materials such as masonry, the temperature distribution through the wall will be non-linear and thermal stresses will be induced. The thermal gradient leads to deflection of the wall towards the fire resulting in further stresses due to the eccentricity of the load. This is illustrated in Figure 7.2. As long as the extent of deflection is less than the wall thickness, the resulting eccentricity, whilst imposing higher compressive stresses, is unlikely to promote failure.

In general, for a given material type, fire resistance increases with:

- thickness
- thermal resistance of the body material
- absorbed or combined water content
- any non-combustible applied finishes, especially insulating plasters and renders.

7.2 Established practice

Walls in a building may be required to act as structural support to transfer vertical or horizontal forces or may act as barriers to a fire in order to restrict its spread and minimize damage. The ability of a wall to perform these functions in the fire condition is termed its fire resistance.

Fire resistance of masonry is always based on tests carried out on the element (i.e. brickwork or blockwork walls, columns etc). There are no provisions for individual masonry units or for the bonding mortar.

Taking the UK as an example, current assessment of the performance of masonry walls is based on either:

a) a specific result obtained to determine the performance of a particular wall construction in accordance with the requirements of BS 476[7.1] or
b) the use of tables which detail the fire resistance of various walls and forms of construction which can be used to satisfy design requirements. These tables are based on wall test results from standard fire tests carried out over a period in excess of 50 years.

The *Approved Document B*[7.2] to the Building Regulations provides practical guidance on meeting the fire safety requirements, and contains details intended to cover some of the more common building situations, while at the same time allowing alternative ways of demonstrating compliance with the appropriate requirements. Appendix A of the Approved Document[7.2] draws attention to the fact that any test evidence used to substantiate the fire resistance rating of a construction should be carefully checked to ensure that it demonstrates compliance adequate and applicable to the intended use. Small differences in detail, such as fixing methods, joints, dimensions, may significantly affect performance. For example the use of polypropylene wall ties, which are likely to be destroyed during fire will have an impact on the assessment[7.3].

The tables list the minimum thickness of walls, of various types of construction, required in order to provide a stated fire resistance period, ranging from 30 minutes up to 6 hours. Tables are detailed in:-

- BS 5628 Part 3: 2001[7.4]. Code of practice for use of masonry, Part 3: Materials and components, design and workmanship.
- Part II of the Building Research Establishment Report *Guidelines for the construction of fire resisting structural elements*, BR128, BRE 1988, Authors Morris W.A., Read R.E.H., Cooke G.M.E[7.5].
- Eurocode 6 *Design of masonry structures, Part 1-2, General Rules – Structural Fire Design*, together with the related UK National Application Document[7.6].

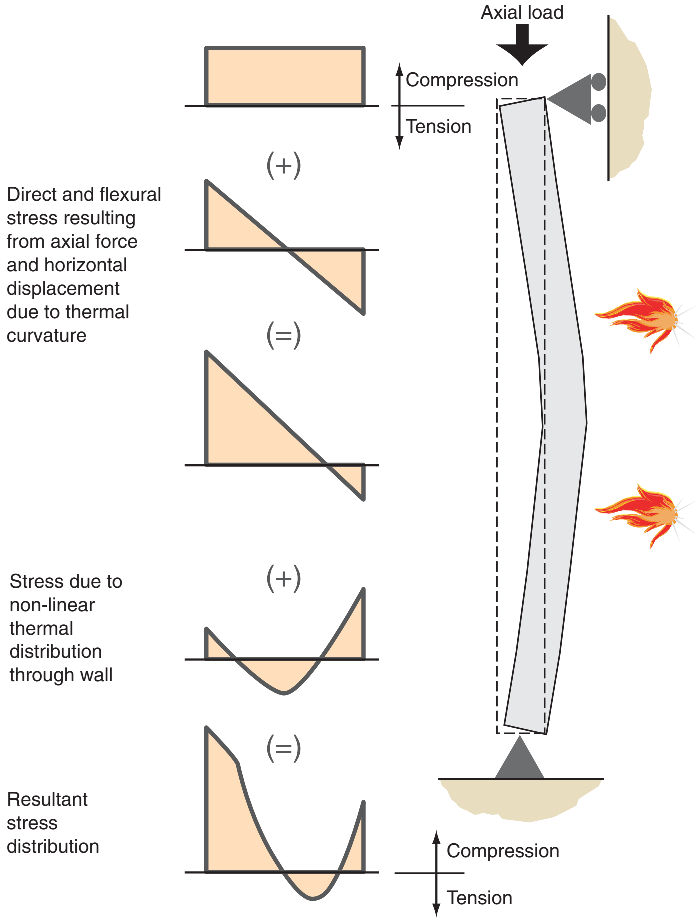


Fig 7.2 Resultant stress distribution in walls due to an external axial load and induced thermal stresses

The approach used by the three documents referred to in the left hand column is described in the following sections to illustrate the use of the different tables.

