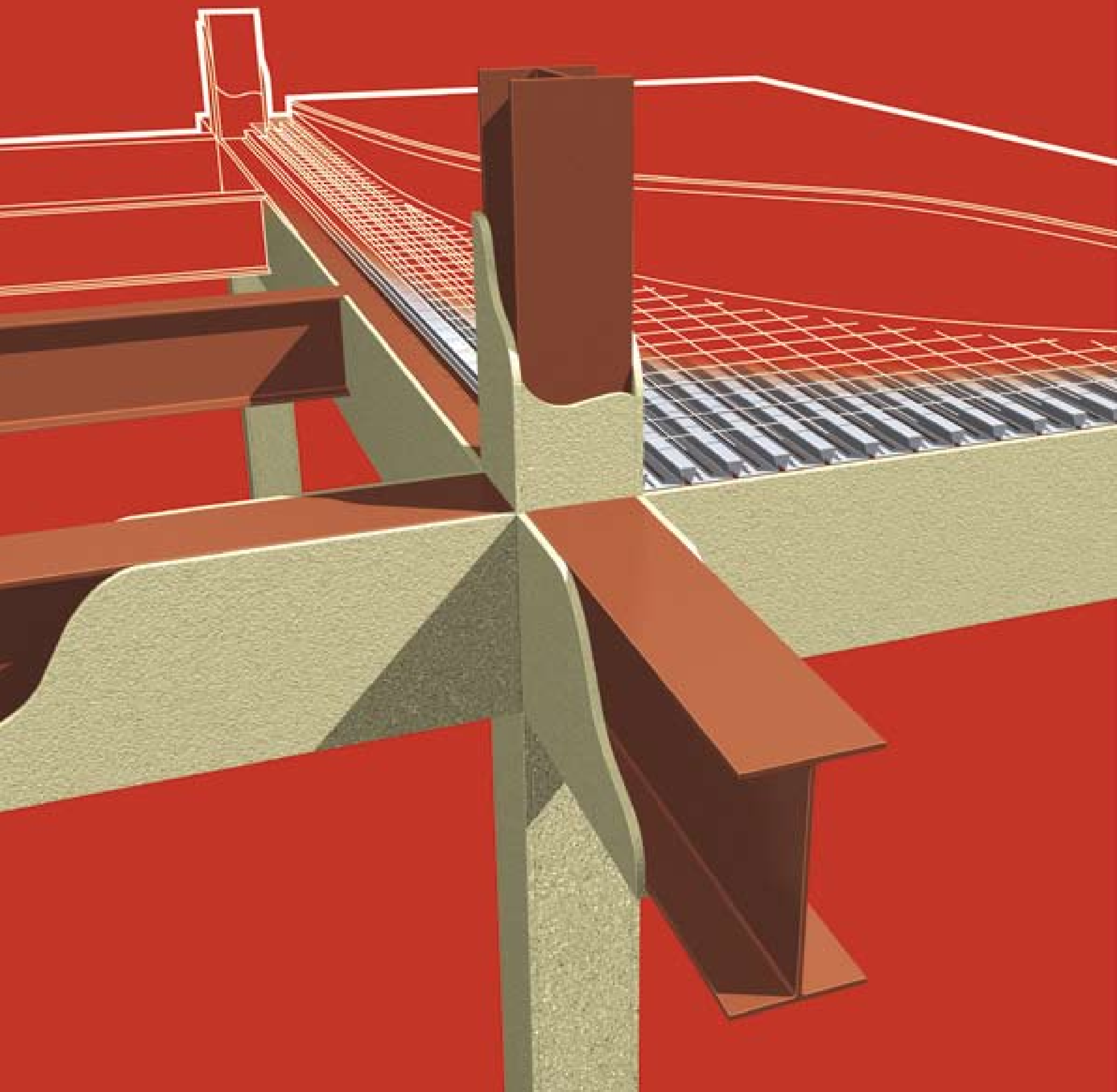
Corus Construction & Industrial

Fire resistance of steel-framed buildings

2006 edition



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Foreword

The Approved Document approach to satisfying regulatory requirements in England and Wales in the mid 1980s began a recognition of modern practice that continued into the ’90s with the introduction of the structural codes for fire resistant design embodied in BS5950 Part 8, and the draft Eurocodes 1991-1-2, 1993-1-2 and 1994-1-2. This has further developed with the publication of BS7974, the Code of Practice for Application of Fire Safety Engineering Principles to the Design of Buildings (page 30).

Even the basic shape of structural sections, substantially unchanged for over 100 years, is now being enhanced with a shape specially developed for optimum performance in fire in the form of the asymmetric beam (page 23). The pace of change will continue through this decade as increasingly sophisticated methods are developed to allow design for fire to move away from consideration only of simple elements towards whole building behaviour in fire (pages 32-34).

This publication is a guide to the latest thinking in the field of fire safety. It is concerned primarily with solutions to structural fire resistance issues in steel-framed buildings. It will be updated frequently to ensure its relevance as a source of information on the fire resistance of buildings.

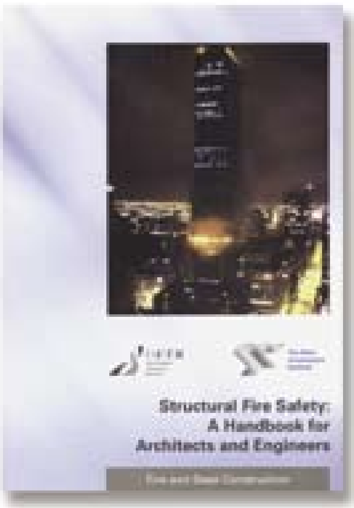


Figure 1
This brochure may be used in conjunction with The Steel Construction Institute publication: Structural Fire Safety: A Handbook for Architects and Engineers [1].

1. The Building Regulations and structural fire resistance

England and Wales

Provision for structural fire resistance of buildings is embodied in Part B of Schedule 1 of the Building Regulations 2000 as follows:

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

Approved Document B[2] (Figure 2) interprets the requirements of the Building Regulations and states that the stability criterion will be satisfied if *"the load bearing elements of the structure of the building are capable of withstanding the effects of fire for an appropriate period without loss of stability"*.

The Approved Document contains detailed provisions for the maintenance of structural stability in fire. These are intended to provide guidance for some of the most common building situations.

Figure 2 Approved Document B to the Building Regulations for England and Wales, 2000.

Table 1 – Fire resistance in minutes

England and Wales recommendations 2000	Height of top storey – metres			
	<5	**<18**	**<30**	**>30**
Approximate no. of storeys	2	5/6	8/9	9+
Residential (non domestic)	30	60	90	120
Offices	30	60 *	90 *	120 plus sprinklers
Shops, Commercial Assembly	60 *	60	90 *	
Industrial and Storage	60 *	90 *	120 *	
Car parks – closed	30	60	90	
Car parks – open-sided	15	15	15	60

* Reduced by 30 minutes when sprinklers are installed.

Table 1 Summary of structural fire resistance requirements from Approved Document B.

Guidance on 'appropriate periods' for different building occupancies is given in Table A2 of the Approved Document (summarised in Table 1). However these fire resistance periods are not mandatory. The Approved Document states that:

"There is no obligation to adopt any particular solution contained in an Approved Document if you prefer to meet the relevant requirement in some other way".

The Approved Document goes on to suggest 'other means' to demonstrate compliance by stating that:

"Fire safety engineering can provide an alternative approach to fire safety. It may be the only practical way to achieve a satisfactory standard of fire safety in some large and complex buildings and in buildings containing different uses"
(see pages 30 to 34).

The most important aspects of the Approved Document concerning structural fire resistance are:

- Fire resistance periods are based on building height and occupancy.
- The height of a building, for the purpose of determining fire resistance, is measured from the ground to the floor of its uppermost storey. The top storey is not included (Figure 3).
- A reduction of 30 minutes in the required fire resistance may be applied to most types of non-domestic occupancies less than 30 metres in height when an approved sprinkler* system is installed.
- The maximum fire resistance period for superstructures and basements is 120 minutes.
- Compartment sizes can be doubled in many instances where sprinklers* are installed.
- All non-residential buildings over 30 metres in height must now be equipped with sprinklers*.
- Structural elements of open deck car parks require only 15 minutes fire resistance. The majority of Universal steel sections will survive a 15 minute standard fire test and thus most steel framed open deck car parks do not require structural fire protection. Full details are given in the Corus publication, Steel Framed Car Parks[3].

** Sprinklers mean an automatic sprinkler system meeting the relevant recommendations of BS EN 12845: 2004, with additional requirements for life safety.*

Height of top storey

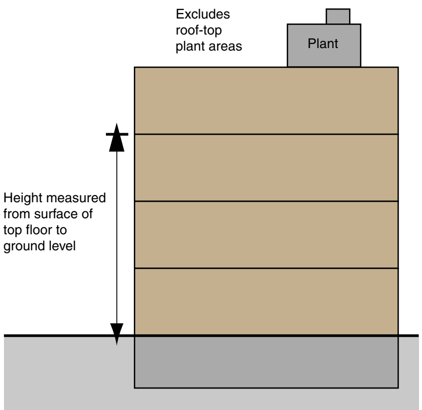


Figure 3 Definition of building height as measured in Approved Document B.

Figure 4 Steel in open deck car parks is usually unprotected.

Scotland

The Scottish Building Regulations underwent a fundamental change in 2004 following the introduction of the Building (Scotland) Act 2003. In 2005, the existing Technical Standards were withdrawn and replaced by two Technical Booklets covering domestic buildings and non-domestic buildings. (Note: at the time of writing, proposed changes to Approved Document B, due in its next edition in 2007, indicate that England and Wales may introduce a similar distinction.) Both handbooks are available on the Building Standards Agency Scottish website[4] (Figure 5).

The Technical Booklets give guidance on achieving the standards set out in the Building Regulations. The standards are in the form of expanded functional requirements, i.e. they describe the functions the buildings should perform, such as *"providing resistance to the spread of fire."*

The regulations are mandatory, but the choice of how to comply lies with the building owner. The Technical Booklets have been issued for the purposes of providing practical guidance on this.

If the guidance is followed in full, it will be accepted that compliance with the Building Regulations has been achieved. Proof of compliance with the guidance may be relied on in any proceedings as tending to negate liability for any alleged contravention of the Building Regulations.

It is acceptable to use alternative methods of compliance provided that they fully satisfy the regulations. Where alternative solutions are put forward however, it is necessary to have regard to the details of the guidance. Where performance standards or policy statements are given, every part of the solution is expected to meet them.

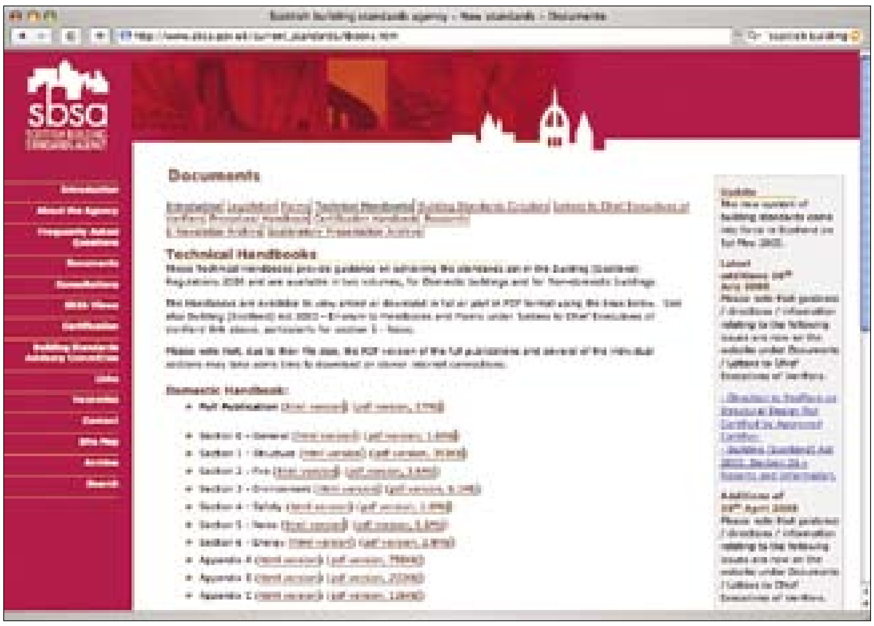


Figure 5 Building Standards Agency Scottish website www.sbsa.gov.uk.

Typical of the type of structure which has been designed using an alternative method, in this case a fire engineering approach, is the stands at Glasgow Celtic Football Club in Parkhead (Figure 6).

Figure 6 New stand Glasgow Celtic football stadium, Parkhead, Glasgow.

Some important aspects of the Technical Booklet concerning structural fire resistance are:

- Fire resistance requirements are based on a mixture of building height, occupancy, and floor area. Fire resistance is given as short, medium or long, equating to 30, 60 and 120 minutes.
- Structural elements of open deck height car parks less than 18m in height require only 15 minutes fire resistance. (The majority of universal steel sections have 15 minutes inherent fire resistance and thus most steel framed open deck car parks do not now require structural fire protection).

Northern Ireland

In Northern Ireland new Building Regulations came into force in November 1994. The fire safety requirements for these regulations are supported by Technical Booklet E[5] (Figure 7) which contains provisions regarding structural fire resistance, compartmentation etc. similar to those in the Approved Document for England and Wales.

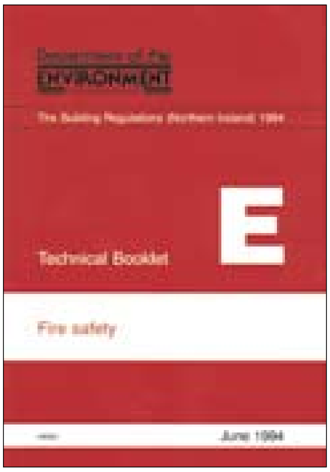


Figure 7 Technical Booklet E to the Northern Ireland Building Regulations 1994.

Unlike the provisions of the Approved Document, which are for guidance, the provisions of Technical Booklet E are deemed to satisfy the requirements of the Building Regulations. Where the provisions of the Technical Booklet are not followed, then the onus falls on the designer to show that the requirements of the regulations can be met by other means.

Other sources of information

DD9999[6] is a Draft for Development published by the British Standards Institution. The intention behind the development of the document is to provide a more transparent and flexible approach to fire safe design through the use of a structured approach to risk based design.
It is intended that the Code will eventually succeed a number of existing British Standards, including the BS5588 series which form the basis of most existing fire precautions.

DD9999 contains structural fire resistance requirements based on a risk approach and the parametric time-temperature curve (see page 21). These can be higher, lower or the same as those found in Approved Document B. In general, considerable credit is given for the presence of sprinkler protection so the requirements for high rise building, where sprinklers are usually mandatory, are often lower than those in Approved Document B.

It is intended that DD9999 will become a full British Standard in 2007.

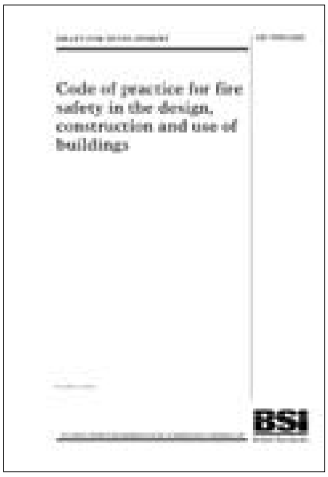


Figure 8 DD9999 Draft for Development.

Buildings located within the inner London area are subject to the requirements of the London Building Act 1939. Within this act, precautions against fire in buildings are covered by Section 20. This ensures that *"proper arrangements will be made and maintained for lessening so far as is reasonably practicable danger from fire in buildings."*

In 1990 the London District Surveyors Association published Fire Safety Guide, No. 1: Fire Safety in Section 20 Buildings[7] (Figure 9). This document contains detailed information on fire resistance requirements for high risk buildings within the inner London area. The main differences with regard to structural fire resistance are that basement car park requirements are more onerous than those in Approved Document B. Also, mandatory sprinklers are introduced in high rise, non-residential buildings above 25 metres.

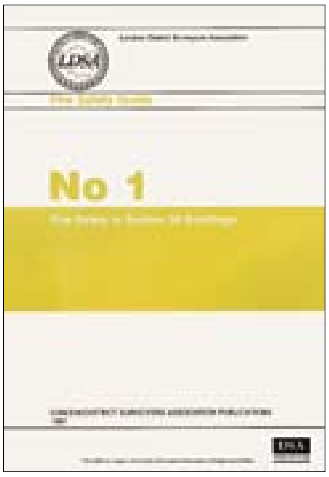


Figure 9 Fire Safety Guide No. 1: Fire Safety in Section 20 Buildings.

2. Sprinklers

Sprinklers are designed to suppress automatically small fires on, or shortly after, ignition or to contain fires until the arrival of the fire service.

In Europe most sprinklers work on the exploding bulb principle. The water nozzle is sealed by a glass bulb containing a volatile liquid. When heated by the fire, the liquid expands and breaks the bulb thus activating the sprinkler head (Figures 10 and 11). As only individual sprinkler heads affected by the hot gases from the fire are activated, water damage is minimised.

In Approved Document B to the Building Regulations for England and Wales, a reduction of 30 minutes in the required fire resistance may be applied to most types of non-domestic occupancies less than 30 metres in height when an approved life safety sprinkler system is installed (see page 4).

All non-domestic buildings over 30 metres in height are now required to have sprinklers, as do shopping centres. This trade-off between passive and active systems has given an impetus to their use in England and Wales; it is widely seen to be a positive development since statistical experience shows that the use of sprinklers provides a significant improvement in life safety, and also has considerable social and economic benefits (Figure 12).

The major cause of fatalities in fire is smoke and most deaths occur long before there is any significant risk of structural collapse. In addition, the major costs of fire typically result from destruction of building contents, finishes and cladding and from the consequential losses. Structural damage is normally of secondary importance. By suppressing fire and smoke, sprinklers are an extremely effective means of enhancing life safety and reducing financial losses.

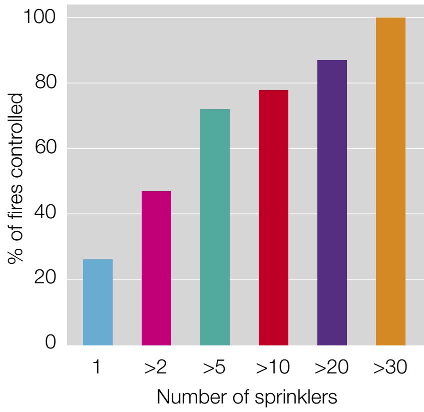


Figure 12 It is estimated by the Fire Protection Association that up to 76.5% of fires are controlled with five sprinkler heads or less.

More information on the benefits of sprinklers, both in terms of life safety and property protection can be obtained from the British Automatic Sprinkler Association (BASA)(8) (Figure 13). This publication contains detailed cost examples which describe the value of trade-offs in passive fire protection. Larger allowable compartment sizes, reduced number of fire fighting lifts and shafts etc. can, in some instances, cancel out any additional costs incurred in installing sprinklers.

Figure 10 Typical sprinkler head configuration. The red colour of the volatile liquid indicates that the glass will break at 68°C. This is the most common activation temperature.

Figure 11 Sprinkler head exploding. Courtesy of Wormald Ltd.

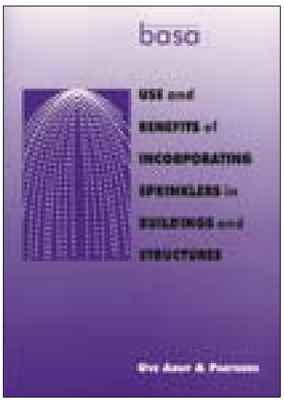


Figure 13 BASA sprinkler publication: Use and Benefits of Incorporating Sprinklers in Buildings and Structures.

3. Section factor and protection thickness assessment

Effect of section dimensions

Fire resistance is expressed in units of time so one of the contributory factors to fire resistance is the heating rate of the member. This governs the time taken to reach it's failure (or limiting) temperature and varies according to the dimensions of the section. Clearly, a heavy, massive section will heat up more slowly (and thus have a higher fire resistance) than will a light, slender section. This massivity effect is quantified in the 'Section Factor' (Hp/A)* Concept (Figure 14).

$$\textbf{Section Factor} = \frac{\textbf{Heat Perimeter (Hp)}}{\textbf{Cross-Sectional Area (A)}}$$

An example of this concept is given in Figure 15 which shows the heating rate for three unprotected beams when subjected to the standard fire test (see page 16).

Because heavy sections (lower Hp/A) heat up more slowly than light sections (higher Hp/A), a heavy section will require less insulation than a light section to achieve the same fire resistance.

Beams supporting non-composite concrete floor slabs with section factors less than $90m^{-1}$ heat so slowly that, where the load ratio (see page 17) is less than 0.6, they do not reach their limiting temperature for over 30 minutes, thus achieving 1/2 hour fire resistance without any fire protection. Columns in simple construction achieve 30 minutes fire resistance under the same circumstances when the section factor is less than $50m^{-1}$.