7.2.1 BS 5628 Part 3: 2001 Approach

Table 15 of BS 5628 Part 3[7.4] gives fire resistances of walls for various types of construction. Other forms of construction may be used, if evidence of satisfactory performance in use, based on the results of standard fire resistance tests, is produced. Data is provided for:

- Loadbearing single-leaf walls (section A)
- Non-loadbearing single-leaf walls (section B)
- Loadbearing cavity walls (section C)
- Non-loadbearing cavity walls (section D)

The fire resistance period given is the time from commencement of the tests laid down in BS 476[7.1] until failure first occurs under any one of the listed criteria – stability, integrity and insulation.

Non-loadbearing walls are assumed to carry only their own weight and be provided with suitable edge restraint. Loadbearing walls may carry any load as determined by the relevant design codes. Interpolation, between A and B, or between C and D is not permitted.

A typical table extracted from Table 15 of BS 5628[7.4], covering loadbearing single-leaf walls, is reproduced in Table 7.1, and indicates the masonry materials and unit types covered.

Table 15 of BS 5628[7.4] states no limit on the wall height. However, the test procedure in BS 476[7.1] requires specimens to be 3m in height, and care should therefore be exercised when designing walls in excess of this height. The draft Eurocode provides some slenderness ratio limitations for loadbearing and non-loadbearing walls.

Data is provided for walls without a finish, or with surface finishes – vermiculite gypsum, (designated VG) or sand gypsum (SC/SG, with or without lime). The latter two are deemed to provide an equivalent performance, and both may be substituted by an equivalent thickness of plasterboard for fire resistance periods up to 2 hours. Finishes should be not less than 13mm plaster or rendering on each face of a single-leaf wall and on the exposed faces of a cavity wall.

Table 7.1 Extract from Table 15 of BS 5628[7.4] – Fire resistance of loadbearing single-leaf walls.

Material	Masonry unit type	Finish		Minimum thickness of masonry for period of fire resistance (mm)						
				6 h	4 h	3 h	2 h	90 min	60 min	30 min
(A) Loadbearing single-leaf walls										
Fired, brick-earth, clay or shale	Brick	Solid brick	None	200	170	170	100	100	90	90
			VG	170	100	100	90	90	90	90
		Not less than 75% solid, e.g. perforated	None or SC/SG	–	200	200	170	170	170	100
			VG	200	170	170	170	100	100	90
		Not less than 50% solid	SC/SG	–	–	–	215	215	215	215
			VG	–	215	215	215	215	215	215
		Not less than 40% solid	None or SC/SG	–	–	–	–	–	215	215
	Block (outer-web not less than 13mm thick)	Two cells not less than 50% solid	SC/SG	–	–	–	100	100	100	100
		Three cells 60% solid	SC/SG	–	150	150	150	150	150	150
Concrete or calcium silicate	Brick	Solid brick	None	200	190	190	100	100	90	90
			VG	200	100	100	90	90	90	90
Concrete, class 1 aggregate	Block	Solid brick	None	150	150	140	100	100	90	90
			VG	150	100	100	90	90	90	90
		Other, e.g. hollow	None	–	–	–	100	100	100	90
Concrete, class 2 aggregate	Block	Solid brick	None or SC/SG	–	–	–	100	100	90	90
			VG	–	100	100	90	90	90	90
		Other, e.g. hollow	SC/SG	–	–	–	–	–	–	190
			VG	–	–	–	200	200	190	190
Aerated concrete, density 480kg/m^3 to 1200kg/m^3	Block	Solid brick	None	215	180	140	100	100	90	90
			VG	180	150	100	100	90	90	90

Note: Data is provided for walls without a finish, or with surface finishes – vermiculite gypsum, (designated VG) or sand gypsum (SC/SG, with or without lime).

For solid walls the thickness is the sum of the work sizes of two units together with the work size of the joint between them. BS 3921[7.7] defines 'work size' as the size of a brick (or unit) specified for its manufacture, to which its actual size should conform within specified permissible deviations.

Where the fire resistance of a loadbearing cavity wall is more than 2 hours, the imposed load should be shared by both leaves. Otherwise, if the load is carried by the fire exposed leaf only, the minimum thickness of the fire exposed leaf should equate to that given for loadbearing walls.

As a simple example using Table 15 of BS 5628[7.4], a standard 2.65m high single leaf loadbearing wall, with an overall thickness of 102.5mm, of solid clay brick with no surface finish, should achieve a fire resistance rating with respect to stability, integrity and insulation, of 2 hours.

7.2.2 BR128 Approach

Specific reference is made to this BRE report[7.5] in Appendix A of *Approved Document B*[7.2]. Part II of the BRE report[7.5] tabulates periods of fire resistance of masonry walls in Tables 1-3 (of BR128). Although presented in a different manner, the fire resistance data provided is equivalent to that given in BS 5628 Part 3[7.4].

Tables 1-3 of BR128[7.5] detail the information for masonry walls, and includes data for reinforced and unreinforced concrete walls. Other tables relate to framed internal and external walls. Table 1 of BR128 applies to solid loadbearing and non-loadbearing walls required to resist fire from one side at a time. Table 2 of BR128 applies to hollow masonry walls required to resist fire from one side. Table 3 of BR128 provides information on loadbearing and non-loadbearing cavity walls required to resist fire from one side. A typical table for masonry walls, extracted from Table 1 of BR128 of the report, is reproduced in Table 7.2, for solid loadbearing walls.

Whereas BS 5628 Part 3[7.4] gives wall thicknesses necessary to achieve up to 6 hours fire resistance, these tables limit information to achieving a maximum of 4 hours for all wall types. This is usually sufficient to meet all practical needs in respect of Building Regulations requirements and any additional insurance requirements.

Table 7.2 Extract from Table 1 of BR128[7.5] Masonry walls: solid loadbearing (Required to resist fire from one side at a time)

Nature of construction and materials	Minimum thickness (mm), excluding any finish, for a fire resistance (mins) of:					
	30	60	90	120	180	240
6 Bricks of clay, brickearth or shale						
a) without finish	90	90	100	100	170	170
b) with 13mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
7 Bricks of concrete or calcium silicate						
a) without finish	90	90	100	100	190	190
b) with 13mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
8 Blocks of dense concrete						
a) without finish	90	90	100	100	–	–
b) with 13mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
9 Blocks of lightweight concrete						
a) without finish	90	90	100	100	140	150
b) with 13mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
10 Blocks of autoclaved aerated concrete (of density 480 to 1200kg/m^3)						
a) without finish	90	90	100	100	140	180
b) with 13mm lightweight aggregate gypsum plaster	90	90	90	100	100	–
11 Blocks of autoclaved aerated concrete (of density 400 to 479kg/m^3)	100	100	120	130	160	200

Referring to the same example as before, the standard 2.65m high single leaf loadbearing wall, 102.5mm thick, item 6 of Table 7.2 (extract of Table 1 of BR128[7.5]) indicates that the clay masonry wall would achieve a fire resistance of 120 minutes.

7.3 Construction and detailing

Construction and detailing of masonry walls for fire relates mainly to compartmentation requirements. The requirements for masonry walls are readily met. Guidance is given in the Building Regulations on aspects relating to the passage of pipes and services through separating walls, and in particular, the need for fire stopping.

Recesses and chases, which are permitted in loadbearing walls without the need for separate calculation, can be assumed not to reduce the fire resistance performance of the wall.

For non-loadbearing walls, vertical chases and recesses should leave at least:

- $^2/_3$ of the required minimum thickness of the wall, including any applied fire resistant finish such as plaster
- 60mm.

Horizontal and inclined chases and recesses should not be positioned within the middle one-third height of the wall and in non-loadbearing walls should leave at least

- $^5/_6$ of the required minimum thickness of the wall (including any applied fire resistant finish)
- 60mm.

The width of individual recesses in non-loadbearing walls should not be greater than twice the required minimum thickness of the wall (including finishes as above).

Where movement joints or edge clearances are specified in walls required to resist fire, they should be filled with a non-combustible material, such as mineral fibre, allowing the movement joint to continue to function. Consideration should be given to non-combustible cover strips fixed to both faces of the wall on one side of the joint. Joints in walls, or between walls and other fire separating members, must be designed and constructed to meet the required fire resistance.

7.3.1 Applied fire protection

It is not normally necessary to enhance masonry wall performance with protective surface finishes, but where used for decorative reasons, they can enhance performance.

Design tables allow for enhancement of performance using vermiculite: gypsum plaster. Insulation layers of non-combustible materials, e.g. mineral wool, can also improve performance when compared with a wall having no surface finish.

Combustible thin damp proof materials incorporated into a wall may be ignored when assessing fire resistance.