** sometimes written as A/V (Area/Volume)*

Hp/A concept

The heating rate of a steel section in a fire depends upon:

a) The perimeter of the steel exposed to flames - Hp (m)

b) The cross sectional area of the section - A (m^2)

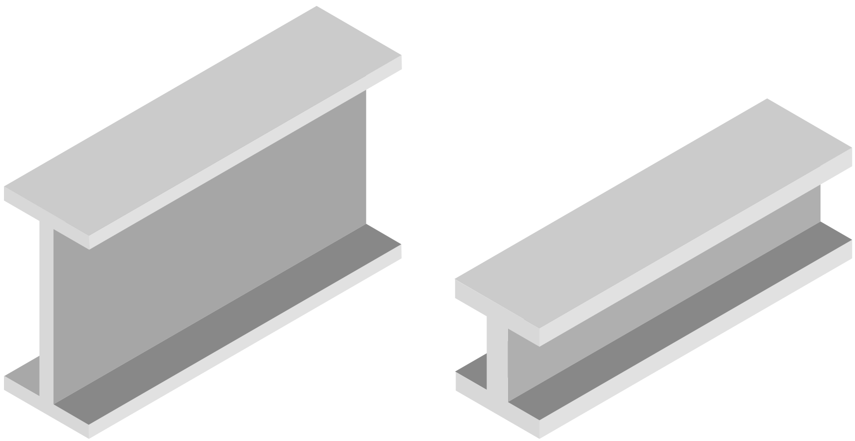


Figure 14 The section factor concept.

Unprotected beams - Design temperature

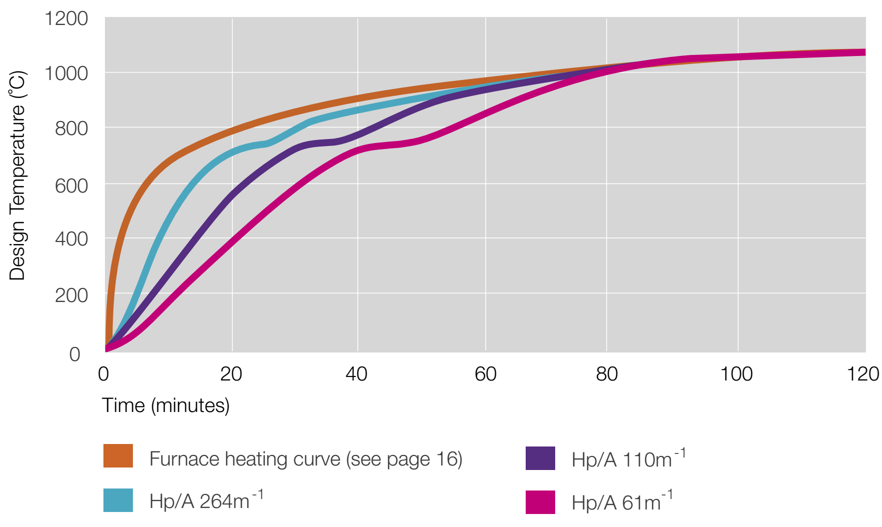


Figure 15 Heating rate curves for three different size beams in the standard fire test.

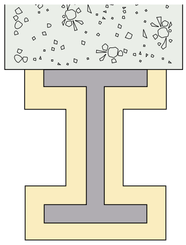
Hot rolled H and I sections

When proprietary passive fire protection is necessary to achieve fire resistance, the required thickness can be determined from manufacturer's published data. Much of this information has been consolidated into a reference text commonly known as 'The Yellow Book'[9] (Figure 16) published by the Association of Specialist Fire Protection (ASFP) and The Steel Construction Institute. This publication is easy to use and gives valuable guidance on approved proprietary fire protection systems.

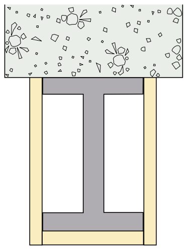
Manufacturer's recommendations generally relate the thickness of protection to the section factor (Hp/A) and the fire resistance time required. In general, protection thickness recommendations are derived from the BS476 Standard Fire Test (see page 16) and are designed to restrict steelwork in fire to a limiting temperature of 550°C (or 620°C for intumescent coated, 3 side exposed beams). However, where manufacturer's data for other limiting temperatures is available, it may be used and could yield economies.

Figure 16 'The Yellow Book'.

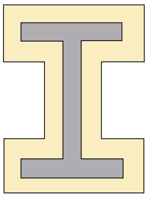
Figure 17 The four most common protection configurations for calculation of Hp/A.



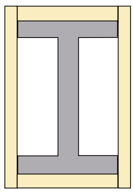
3-sided profile protection
Hp/A=160m^{-1}



3-sided box protection
Hp/A=120m^{-1}



4-sided profile protection
Hp/A=180m^{-1}



4-sided box protection
Hp/A=140m^{-1}

For typical building construction using universal I and H sections, the value of Hp/A is usually in the range 20-325m^{-1}, the value of 20m^{-1} being associated with the heavy 356 x 406 x 634 kg/m column for three sided box protection (e.g. boards), whilst the light 127 x 76 x 13 beam has a Hp/A value of 325 for four sided profile protection (e.g. intumescent coatings). In published tables, values of Hp/A are normally rounded to the nearest 5 units.

Figure 17 shows four protection configurations for a 533 x 210 x 82 kg/m beam. To determine the thickness of a spray protection for a three sided profile to give 1 hour fire resistance, first define the section factor – 160m^{-1} – then refer to manufacturer's data or 'The Yellow Book', which shows the required thickness to be 16 mm (Figure 18).

This procedure provides a relatively simple method for establishing the protection requirements for most sizes of steel section and fire resistance periods.

	Dry thickness in mm to provide fire resistance of up to:					
Hp/A	**0.5hr**	**1hr**	**1.5hr**	**2hr**	**3hr**	**4hr**
3	10	10	14	18	26	35
50	10	12	17	22	33	43
70	10	13	19	25	37	48
90	10	14	21	27	39	52
110	10	15	22	28	41	54
130	10	16	22	29	42	56
150	10	16	23	30	44	57
170	10	16	23	30	44	57

Figure 18 Extract from 'The Yellow Book' as it applies to a typical spray fire protection material.

Castellated and cellular beams

Figure 19 Cellular beams used at Lincoln University.

For castellated or cellular beams, or fabricated beams with holes, the thickness of the fire protection material should be 20% more than the thickness determined from the section factor of the original, uncut section for boards and sprays. Therefore an 800 x 210 x 82 kg/m castellated beam formed from the 533 x 210 x 82 kg/m section used in the previous example would require 1.2 x 16 = 19.2 mm, (rounded up to 20 mm), protection thickness.

The 20% rule is not suitable for use with intumescents and recent testing has indicated that the amount of added protection is product specific. The advice of the intumescent manufacturer should be sought.

Hot rolled unfilled hollow sections

Figure 20 Intumescent coated hollow sections. Courtesy of Carboline Ltd.

For unfilled hollow sections, the required thickness of fire protection is also determined from values of section factor. For board and spray fire protection materials, the thickness required for an unfilled hollow section may be obtained by reference to the thickness required for an I or H section with the same section factor.

Where the thickness of a board or spray fire protection material was originally assessed from tests using boxed systems which enclose the section, the same protection thickness can be used.

Where the thickness of a board or spray fire protection material was originally assessed from tests using sprayed systems, a modified thickness must be used. The modification factor is calculated as:-

For a section factor, $H_p/A < 250 m^{-1}$
Thickness = $t(1 + (H_p/A)/1000)$.

For a section factor, $H_p/A > 250 m^{-1}$
Thickness = $1.25t$

Where t is the thickness of fire protection material calculated for the equivalent I or H section.

This method is not applicable to intumescent coating systems. In this situation, confirmation must be sought from the manufacturers regarding required thicknesses. Most suppliers clearly differentiate between open (H & I) and closed (hollow) sections in their specifications.

Concrete filled hollow sections are discussed on page 25.

Traditional fire protection materials

For fire protection using concrete, blockwork and plasterboard, the best source of information on material thickness for specific fire resistance times is Guidelines for the Construction of Fire Resisting Structural Elements[10] (Figure 21).

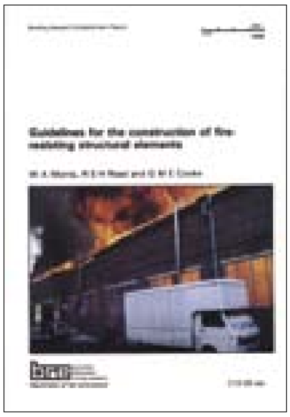


Figure 21 Guidelines for the Fire Protection of Fire-resisting Structural Elements.

4. Site applied protection materials

Passive fire protection materials

Passive fire protection materials insulate steel structures from the effects of the high temperatures that may be generated in fire. They can be divided into two types, non-reactive, of which the most common types are boards and sprays, and reactive, of which intumescent coatings are the best example.

Boards

Board systems (Figures 22 and 23) are one of the most popular type of fire protection in the UK. They are widely used both where the protection system is in full view and where it is hidden.

The principal advantages are:
Appearance - rigid boards offer a clean, boxed appearance which may be pre-finished or suitable for further decoration. The specifier should be aware, however, that cheaper board systems are available where appearance is not important.
Fixing - application is dry and may not have significant effects on other trades.
Quality assured - boards are factory manufactured thus thicknesses can be guaranteed.
Surface preparation - boards can be applied on unpainted steelwork.

The principal disadvantages are:
Cost - a non-decorative board system can be relatively cheap however a decorative system can significantly increase costs.
Application - fitting around complex details may be difficult.
Speed - board systems may be slower to apply than some other methods.

Figure 22 Board protection systems. Courtesy of Promat Ltd.

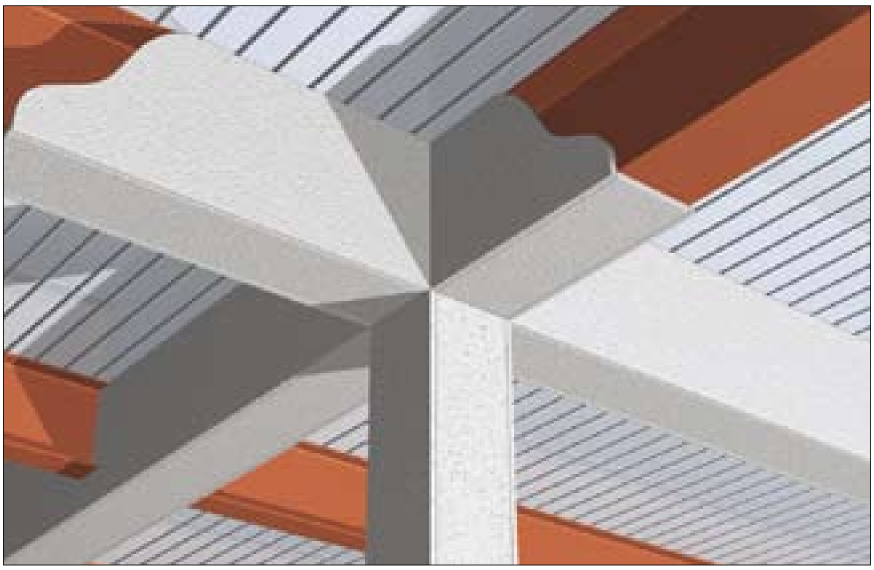


Figure 23 Fibre board applied to beams.