7.4 Eurocode Approach

As in BS 5628, Part 3[7.4], Eurocode 6[7.6] (EC6) assessment of fire resistance is made by providing tables giving minimum thicknesses of masonry for stated periods of fire resistance. The Eurocode[7.6] itself provides the models for these tables with no values, allowing each country to insert its own required values in the NAD. The UK NAD[7.6] provides tables for:

- Loadbearing single leaf walls, with a separating function complying with criteria REI (loadbearing (R), integrity (E), thermal insulation (I)). Table 2 of EC6-1-2[7.6] provides data for walls carrying loads up to a resistance of N_{Rd}/average γ_f, where N_{Rd} is the loadbearing capacity taking into account the effects of slenderness and eccentricity, and γ_f is the total characteristic load divided by total design load (see EC1-1[7.8]). Table 3 of EC6-1-2[7.6] provides data for the same wall types, but carrying load up to $0.6N_{Rd}$/average γ_f.
- Non-loadbearing single-leaf walls, separating function EI, with data contained in Table 4 of EC6-1-2[7.6].
- Loadbearing cavity walls with one leaf loaded, subject to load up to a resistance of N_{Rd}/average γ_f, separating function REI (Table 5 of EC6-1-2[7.6]).

Where neither leaf of a cavity wall carries a load, performance is equivalent to the sum of the fire resistances of the individual leaves, the NAD applying a maximum performance of 240 minutes.

An extract from Table 2 of EC6-1-2[7.6] is shown in Table 7.3, covering loadbearing single-leaf walls constructed using clay units. The table also contains data for calcium silicate, aerated concrete, aggregate concrete and manufactured stone units.

Table 7.3 Extract from Table 2 of Eurocode 6 Part -1.2[7.6] Loadbearing single-leaf walls (subject to load up to a resistance of N_{Rd}/average γ_f) (equivalent to EC6-1-2[7.6], Table C.1)

Material	Finish(Note)	Minimum masonry thickness (mm) for standard fire resistance (min) of:					
		30	60	90	120	180	240
Clay units conforming to EN 771-1 laid in general-purpose mortar							
Group 1 units $\rho \geq 1000$kg/m^3	(a)	100	100	100	170	200	200
	(b)	90	100	100	100	170	170
Group 1 solid units $\rho \geq 1200$kg/m^3	(a)	90	90	100	100	170	170
	(b)	90	90	90	100	140	170
Group 2a units $\rho \geq 700$kg/m^3	(a)	215	215	215	215	–	–
	(b)	215	215	215	215	215	215
Calcium silicate units conforming to EN 771-2, laid in general-purpose or thin-layer mortar							
Group 1 units $\rho \geq 1000$kg/m^3	(a)	90	100	100	190	200	–
	(b)	90	100	100	100	170	–
Group 1 solid units $\rho \geq 1600$kg/m^3	(a)	90	90	100	100	190	190
	(b)	90	90	90	100	140	190
Aerated concrete units conforming to EN 771-4, laid in general-purpose or thin-layer mortar							
Group 1 units $400 \leq \rho \leq 550$kg/m^3	(a)	100	100	120	125	150	150
	(b)	90	100	110	125	150	150
Group 1 units $\rho > 550$kg/m^3	(a)	90	90	100	100	140	150
	(b)	90	90	90	90	100	100
Aggregate concrete units conforming to EN 771-3 and manufactured stone units conforming to EN 771-5, laid in general-purpose, thin-layer or lightweight mortar							
Group 1 units $500 \leq \rho \leq 1500$kg/m^3	(a)	90	90	100	100	140	150
	(b)	90	90	90	90	100	100
Group 1 units $\rho > 1500$kg/m^3	(a)	90	90	90	100	140	150
	(b)	90	90	90	90	100	100
Group 2 units $500 \leq \rho \leq 800$kg/m^3	(a)	90	100	100	100	140	150
	(b)	90	90	90	100	140	140
Group 2 units $\rho > 800$kg/m^3	(a)	100	100	140	140	150	190
	(b)	90	100	100	140	140	150

Note: (a) for walls with no plaster finish and walls with a sand-cement/sand gypsum plaster finish
(b) for walls finished with vermiculite gypsum plaster (1½ : 1 to 2 : 1 by volume)

Data contained in design tables take account of recent research test data obtained by the material producers. Masonry materials and unit types are referenced in relation to their respective European EN product standards. The maximum slenderness ratio for vertically loaded walls should not exceed 27. The maximum height-to-thickness ratio for non-loadbearing walls should not exceed 40.

If the information from Table 7.3 (extract from Table 2 of EC6-1-2[7.6]) is used for the same example as before, for solid units with a gross dry density (ρ) that exceeds 1200kg/m^3 and 'a' type finish (no applied surface finish), we can see by inspection that a 100mm minimum thickness of clay masonry achieves a standard fire resistance of 120 minutes.

The loadbearing capability of the wall will need to be separately verified by calculation to EC6-1-1[7.9]. This comprises vertical load determination from a consideration of masonry strength and slenderness. In this case N_{Rd}, the design loadbearing capacity, divided by average γ_f, the partial safety factor, gives the maximum vertical load capacity of the wall. (See Clause 1.4(1) of EC6-1-2[7.6] together with Clauses 3, 5.3, 5.11 and 5.14.3 of the UK NAD).