Figure 24 Spray protection system.

Sprays

Spray protection systems (Figure 24) have decreased in popularity in the past decade, despite being one of the cheapest forms of fire protection in terms of application costs.

The principal advantages are:
Cost - spray protection can usually be applied for less than the cost of the cheapest board. Because the cost of sprayed material is low compared to that of getting labour and equipment on site, costs do not increase in proportion to fire resistance times.
Application - it is easy to cover complex details.
Durability - some materials may be used externally.
Surface preparation - some materials may be applied on unprimed steelwork.

The principal disadvantages are:
Appearance - sprays are not visually appealing and so are usually used only where they are not visible.
Overspraying - masking or shielding of the application area is usually required on-site.
Application - is a wet trade, this can have significant knock on effects on the construction program with the result that the real cost of spray protection may be higher than that assumed using the application costs only.

Thin film intumescent coatings

Intumescent coatings (Figure 25) are paint like substances which are inert at low temperatures but which provide insulation by swelling to provide a charred layer of low conductivity materials at temperatures of approximately 200-250°C. At these temperatures the properties of steel will not be affected.

The principal advantages are:
Aesthetics - the thin coating allows the shape of the underlying steel to be expressed.
Finish - attractive, decorative finishes are possible.
Application - complex details are easily covered.
Servicing - post-protection fixing is simplified.

The principal disadvantages are:
Cost - typical application costs are higher than sprays although costs have decreased in recent years.
Application - is a wet trade which requires suitable atmospheric conditions during application and precautions against overspray.
Limited Fire Resistance Periods - Most intumescent coatings can traditionally provide up to 60 minutes fire resistance economically. Improvements in technology in recent years have reduced coating thicknesses considerably and intumescents are increasingly competitive in the 90 minute market also. A limited number of intumescent coatings can achieve 120 minutes fire resistance.

Over the past decade intumescent coatings have come to dominate the passive fire protection market in the UK.

Figure 25 The British Pavilion at the Seville Expo. Structural fire protection with thin film intumescent coating. Courtesy of Leigh's Paints.

Flexible/Blanket systems

Flexible fire protection systems (Figure 26) have been developed as a response to the need for a cheap alternative to sprays but without the adverse effects on the construction program often associated with wet application.

The principal advantages are:
Low Cost - blanket systems are comparable with cheap boards.
Fixing - application is dry and may not have significant effects on other trades.

The principal disadvantage is:
Appearance - unlikely to be used where the steel is visible.

Concrete encasement and other traditional systems

Until the late 1970s concrete was by far the most common form of fire protection for structural steelwork (Figure 27). However the introduction of lightweight, proprietary systems such as boards, sprays and intumescents has seen a dramatic reduction in its use. At present concrete encasement has only a small percentage of the fire protection market with other traditional methods such as blockwork encasement also used occasionally.

The principal advantage of concrete and blockwork is:-
Durability - these robust encasement methods tend to be used where resistance to impact damage, abrasion and weather exposure are important e.g. warehouses, underground car parks and external structures.

The principal disadvantages are:-
Cost - concrete encasement is normally one of the most expensive forms of fire protection.
Speed - time consuming on-site.
Space Utilisation - large protection thicknesses take up valuable space around columns.
Weight - building weight can increase considerably.

Information on thickness of concrete encasement for specific periods of fire resistance can be found in Guidelines for the Construction of Fire Resisting Structural Elements[10].

Figure 26 Flexible blanket protection system.

Figure 27 Concrete encasement.

5. Off-site fire protection

Thin film intumescent coatings

Intumescent coatings are described on page 13. Of the available fire protection materials, it is these which are best suited to large scale off-site application. The coating is applied manually, generally in large heated sheds with good air movement provided by large fans.

Off-site fire protection using intumescent coatings has a number of distinct advantages:

- Reduced construction time: fire protection is often on the critical path of the construction program. Off-site application removes it from this position with significant benefit in terms of increased speed of construction. This was demonstrated in a study by The Steel Construction Institute(11).
- Reduced overall construction cost.
- Simplified installation of services.
- Application is carried out under carefully supervised conditions and so high standards of finish, quality and reliability are achievable.
- The number of on-site activities is reduced.
- Site access and weather related problems are eliminated.
- The need to segregate areas of the building for site application no longer becomes an issue.

A document (Figure 29) to facilitate the specification, application and general use of off-site applied intumescent coatings, has been prepared in two parts containing general guidance and a model specification. This is available from The Steel Construction Institute(12).

At the time of writing, off-site application is thought to have captured 15% of the total fire protection market in steel multi-storey new build in the UK.

Figure 29 Design Guidance and Model Specifications for use with Off-Site Applied Thin Film Intumescent Coatings (2nd edition).

Figure 28 Manual application of off-site intumescent coatings.

6. Steelwork fire resistance

Fire resistance is usually expressed in terms of compliance with a test regime outlined in BS476 Part 20 and 21[13]. It is a measure of the time taken before an element of construction exceeds specified limits for load carrying capacity, insulation and integrity. These limits are clearly defined in the standard. The characteristics of the time-temperature relationship for the test fire from BS476 are shown in Figure 30.

All materials become weaker when they get hot. The strength of steel at high temperature has been defined in great detail and it is known that at a temperature of 550ºC structural steel will retain 60% of its room temperature strength (see Figure 31). This is important because, before the introduction of limit state design concepts, when permissible stress was used as a basis for design, the maximum stress allowed in a member was about 60% of its room temperature strength. This led to the commonly held assumption that 550ºC was the highest or critical temperature that a steel structure would withstand before collapse.

Recent international research has shown, however, that the limiting (failure) temperature of a structural steel member is not fixed at 550ºC but varies according to two factors, the temperature profile and the load.

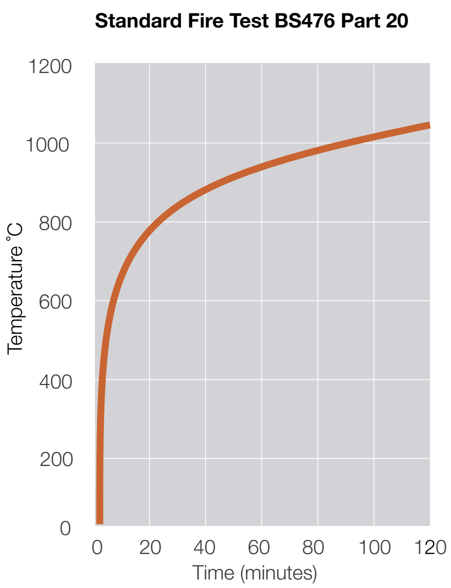


Figure 30 BS476 Part 20. Standard time-temperature relationship for fire tests.

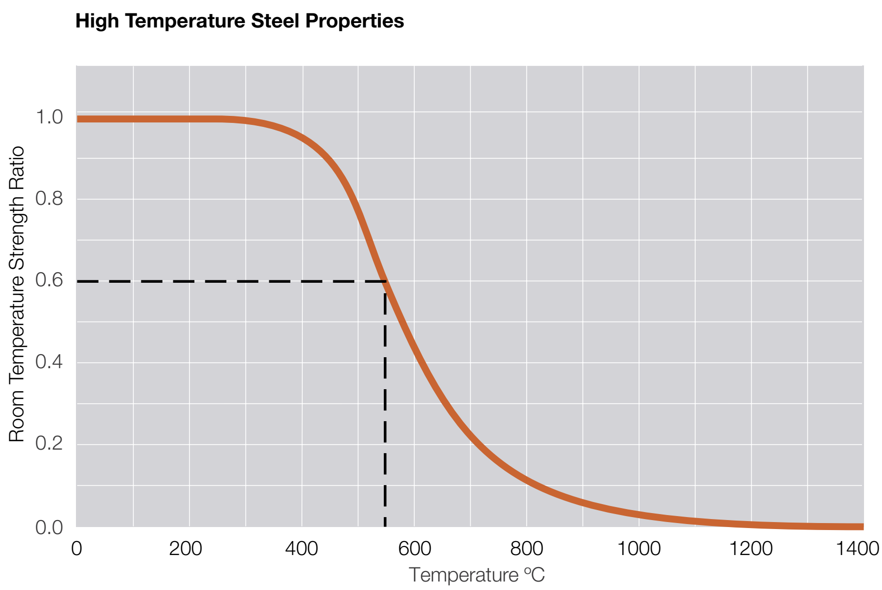


Figure 31 Steel strength decreases with temperature.

Effect of temperature profile

A joint test programme by Corus and the Building Research Establishment has shown that the temperature profile through the cross-section of a steel structural member has a marked effect on its performance in fire.

The basic high temperature strength curve shown in Figure 31 has been generated by testing a series of small samples of steel in the laboratory, where the whole of each test sample is at a uniform temperature and is axially loaded.

When these conditions are repeated in full scale member tests, e.g. unprotected axially loaded columns, then failure does indeed occur at 550°C. But if a member is not uniformly heated then, when the hotter part of the section reaches its limiting temperature, it will yield plastically and transfer load to cooler regions of the section, which will still act elastically. As the temperature rises further, more load is transferred from the hot region by plastic yielding until eventually the load in the cool regions becomes so high that they too become plastic and the member fails.

The most common situation in which temperature gradients have a significant effect on the fire resistance of structural steel is where beams support concrete slabs. The effect of the slab is both to protect the upper surface of the top flange of the beam from the fire and to act as a heat sink. This induces temperature differences of up to 200°C between the upper and lower flanges in standard fire tests. Test data shows that the limiting (lower flange) temperature of fully loaded beams carrying concrete slabs is about 620°C. This compares with 550°C for beams exposed on all four sides.

Effect of load

It is known from full scale fire tests that a simply supported beam carrying a non-composite concrete floor slab and 60% of its cold load bearing capacity will become plastic at about 620°C. It is also known that if it carries a lower load then plasticity will occur at a higher temperature. Thus, at low loads, fire resistance is increased.