Additional slenderness check: The slenderness ratio of the wall also needs to be checked. In this example it is, 25.9 (i.e. wall dimensions: 1.0 x 2650mm x 102.5mm) which is less than the 27 permitted.

7.5 Further Engineering methods

Although masonry constructions have been satisfactorily used traditionally for many years, more advanced design methods and computational techniques, using its properties at high temperature, have not been developed to predict performance. Only limited work has been carried out on the structural and thermal properties of masonry at high temperatures.

The draft Eurocode document[7.6] provides an introduction to alternative procedures for calculation of structural fire resistance. At this stage such procedures are not considered to have developed sufficiently to justify their codified use. The method has not yet been sufficiently calibrated over the whole European test database to ensure its reliability for design use across the whole of Europe.

Such procedures seem likely however to provide a future alternative basis for assessment of the fire resistance of masonry. Research is now being carried out to develop calculation-based methodologies in order to provide a credible alternative to the use of tabular data. Future application of the results of such research will offer a method for fire-resistance design of masonry which is less restrictive than current practice, and provides the potential for significant economies in design.

References.

7.1 *BS 476: Fire tests on building materials and structures*. British Standards Institution, London, various parts and dates.

7.2 *Building Regulations 1991. Approved Document B: fire safety*. London, The Stationery Office, 2000.

7.3 de Vekey, R C. *The fire performance of insulated cavity walls*. BRE Note 52/97. Building Research Establishment, Watford, 1997.

7.4 *BS 5628: Code of practice for use of masonry, Part 3 Material and components, design and workmanship*. British Standards Institution, London, 2001.

7.5 Morris, W A, Read, R E H and Cooke, G M E. *Guidelines for the construction of fire resisting structural elements*. Report BR128. Building Research Establishment, Watford, 1988.

7.6 DD ENV 1996-1-2: 1996. *Eurocode 6*: *Design of masonry structures – Part 1.2. General rules – Structural fire design*. British Standards Institution, London, 1997.

7.7 *BS 3921: Specification for clay bricks*. British Standards Institution, London, 1985.

7.8 DD ENV 1991-1:1996. *Eurocode 1: Basis of design and actions on structures: Part 1, basis of design*. British Standards Institution, London, 1996.

7.9 DD ENV 1996-1-1: 1996. *Eurocode 6: Design of masonry structures – Part 1.1. General rules for buildings – rules for reinforced and unreinforced masonry*. British Standards Institution, London, 1996.

8 Conclusion

This guide has given an introduction, in some detail, to the design of structures for fire. The content is aimed at architects, controlling authorities and structural engineers who want to widen their knowledge in this area. Significant value can be delivered by considering the fire safety aspects of the structure in the early stages of design so that the best compromise solution between the fire and normal load cases can be achieved.

Fire resistance design is just one aspect of fire safety design for any building. It is generally of importance in the post-flashover stage of a fire when construction materials may reach high temperatures. This report is specifically about the performance of structures and does not offer detailed guidance on fire safety management, means of escape or fire fighting although clearly all will be part of the fire strategy that would be developed for a building as a whole.

Traditional design is prescriptive and is based on fire resistance testing in a standard furnace. The design fire exposure is based on the standard temperature-time curve followed by the furnace test in ISO 834[8.1], BS 476[8.2] or ASTM E119[8.3]. There are many issues concerning the suitability of the furnace test to represent adequately the performance of real structures in real fires, but principally:

- Natural fires are very different in peak temperature and duration from the standard fire curve and are dependent on fuel load, ventilation, compartment shape and the thermal properties of the boundary wall materials.
- Single unrestrained elements such as columns, beams or slabs are tested and the results are used to determine the fire resistance of real structures.

As an alternative to testing there is a range of design methodologies ranging from simple calculations to more sophisticated analytical techniques incorporating natural fire data. The most complex design method includes modelling of whole building structures using finite element codes for heat transfer analysis and structural behaviour.

As a direct result of the shortcomings of the fire resistance test there is a great deal of scope for innovative design even at a simple level such as the load ratio concept for steel in BS 5950 Part 8[8.4].

Research into the behaviour of structures in fire has been ongoing for decades but in the last 10 years it has been thriving and substantial progress has been made especially in composite (steel and concrete) frame design. The fire community is beginning to understand the huge benefits that can be gained from considering structures as a full 3D assembly as opposed to a determinate structure heated in a furnace. The findings will enable substantial reduction in the use of passive fire protection materials and savings for the construction industry in terms of time spent on site. The new understanding is growing at a rapid rate because of the advances in computing capabilities. Structural fire design 10 years ago was probably where structural design was 100 years ago. This can only lead to more consistent safety levels, designs which are potentially more robust, and increased innovative design opportunities.

The research solutions are continuously feeding into design codes and guidance. Practitioners should therefore check regularly what the latest information is in their country.

Eurocodes are being produced for all aspects of design and will in due course replace British Standards in the UK. Eurocodes have considerable design content which has been derived directly from research. Simple analytical methods which may give savings when compared to strict prescriptive guidance are described in the Eurocodes. They also give scope for engineers to use advanced research based solutions such as finite element modelling of whole structures. The use of design fires and the time-equivalence approach is encouraged in the Eurocodes and should lead to more cost efficient and safer design.

This guidance should inspire and aid innovative building design involving all relevant parties in the design team.

The design of structures for fire will continue to develop at a considerable rate as research intensifies especially since the events of September 11th 2001.

Sections on further engineering methods throughout the document provide thought provoking references for interested parties who want to read beyond this guidance.

References

8.1 International Organization for Standardization. *ISO 834 Fire resistance tests. Elements of Building Construction*. ISO, Geneva, 1975.

8.2 *BS 476: Part 20: 1987 Fire tests on building materials and structures. Part 20: Method for determination of the fire resistance of elements of construction (general principles)*. British Standards Institution, London, 1987.

8.3 ASTM E119-95a *Standard Test Methods for Fire Tests of Building Construction and Materials*, 34th edn Volume 4. American Society for Testing and Materials. West Conshohocken, Pa., 1995.

8.4 *BS 5950: Structural use of steelwork in building Part 8: Code of practice for fire resistant design*. British Standards Institution, London, 1990.

A.1 Generic fire protection materials

Guidance on the fire performance and use of generic materials such as concrete, brickwork, blockwork, and certain types of plasterboard or gypsum plaster is presented in the Building Research Establishment publication, *Guidelines for the Construction of Fire Resisting Structural Element*[A.1].

A.2 Proprietary fire protection materials

Many forms of proprietary structural fire protection materials are available from manufacturers worldwide. Such materials should have been fire tested to the requirements of BS 476 Part 20[A.2] and BS 476 Part 21[A.3] or equivalent. The thickness of fire protection material required to satisfy a specific fire resistance period in the UK can be selected from authoritative material performance data sheets, published in *Fire protection for structural steel in buildings*[A.4]. This publication, which is referenced in Appendix A of *Approved Document B*[A.5], is commonly referred to as the 'Yellow Book'.

The types of fire protection systems available can be classified into three main product groups, i.e. sprays, boards and intumescent coatings.

A summary of the main factors affecting product selection in each of these groups is given in Table A.1.

A.3 Sprayed cementitious or gypsum based coatings

Cement or gypsum based materials containing mineral fibre, expanded vermiculite, expanded perlite and/or other lightweight aggregates or fillers, are generally the least expensive forms of fire protection. These coatings can provide up to 240 minutes fire resistance and usually are defined as being non-combustible in accordance with BS 476 Part 4[A.6].

Mineral fibre based materials are delivered dry to the spray head, where they are mixed with water and compressed air. Vermiculite or perlite sprays are usually premixed with water before being pumped to the spray head. Surfaces should be clean, and any primer should be compatible with the protection material. While most coatings are applied in situ (see Figure A.1), some can be used externally and may be applied before site assembly if suitable care is taken during subsequent handling.

The coatings appear textured and are often susceptible to mechanical damage. Some may require additional surface protection to prevent air erosion,

Fig A.1 Sprayed protection applied to complex beam detail © Cafco International

Fig A.2 Board fire protection applied to columns © Promat

when used, for example, behind plenum ceilings, or to meet cleanliness requirements. When mechanical retention is required, steel chicken wire, weld mesh or expanded metal lath (EML) is mechanically attached to the surface. Guidance on the use of sprayed cementitious or gypsum based coatings, including thickness measurement, is presented in BS 8202 Part 1[A.7].

A.4 Boards and blankets

Blankets, semi-rigid and rigid boards are used as dry forms of fire protection installed in situ as either profile or boxed protection (see Figure A.2). Base materials include ceramic fibres, calcium silicate,