In BS5950 Part 8[14] (see page 18) load is expressed in terms of the 'Load Ratio' where:

$$\textbf{Load Ratio} = \frac{\textbf{the load at the fire limit state}}{\textbf{the load capacity at 20°C}}$$

The load at the fire limit state is calculated using load factors given in BS5950 Part 8 (see page 18). A fully loaded beam in bending would normally have a load ratio of about 0.50 - 0.6. It is known from the research data that, with a load ratio of 0.25, for example, failure in simply supported beams carrying non-composite concrete slabs will not occur until the steel reaches 750°C, an increase of 130°C on the limiting temperature in the fully loaded case (Figure 32). See also page 19.

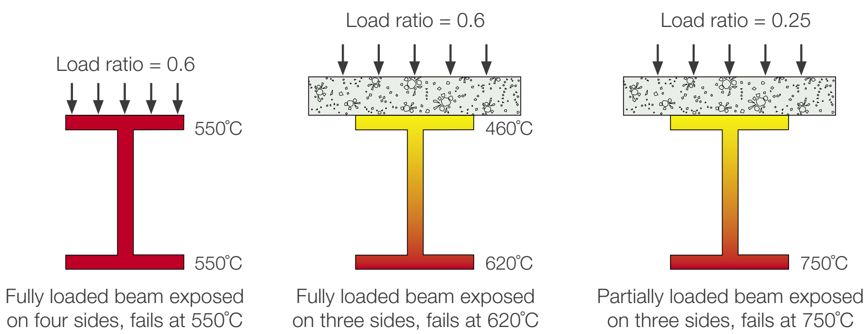


Figure 32 Effect of temperature profile and load on failure temperature.

7. BS5950 Part 8: Code of Practice for Fire Resistant Design

BS5950 Part 8 : Code of Practice for Fire Resistant Design[14]

BS5950 Part 8 (Figure 33) was published in 1990, and redrafted in 2003. It brings together in one document many of the methods of achieving fire resistance for structural steelwork. Although it is based on evaluation of performance of structural steel members in the BS476 Part 20[13] standard fire test (see page 16) it may also be used in fire engineering assessments when natural fire temperatures are derived by calculation (page 31).

BS5950 Part 8 also includes design information and guidance for design of portal frames, hollow sections, external steelwork, composite slabs and beams and calculation of protection thicknesses based on limiting temperatures. The code contains two basic approaches to assessment of fire resistance:

From Tests - in accordance with BS476 Part 21[13].

By Calculation - in accordance with either:-

- the limiting temperature method
- the moment capacity method

A commentary to the standard giving more detailed information and worked examples has been published by The Steel Construction Institute[15] (Figure 34).

Fire resistance derived from tests

All approved protection materials have been tested in accordance with BS476 and the required thickness of each product has been evaluated with regard to fire resistance period and section factor. Recommendations based on these evaluations are given in simple design tables in 'The Yellow Book'[9] published jointly by the Association of Specialist Fire Protection (ASFP) and The Steel Construction Institute (see page 10).

Figure 33 BS5950 Part 8, Code of Practice for Fire Resistant Design.

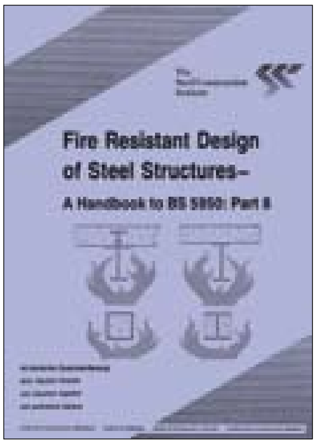


Figure 34 Fire Resistant Design of Steel Structures: A Handbook to BS5950 Part 8.

Limiting temperature method

The limiting temperature method allows the designer to assess the need, or otherwise, for fire protection by comparing the temperature at which the member will fail (the limiting temperature) with the temperature of the hottest part of the section at the required fire resistance time (the design temperature). In BS5950 Part 8 this is done via a set of prepared tables and here it is illustrated graphically. (Figure 35). If the limiting temperature exceeds the design temperature no protection is necessary (see page 9).

This can be of particular value when assessing whether unprotected steel will achieve 30 minutes fire resistance without protection. It can also be of value when calculating failure temperatures to assess how much fire protection is required for higher periods of fire resistance.

For example, if it can be shown that the failure temperature is (say) 700°C rather than 620°C, significant reduction in fire protection thickness may be possible. This can be important for intumescent coatings, especially at high fire resistance periods. It is unlikely to provide any value when using board or spray fire protection.

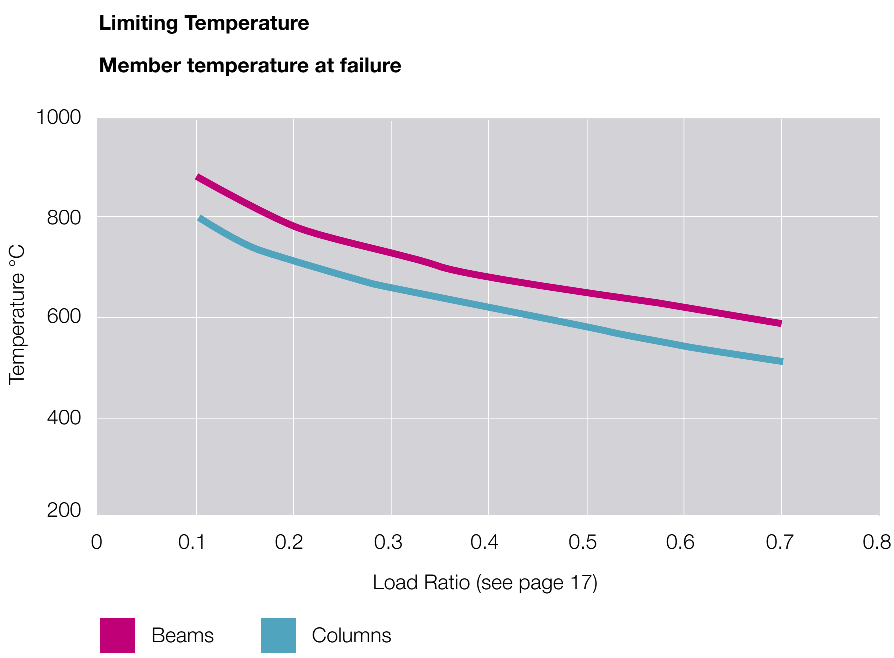


Figure 35

Moment capacity method

This calculation method allows the designer the opportunity to assess the fire resistance of a beam by calculating its moment capacity using the temperature profile at the required fire resistance time. If the applied moment is less than the moment capacity of the beam the member is deemed to have adequate fire resistance without fire protection.

The method is only applicable for beams with webs which satisfy the requirements for a plastic or compact section as defined in BS5950 Part 1[(16)]. It is best suited for use with shelf angle floor beams. Appendix E of BS5950 Part 8[(14)] gives all the information required to calculate the moment capacity of shelf angle floor beams at 30, 60 and 90 minutes and a more detailed treatment is given in the appropriate Steel Construction Institute publication (see page 22).

8. Eurocodes and fire

The Commission of the European Community began work on the harmonisation of technical specifications for construction in 1975, with the objective of eliminating technical obstacles to trade between member states. Part of this programme of work was the development of a set of harmonised technical rules, the Eurocodes, for the design of construction works, which in the first instance would provide an alternative to national design rules and, ultimately would replace them.

The following standards describe the rules for the fire design of buildings using structural steelwork:

BS EN 1991-1-2 Actions on Structures. Actions on Structures Exposed to Fire.

BS EN 1993-1-2 Design of Steel Structures. General Rules Structural Fire Design.

BS EN 1994-1-2 Design of Composite Steel and Concrete Structures. General Rules Structural Fire Design.

All are available from The British Standards Institution.

The Eurocodes provide common rules for the design of whole structures and component products. Innovative forms of construction or unusual design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

Eurocode standards recognise the responsibility of the regulatory authorities in each member state to define the required levels of safety. Consequently, each member state is required to publish a National Annex to each part of the Eurocode.
The national standards written to implement the Eurocodes will contain the full Eurocode text including annexes, which may be proceeded by a national title page and national foreword and followed by the National Annex. The National Annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters. The National Annex may also contain guidance on the application of informative annexes in the Eurocode and references to non-contradictory complementary information to assist the user to apply the design rules in the Eurocode.

At the time of writing, early 2006, the National Annex to BS EN 1991-1-2 is about to be published. This will be followed by a Published Document which will give some of the background to the National Annex and guidance on situations where the Annex cannot be accepted as alternative guidance. The National Annexes to BS EN 1993-1-2 and BS EN 1994-1-2 are expected in 2007. These standards will eventually replace BS5950 Part 8 (see page 18). The Government in the United Kingdom has not given any indication as to when this will take place and it is likely that both codes will be available simultaneously for a period of time.

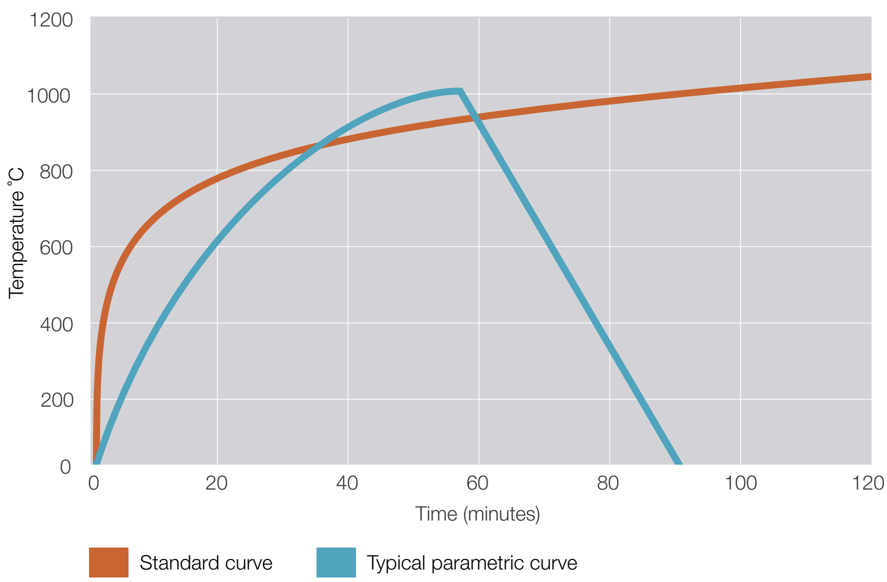
New European standards have also been developed for fire testing. Existing test standards will also be replaced in the future but, again, the time scale is not known. The new Eurocode equivalent to BS476 Part 20 (see page 16) is BS EN 1363-1:1999. At the time of writing, most intumescent fire protection manufacturers have indicated that they intend to embrace the new European requirements and work only with product performance claims to the new European Standards by mid 2008.