Table A.1 Characteristics of fire protection systems

	Sprayed cementitious or gypsum based coatings	Boards and blankets	Intumescent coatings
Relative cost	Low to medium	Low to high	Medium to high
Wet or dry	Wet	Mainly dry	Wet, thick film may be preformed
Application	Messy, with protection required to adjacent surfaces	Relatively clean, labour intensive	If applied on site protection required to adjacent surfaces, otherwise use off-site
Tools required for application	Specialist equipment required	Simple application tools	Application by painting equipment for thin films. Thick film requires specialist equipment
Internal/ external use	Internal and external materials available. Not all suitable for external use.	Internal use. Additional protection required for external use	Internal with some external systems
Preparation	No primer required for internal use, but surfaces to be clean and compatible	Contact surfaces for noggins, etc. to be clean and compatible	Compatible primer required on cleaned surfaces
Robustness	Relatively brittle and may be vulnerable to mechanical damage. Some coatings unsuitable for use behind plenum ceilings or in clean areas	Some rigid boards relatively brittle and may be vulnerable to mechanical damage. Batts and blankets may require additional covering	Similar to that of paint systems. Thick film very tough and durable.
Finish	Textured finish	Variable: boards mainly smooth with joints visible unless a wet finishing coat is applied; batts/blankets are textured with fixings visible	Smooth or slightly textured surface. A coloured decorative finish can be applied
Mechanical retention	Necessary where no re-entrant angles available or thickness is high	Normally requires some mechanical retention	Mesh retention required at higher thicknesses
Thickness range	10 to 75mm	Multiple layers used. Boards 6 to 100mm; batts/blankets 12 to 76mm	Thin film 0.3 to 6.5mm Thick film 2.0 to 32mm
Maximum fire resistance	240 mins	240 mins	Thin film 120 mins Thick film 240 mins
Class O surface[Note 1]	Yes	Usually	Possibly

Note 1: Class 0 is the highest product performance classification for wall and ceiling linings. It is not identified in any British Standard Test. Class 0 is met if any material or surface of a composite product are composed throughout of materials of limited combustibility; or a Class 1 material has a fire propagation index (I) of not more than 12 and sub-index (I_1) of not more than 6. The rating Class 0 limits fire spread over and energy released from linings.

rock fibre, gypsum and vermiculite. Most are only suitable for interior use or limited external exposure during construction. However a few board systems can be subjected to full external exposure.

Up to 240 minutes fire resistance can be provided and many materials are defined as being non-combustible and therefore meet the requirements of Class 0.

Calcium silicate and vermiculite boards are hard and smooth in appearance but also vulnerable to impact damage. Mineral fibre boards are softer to the touch while the blanket materials are fully flexible. Potential problems associated with loose fibres in the latter products may be minimized by an outer sheathing of aluminium foil or similar (see Figure A.3), and by the use of taped joints. Visual appearance will vary with the system chosen.

Flexible blanket materials are typically fixed with steel weld pins (see Figure A.4) and non-return washers, wire ties and chicken wire. Rigid boards may be retained by a variety of methods such as with pins, nails, special spiral screws or sometimes a bonding agent to a timber sub-frame. When noggins are friction fixed, contact surfaces may need to be clean and unpainted. As an alternative to a timber sub-frame, lightweight galvanised mild steel internal framing members may be used with the plasterboard and calcium silicate board encasement systems.

Longer fire resistance periods often require the use of multiple layers of boards. In this case the joints in the layers are staggered. Where only a single layer of board is required, joints are normally backed by noggins or a fillet of the same board material.

A.5 Intumescent coatings

Intumescent coating systems are classified as either thin film, which account for the vast majority of systems used in general construction, or thick film, sometimes referred to as mastics. The materials are reactive, swelling to many times their original thickness when exposed to fire, with the resultant char insulating the underlying steel substrate (see Figure A.5).

Thin film intumescent coating systems are similar in appearance to conventional paints and are applied either by airless spray, brush or roller (see Figure A.6). They may be solvent based, or water borne, and usually include a compatible primer(s), the intumescent coat(s) and a top coat or sealer coat (often available in a wide range of colours). Surfaces must be thoroughly cleaned before the paint is applied.