Historically, most countries use a fire test standard similar to that outlined in BS476. However, due to differences in the furnace manufacture, fuel used and control mechanisms, different furnaces gave very different results in what was nominally a test carried out to similar parameters. The UK test was recognised as one of the more benign regimes, the German test by contrast was one of the more onerous. The new test standards attempt to solve this problem by imposing a common mechanism of furnace control which will ensure that all furnaces across the European Union give the same results.

Much has been written about the increased severity of the new European harmonised fire test compared to that in widespread use in the United Kingdom. It is considered that the effect will be a general increase in thickness of fire protection but that this is unlikely to have significant cost implications for the steel construction sector.

The new structural design standards have a wider scope than BS5950 Part 8. They open up a number of new design possibilities including the use of what is called the parametric time-temperature curve. This is a mechanism of calculating the actual time-temperature relationship in a compartment of known dimensions and occupancy and removes historic dependence on the standard fire test. It is a major advance in the development of performance based design and forms the basis of the methods used to determine the fire resistance periods in DD9999 (see page 7).

Comparison of standard fire test curve with parametric time-temperature curve



The standard fire curve represents a fully developed room fire.
It does not account for fuel load. It does not account for ventilation.
The natural fire curves offer a more realistic assessment.

Figure 36

9. Partially exposed steelwork

Standard fire tests have shown that structural members which are not fully exposed to fire can exhibit substantial levels of fire resistance without applied protection. Methods have been developed using this effect to achieve 30 and 60 minutes fire resistance. Where higher periods of fire resistance are called for, reduced fire protection thicknesses can be applied to the exposed steelwork since the heated perimeter is less than that for the fully exposed case (see page 9).

There are four common ways in which this principle can be used:

Block-infilled columns - (Figure 37) 30 minutes fire resistance can be achieved by the use of autoclaved, aerated concrete blocks cemented between the flanges and tied to the web of rolled sections. Longer fire resistance periods are possible by protecting only the exposed flanges[17].

Web-infilled columns - (Figure 38) 60 minutes fire resistance is obtained when normal weight, poured concrete is fixed between column flanges by shear connectors attached to the web. The concrete is retained by a web stiffener fixed at the bottom of the connection zone.

The load carrying capacity of the concrete is ignored in the design of the column but in fire, as the exposed steel weakens at high temperatures, the load carried by the flanges is progressively transferred to the concrete. This provides stability in fire for periods of up to 60 minutes. The connection zone at the top of the column is protected along with the beam[18].

Shelf angle floor beams - (Figure 39) are beams with angles welded or bolted to the web to support the floor slab. This protects the top part of the beam from the fire while the bottom part remains exposed. Fire resistance increases as the position of the supporting angle is moved further down the beam and fire resistance periods of 60 minutes are achievable in some instances[19].

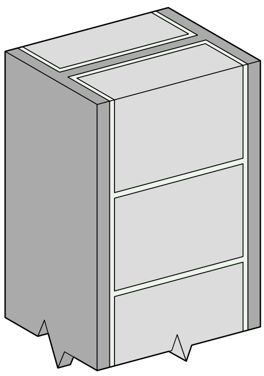


Figure 37 Block infilled column.

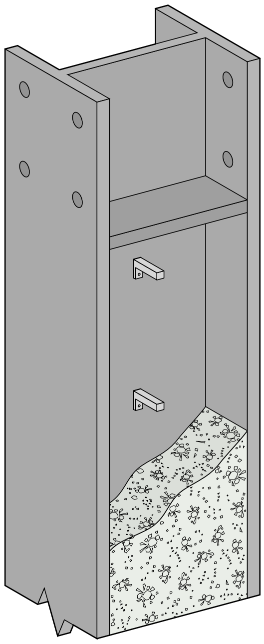


Figure 38 Web infilled column.

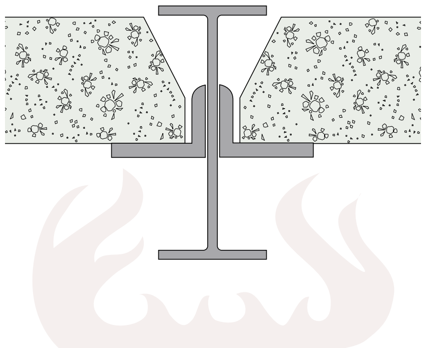


Figure 39 Shelf angle floor beam.

Slim floor beams - (Figures 40 and 41) In the UK there are two main slim floor options. The first, known as Slimflor®, comprises a column section with a plate welded to the bottom flange to support deep steel decking, or in some circumstances pre-cast concrete slabs. Almost the whole section is protected from the fire by the floor slab and periods of fire resistance up to 60 minutes are achievable without protection to the exposed bottom plate[20] [21].

The second option also uses deep decking but removes the support plate by using an asymmetric beam (Figure 42). This eliminates welding but retains the easy assembly and the 60 minute fire resistance properties of the original design. This system has been patented by Corus under the trade name Slimdek®[22].

The shape of the asymmetric beam is uniquely designed to give optimum performance in fire. A thick web / thin flange configuration gives maximum capacity under the non-uniform temperature distribution at the fire limit state. Slimdek can also be used with precast planks and design guidance will be available from mid 2006.

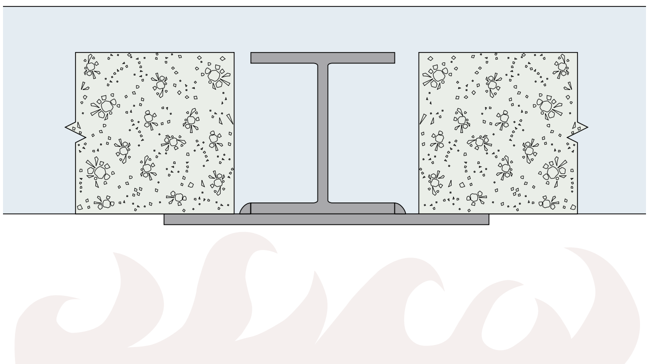


Figure 40 Slimflor® with precast slab.

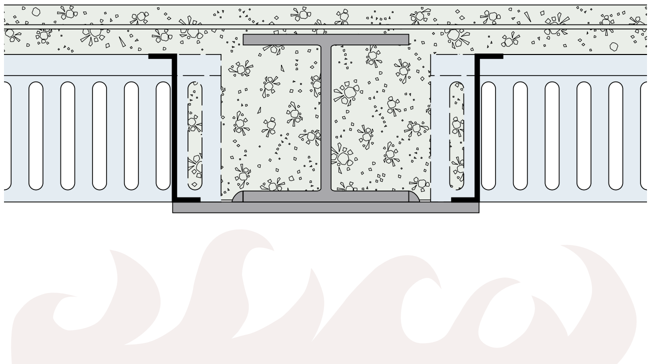


Figure 41 Deep deck Slimflor® system.

Figure 42 The asymmetric beam used in the Slimdek® system is designed for 60 minutes fire resistance without protection and composite action without welded studs.

10. Combining fire resistant design methods

Fire resistance (in minutes) that can economically be obtained for various structural forms.

BEAM TYPE / COLUMN TYPE	Unprotected beam	Slimflor systems	Shelf angle floor	Partially encased	Protected beam
Unprotected column	15	15	15	15	15
Blocked infilled column	15	30	30	30	30
Concrete infilled unreinforced	15	60	60	60	60
Concrete infilled reinforced	15	60	60	>60	>60
Concrete filled hollow section	15	60	60	>60	>60
Protected column	15	60	60	>60	>60

The innovative design solutions for beams and columns described above can be combined so that whole buildings with fire ratings up to 1 hour can be realised without recourse to site applied protection.

Further details can be found in SCI publication Design of Steel Framed Buildings Without Applied Fire Protection[23].

Figure 43 Design of Steel Framed Buildings Without Applied Fire Protection.

11. Filled hollow sections in fire

Unprotected hollow sections can attain up to 2 hours fire resistance when filled with concrete. When the combined section is exposed to fire, heat flows through the steel into the concrete core which, being a poor conductor, heats up slowly. As the steel temperature rises its yield strength steadily decreases and the load is progressively transferred to the concrete. The steel then acts as a restraint to restrict spalling of the concrete. BS5950 Part 8[14] contains a calculation method for checking the axial and moment capacities of square and rectangular columns in fire. Guidance on the fire resistant design of unprotected concrete-filled circular, elliptical, square and rectangular hollow sections is given in BS EN 1994-1-2[24] and in CIDECT Design Guide No. 4[25] (Figure 44).

Three types of filling are possible, plain, fibre reinforced or bar reinforced concrete. Plain and/or fibre reinforced concrete performs well under compression loading but performs less well when a column is subject to significant moments. The moments about the major and minor axes must be limited, when using plain or fibre reinforced concrete, so as to ensure that the column remains in overall compression under the combined fire limit state axial load and moments.

When moments above these limits are present, the capacity of the concrete filled column can be further enhanced by the addition of bar reinforcement. The calculation method for checking the axial and moment capacities is given in BS5950 Part 8 Section 8.6.1 and the references contained in this section.

As an alternative, a concrete filled hollow section column can be designed to its full composite capacity and then be protected by a board, spray or intumescent coating system. In this case it is still possible to exploit the improved thermal properties of the filled column to reduce the level of external protection used. For board and passive spray systems, this is determined by calculating the passive protection requirement based on the empty hollow section and then reducing the thickness by a modification factor using a tabulated method given in BS5950 Part 8 Section 8.6.2. Similar reductions are also possible with an intumescent coating. However, each individual product must be assessed separately to ascertain these allowable reductions. Further information is available in the Corus Tubes publication Intumescent Coatings and SHS Concrete Filled Columns[26] (Figure 45).

Most of the above can also be found in greater detail, together with information on the advantages, limitations and methodologies of achieving fire resistance using concrete filled tubes in: Design Manual for Concrete Filled Columns, Part 2: Fire Resistant Design for designs done to BS Codes and, for designs to Eurocodes, Design Guide for SHS Concrete Filled Columns, CT26[27]. These publications are available on the Corus Tubes website (www.corustubes.com) and on a Corus Tubes CD.

Design software for the design of unprotected SHS columns has now been developed, using advanced methods based on Eurocode 4. This software takes account of axial load and bending, as well as the use of steel section inserts within the tubular section. Further information is available on the Corus Tubes website or by contacting Corus Tubes on +44 (0)1724 405060.

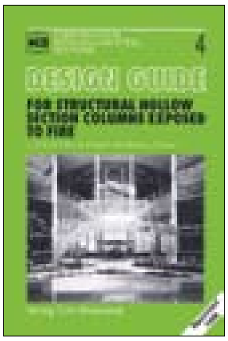


Figure 44 CIDECT Design Guide No. 4.

Figure 45 Intumescent Coatings and SHS Concrete Filled Columns.

12. Single storey buildings in fire

In the UK, single storey buildings do not normally require fire protection. The definition of elements of structure in Approved Document B (see Page 4), Section 8.4, excludes structure that only supports a roof. Exceptions may occur where the structural elements form part of:

- a separating wall.
- a compartment wall or the enclosing structure of a protected zone.
- an external wall which must retain stability to prevent fire spread to adjacent buildings (i.e. a boundary condition).
- a support to a gallery or a roof which also forms the function of a floor (e.g. a car park or a means of escape).

By far the most common structural form for single storey industrial buildings are portal frames and the most common scenario in which fire protection is required is a boundary condition. Boundary conditions occur as a result of the requirement for adequate space separation between buildings as outlined in Part B of Schedule 1 of the Building Regulations 2000:
"The external walls of the building shall offer adequate resistance to the spread of fire over the walls and from one building to another, having regard to the height, use and position of the building."

Where fire resistance is required in a boundary condition, it has been widely accepted that it is necessary only for the affected wall and its supporting stanchions to be fire protected. The rafters and other walls may be left unprotected but the stanchion base on the affected side must be designed to resist the overturning moments and forces caused by the collapse of the unprotected parts of the building in fire. The method of calculation used to derive the horizontal forces and moments created by rafter collapse is given in The Steel Construction Institute publication, Single Storey Steel Framed Buildings in Fire Boundary Conditions[28] (Figure 46).

Figure 46 Single Storey Steel Framed Building in Fire Boundary Conditions.

This document contains guidance not just on simple portal frames but also on portal frames with lean-to structures, two storey sections etc. as well as the design of single storey buildings utilising truss and lattice rafters.

Most authorities expect engineers to design single storey buildings for boundary conditions in this way. In England, Wales & Northern Ireland it is not necessary to apply for a relaxation if it is shown that The Steel Construction Institute document has been used as the basis for design. On the same basis, a class relaxation is available in Scotland.

The SCI document advises on the use of sprinklers in single storey boundary conditions:

It advises that Approved Document B (see page 4) recognises that there is a reduced risk of fire spread in buildings where sprinklers are installed. The boundary distance for a building with sprinklers may be halved or the unprotected area in the wall may be doubled. Also, where the recommendations of the SCI document are followed, the requirements to design the foundation to resists the overturning moment from the collapse of the roof need not be followed.

In Scotland, although the England & Wales approach is considered reasonable, it is up to local authorities to grant relaxations to the regulations on an individual basis.

In Northern Ireland, the regulations follow the England & Wales approach although there is no specific statement as to the issue of design for overturning moment.

13. External steelwork

A number of modern steel buildings have being constructed with the steel skeleton on the outside of the structure (Figures 47 and 48). Since an external structural frame will only be heated by flames emanating from windows or other openings in the building facade, the fire that the steelwork experiences may be less severe than in an orthodox design. It may be possible to design the frame members to remain unprotected or to have reduced protection if they are positioned so that they not be engulfed by flames and hot gases issuing from facade openings. Assessment can be carried out in accordance with The Steel Construction Institute publication Fire Safety of Bare External Structural Steel[29] (Figure 49). This describes a method to define the design temperature (see page 19) of the structural members from consideration of their location in relation to the openings, their distance from the facade, the fire load and ventilation characteristics of the compartments and the potential effects of wind.

Comparison of the calculated design temperature with the limiting temperature of members calculated from BS5950 Part 8 (see page 19) will indicate whether or not protection is necessary.

Clearly consideration must be given to suitable corrosion protection methods and guidance can be found in the appropriate Corus design guide[30]. In addition design against brittle fracture should also be considered and design guidance is given in BS5950 Part 1[16].

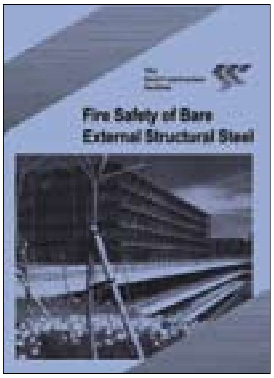


Figure 49 Fire Safety of Bare External Structural Steel.

Figure 47 DSS Building, Newcastle.

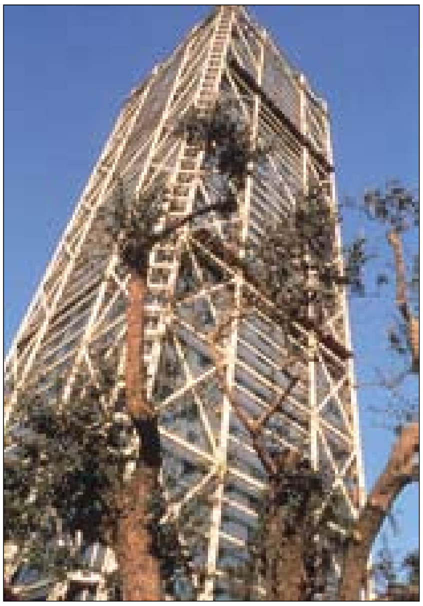


Figure 48 Hotel de las Arte, Barcelona.

14. Composite steel deck floors in fire

Assessment of composite slabs

A composite steel deck floor (Figure 50) is designed in bending as either a series of simply supported spans or a continuous slab. Strength in fire is ensured by the inclusion of reinforcement. This can be the reinforcement present in ordinary room temperature design; it may not be necessary to add reinforcement solely for the fire condition.

In the fire condition it is normal, although conservative, to assume that the deck makes no contribution to overall strength. The deck does however play an important part in maintaining integrity and insulation. It acts as a diaphragm preventing the passage of flame and hot gases, as a shield reducing the flow of heat into the concrete and it controls spalling. It is not normally necessary to fire protect the exposed soffit of the deck.

In fire the reinforcement becomes effective and the floor behaves as a reinforced concrete slab with the loads being resisted by the bending action. Catenary action may develop away from the edges of the floor with the reinforcement then acting in direct tension rather than bending. Slab failure occurs when the reinforcement yields.

Two methods are available for the design of composite metal deck floors, both of which are described in The Steel Construction Institute publication, The Fire Resistance of Composite Floors with Steel Decking[31] (Figure 51). These are the fire engineering and the simple method.

In the fire engineering method it is assumed that the plastic moment capacity of the floor can be developed at elevated temperatures and that redistribution of moments takes place in continuous members. The hogging and sagging moment capacities of the slab are calculated via temperature distributions based on extensive fire testing covering periods of up to four hours. These are then compared with free bending moments for both internal and end spans at the required fire resistance period and the design adjusted as necessary to ensure that the floors meet the required criteria.

The simple method consists of placing a single layer of standard mesh in the concrete. Guidance is available on maximum loads, reinforcement size and position and also allowable span and support conditions.

In practice the simplified method will almost invariably lead to the use of less reinforcement than the fire engineering method. The fire engineered method however allows greater flexibility in reinforcement layout, loading and achievable fire resistance times.

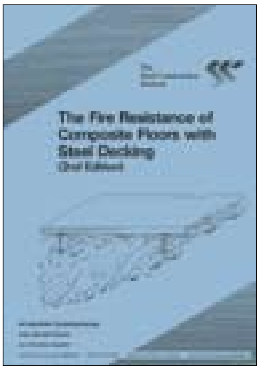


Figure 51 The Fire Resistance of Composite Floors with Steel Decking.

Typically the use of the fire engineering method will result in thinner slabs.

Lightweight concrete is a better insulator and thus loses strength less rapidly in fire than normal weight concrete. Hence lightweight concrete floors tend to be thinner than normal weight alternatives.

Figure 50
Composite steel deck floor.

Deck voids

Research has shown that filling the gaps between the raised parts of the deck profile and the beam top flange in composite construction is not always necessary. The upper flange of a composite beam is so close to the plastic neutral axis that it makes little contribution to the bending strength of the member as a whole. Thus, the temperature of the upper flange can often be allowed to increase, with a corresponding decrease in it’s strength, without significantly adversely affecting the capacity of the composite system.

Gaps under decking with dovetail profiles can remain unfilled for all fire resistance periods. The larger voids which occur under trapezoidal profiles can be left open in many instances for fire ratings up to 90 minutes, although some increase to the thickness of protection applied to the rest of the beam may be necessary (Figure 52). Details are given in The Steel Construction Institute publication The Fire Resistance of Composite Floors with Steel Decking[31] (Figure 51).

Designers should take care that gaps are filled where the beam forms part of the compartment wall to ensure the integrity of the compartment. In the rare case where non-composite metal deck construction is used, the gaps must always be filled.

Recommendations for unfilled voids in composite and non-composite beams

Trapezoidal deck

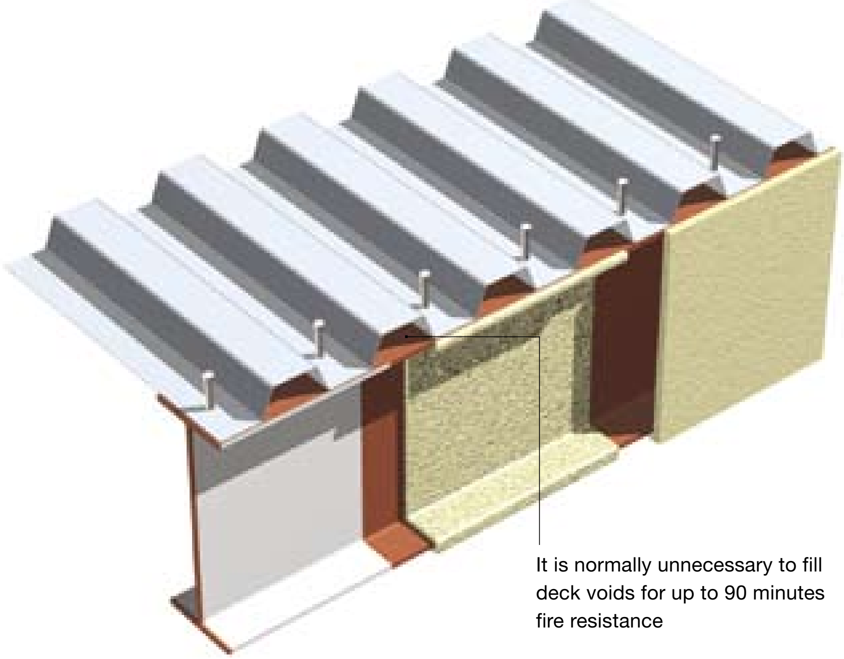
Beam type	Fire protection on beam	Fire resistance (minutes)		
		Up to 60	90	Over 90
Composite	Insulating sprays and boards (assessed at 550°C)	No increase in thickness	Increase thickness by 10% or assess thickness using Hp/A increased by 15%*	Fill voids
	Intumescent coatings (assessed at 620°C)	Increase thickness by 20% or assess thickness using Hp/A increased by 30%*	Increase thickness by 30% or assess thickness using Hp/A increased by 50%*	Fill voids
Non-Composite	All types	Fill voids		

Dovetail deck

Beam type	Fire protection on beam	Up to 60	90	Over 90
Any	All types	Voids may be left unfilled for all periods of fire resistance		

* The least onerous option may be used.
This table should be used in conjunction with the thicknesses of fire protection specified in 'The Yellow Book'[9] or manufacturer's data. Table reproduced from 'The Yellow Book'.