Fig A.3 Foil covered fibre board applied to beams © Rockwool Ltd

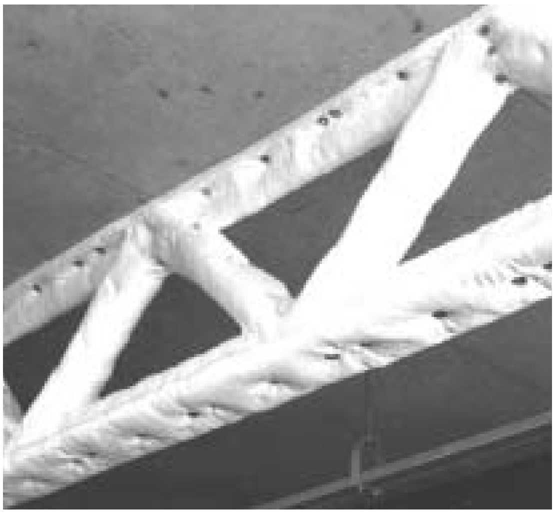


Fig A.4 Pin-fixed blanket applied to truss © Unifrax Corporation

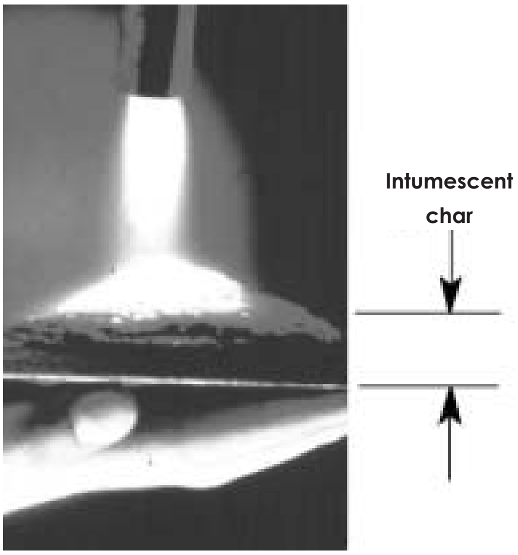


Fig A.5 Demonstration of intumescence

Most coatings are applied in situ and are suitable for interior or limited exterior exposure during construction. However, a few coatings may be used externally and/or applied off site, if appropriate care is taken during subsequent handling. Off-site application is increasing because it has the advantage of removing fire protection from what is sometimes the critical path in the construction programme.

It is common when considering off-site application to use simple fire engineering methods to ensure that the required protection can be achieved using only a single coat of intumescent. This limits the drying time and hence the throughput of any paint facility. Increasing the weight of steel to reduce the number of coats will often be economic. The use of the load ratio/limiting temperature method is very important in achieving this goal and software will soon be available to assist in this process.

Thin film intumescent coatings can provide up to 120 minutes fire resistance. General guidance on the selection and use of intumescent coatings can be found in BS 8202 Part 2[A.8]. Guidance on the measurement of coating thickness can be found in an ASFP document *On Site Measurement of Intumescent Coatings* [A.9].

Fig A.6 Thin film intumescent coating being applied

© Leigh Paints

References

A.1 Morris, W A, Read, R E H, and Cooke, G M E. *Guidelines for the construction of fire-resisting structural elements.* Report BR128. Building Research Establishment, Watford, 1988.

A.2 *BS 476 Part 20:1987. Fire tests on building materials and structures. Part 20: Method for the determination of the fire resistance of elements of construction (general principles).* British Standards Institution, London, 1987.

A.3 *BS 476 Part 21:1987. Fire tests on building materials and structures. Part 21: Method for the determination of the fire resistance of loadbearing elements of construction.* British Standards Institution, London, 1987.

A.4 Association for Specialist Fire Protection. *Fire protection for structural steel in buildings*, 3rd edn. SCI Pubn 13. Steel Construction Institute, Ascot, 2002.

A.5 *Building Regulations 1991. Approved Document B: fire safety*. London, The Stationery Office, 2000.

A.6 *BS 476 Part 4:1970 (1984). Non-combustibility test for materials*. British Standards Institution, London, 1984.

A.7 *BS 8202: Part 1:1995. Coatings for fire protection of building elements. Part 1: Code of practice for the selection and installation of sprayed mineral coatings*. British Standards Institution, London, 1995.

A.8 *BS 8202: Part 2: 1992. Coatings for fire protection of building elements. Part 2: Code of practice for the use of intumescent coating systems to metallic substrates for providing fire resistance*. British Standards Institution, London, 1992.

A.9 Association for Specialist Fire Protection. *On Site measurement of intumescent coatings: Part 1 Technical guidance note for measurement of dry film thicknesses (dft's) for intumescent coatings*. ASFP TGN 003: Part l. ASFP, Farnham, 1996.

Introduction to the fire safety engineering of structures

The design of building structures for fire is developing at a significant pace in line with fire safety engineering as a whole. It is certainly starting to have a growing impact on the way structures are designed, procured and specified in many countries throughout the world. New procedures, advanced analytical methods and improved risk assessment techniques are now available to the experienced engineer to support performance-based design for the fire load case. However this knowledge tends to be in the hands of a few specialists and consequently the Institution of Structural Engineers has identified the need for guidance at a level that will be of value to a wide range of construction professionals.

This guidance has been prepared to provide the engineer, the architect, the regulatory authorities and other construction industry professionals with the inspiration to develop safer and better value solutions for the performance of building structures during fire. There is a considerable opportunity for the engineer and the architect to work together to develop improved designs based on new and developing technologies within a sensible regulatory framework.

This document should be of benefit to:

- architects looking for better solutions
- controlling authorities wishing to ask the right questions
- engineers seeking to develop new skills and approaches as technology develops in fire safety engineering
- contractors, manufacturers and suppliers who want to appreciate the broader approach being adopted with a view to adapting their products and future development.

The Institution of Structural Engineers
11 Upper Belgrave Street, London SW1X 8BH, United Kingdom
T: +44 (0) 20 7235 4535
F: +44 (0) 20 7235 4294
E: mail@istructe.org.uk
W: www.istructe.org.uk

IStructE