Figure 52 Composite beams can be protected with intumescent, spray or board protection.



15. Structural fire engineering

Increasing innovation in design, construction and usage of modern buildings has created a situation where it is sometimes difficult to satisfy the functional requirements of the Building Regulations by use of the provisions given in Approved Document B and other similar documents (see pages 4-7). Recognition of this, and also increased knowledge of how real buildings react in fire and of how real fires behave, made possible by a wide ranging and intensive and programme of research and development world-wide, has led many authorities to acknowledge that improvements in fire safety may now be possible in many instances by adopting analytical approaches. Thus Approved Document B states that:

"Fire safety engineering can provide an alternative approach to fire safety. It may be the only practical way to achieve a satisfactory standard of safety in some large and complex buildings and in buildings containing different uses."

Fire safety engineering can be seen as an integrated package of measures designed to achieve the maximum benefit from the available methods of preventing, controlling or limiting the consequences of fire. The Institution of Structural Engineers defines it as a process *"…aimed at adopting a rational scientific approach which ensures that fire resistance/protection is provided where it is needed rather than accepting universal provisions which may over or under estimate the level of risk."* [32]

The move from prescriptive to functional requirements in the Building Regulations in the United Kingdom provided a huge boost to the development of fire engineering and this country can now lay claim to many of the leading consultancies in this field in the world. As a consequence, the majority of tall and complex buildings now benefit from a fire engineering approach rather than relying on the blanket provisions of the Approved Documents or similar. This has proved extremely beneficial to the construction industry as a whole.

BS7974[33] contains guidance on the procedures for carrying out a fire engineering analysis.

The introduction describes its purpose as being to provide a framework for developing a rational methodology for the design of buildings using a fire safety engineering approach based on the application of scientific and engineering principles to the protection of people, property and the environment from fire.

The code is accompanied by a series of published documents giving detailed guidance on the principles of fire engineering, fire development, spread of smoke, structural response, fire detection, fire service intervention, evacuation and risk assessment.

Fire engineering can deliver value across the five areas of activity in the provision of fire precautions in buildings. These are:

- means of warning and escape
- internal fire spread
- structural response
- external fire spread
- access and facilities for the fire service.

The following text concentrates on the third of these, structural response.

Structural Response

Structural fire engineering is a three stage process:

1. Predicting the heating rate and maximum temperature of the atmosphere inside the fire compartment. This involves assessing the fire load (the quantity and type of combustible material) in the compartment, the ventilation and the thermal characteristics of the compartment linings. These variables can be calculated or obtained from tabulated data. Once known, one can estimate the temperature rise in the compartment with time either as a parametric time-temperature relationship (see page 21) or as a time equivalent (the exposure to a standard BS476 fire that would have the same effect as the natural fire in the compartment under consideration).

2. Predicting the temperature of the structure. This depends on the location, the section factor and any protection applied. The temperatures attained by unprotected steel members can be determined using heat transfer relationships given in BS EN 1991-1-2 and BS EN 1993-1-2 (see page 20). In BS 5950 Part 8[14] the temperatures attained in a standard fire test are also given in tabular format. These are provided for fire ratings from 15 to 60 minutes. The temperatures attained by protected members can also be calculated according to BS EN 1993-1-2. In addition, temperatures can also be calculated using bespoke models calibrated against actual test results.

3. Predicting the response of the structure. The response of the structure depends not only on the temperature it reaches in the fire but also on the applied loads and the effects of any composite action, restraint and continuity from the remainder of the structure. Once it is known, protection requirements can be specified to meet the fire hazard.

This design concept proves most cost effective when it can be shown that the structure, or parts of the structure, has sufficient inherent fire resistance to avoid the need to apply fire protection.

Typical of the type of situation where structural fire engineering is of considerable value is the design of sports stadia. Modern developments incur considerable investment and clients are seeking alternative means of attracting revenue on capital outlay. This means that some sports stadia can no longer be describes as simple steel, concrete and blockwork structures for the sole purpose of watching sport. Instead they are mixed occupancy often containing shops, restaurants and conference facilities. This can create difficulties in developing fire safety policies consistent with the approaches assumed in documents such as the Approved Document. A solution can often be found for such situations using fire engineering. Examples are given in Stadium Engineering[34].

Other structures designed using modern fire engineering techniques include offices, industrial buildings, airport terminals, leisure centres hospitals, shopping centres and car parks.

Typical buildings which have benefited from a fire engineering approach.

Figure 53 Emirates Stadium at Ashburton Grove, the new home of Arsenal Football Club. Courtesy of HOK Sport Architecture.

Figure 54 GLC building in London.

Figure 55 XSCAPE building in Milton Keynes.

16. Cardington fire tests design guidance

Between 1994 and 2003, a series of seven fire tests were carried out on an eight storey steel framed building with composite metal deck floors at the Building Research Establishment facility at Cardington in Bedfordshire. The test programme was divided into two parts; the first, comprising a single beam test and three large compartment tests was funded partly by Corus and partly by the European Coal and Steel Community (now the Research Fund for Coal & Steel). A complementary programme, comprising three compartment tests was Government sponsored and carried out by the Building Research Establishment.

Figure 56 The Cardington frame is a multi-storey composite structure, i.e. the floors are constructed using shallow composite slabs with profiled steel decking attached by shear connectors to downstand beams. The design guidance developed from the fire tests applies only to frames of this type.

Cardington fire tests

The tests were carried out to determine if the fire performance of real buildings was better than is suggested by tests on individual elements of construction. Evidence that this was the case had been provided by actual fires in real buildings[35], tests carried out in Australia[36] and also small scale fire tests and computer modelling of structural behaviour. In all these cases, composite floors had demonstrated robustness and resistance to fire far greater than was indicated by tests on single beams or slabs.

In order to obtain a direct comparison with the standard fire test, the first test was carried out on a single unprotected beam and surrounding area of slab. The results indicated that a failure deflection would have occurred at a temperature over 1000°C, far greater than the temperature of 700°C at which it would have failed if tested in isolation.

Further tests were carried out in compartments varying in size from 50m^2 to 340m^2 with fire loadings provided by gas, wooden cribs and standard office furniture. Columns were protected but beams were not. Despite atmosphere temperatures of over 1200°C and temperatures on the unprotected steel beams of 1100°C in the worst case, no structural collapse took place.

The full set of test data from the Corus tests can be found at www. structuralfiresafety.org (see page 37).

Figures 57, 58, 59 Office fire loading supplemented with wooden cribs produced the most extreme temperatures in any of the six fire tests. Despite this, the unprotected steel beams (which reached temperatures in excess of 1100°C) and floor did not collapse.

Fire resistance of composite floors

Observations from the Cardington fire tests and other large building fires have shown that the behaviour of the composite floor slab plays a crucial role in providing enhanced fire resistance when compared to that achieved by tests on single isolated elements of construction. The Cardington tests demonstrated that, where significant numbers of beams are not protected, the slab acts as a membrane supported by cold perimeter beams and protected columns. As the unprotected beams lose their load carrying capacity, the composite slab utilises its full bending capacity in spanning between the adjacent cooler members. With increasing displacement, the slab acts as a tensile member carrying the loads in the reinforcement which then become the critical element of the floor construction. Using the conservative assumption of simply supported edges, the supports will not anchor these tensile forces and a compressive ring will form around the edges of the slab. Failure will only occur at large displacements with the fracture of the reinforcement.

Cardington design guidance

A simple structural model has been developed which combined the residual strength of the steel composite beams with strength of the slab. This model uses a combined yield line and membrane action approach to take into account the enhancement to slab strength from tensile membrane action. The Steel Construction Institute has developed this model into a series of design tables which have been published in Fire Safe Design: A New Approach to Multi-Storey Steel Framed Buildings[37] (Figure 60).

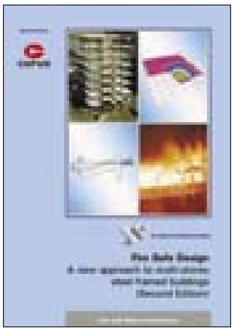


Figure 60 Fire Safe Design: A new approach to multi-storey steel framed buildings (Second Edition).

Use of these tables allows the designer to leave large numbers of secondary beams unprotected in buildings requiring 30 to 120 minutes fire resistance although some compensating features, such as increased mesh size and density may be required. The publication also contains design examples and background to the Cardington tests.

Of necessity, the design tables are restricted in the range of loads and spans which can be addressed. To increase the scope, the programme used to generate the tables has been made available at: www.corusconstruction.com/en/reference/software.

This web site allows the designer to use parametric time-temperature relationships as well as standard fire curves (see page 21). It is recommended that these are exploited only by people experienced in their use.

Figure 61 T-Mobile HQ, Hatfield. Use of Fire Safe Design led to unprotected secondary beams and significant economies in fire protection costs.

Advanced fire modelling

The design tables described in the previous page are limited to use in rectangular grids and the underlying methodology is restricted by some conservative assumptions. These problems can be circumvented by the use of advanced structural models which have been validated using the results of the Cardington tests. Such models are increasingly used and can yield appreciably better results than those derived from the design tables. They are however more complex and specialised.

The use of advanced structural models is not all about reducing the amount of fire protection. Indeed, additional fire protection may well be specified in areas of special risk or on critical components such as connections.

In addition, this type of engineering requires the co-operation of the entire design team if its full potential is to be realised and it is strongly advised that fire safety must be part of the remit of the structural engineer (in particular) from the start.

Structural engineers, fire engineers, architects, clients and representatives of the local authority need to communicate throughout the design process. It is important to understand that the value which the fire engineers can deliver is directly proportional to the input which they have in the design. The fire engineer must be given the opportunity to work closely with the architect and engineer to understand the features of the structure and to be able to communicate detailing changes sometimes required to allow the development of advanced capacity in fire.

Figure 62 Plantation Place South: an office building in the centre of the City. An advanced fire analysis demonstrated that much of the steelwork could be left unprotected.

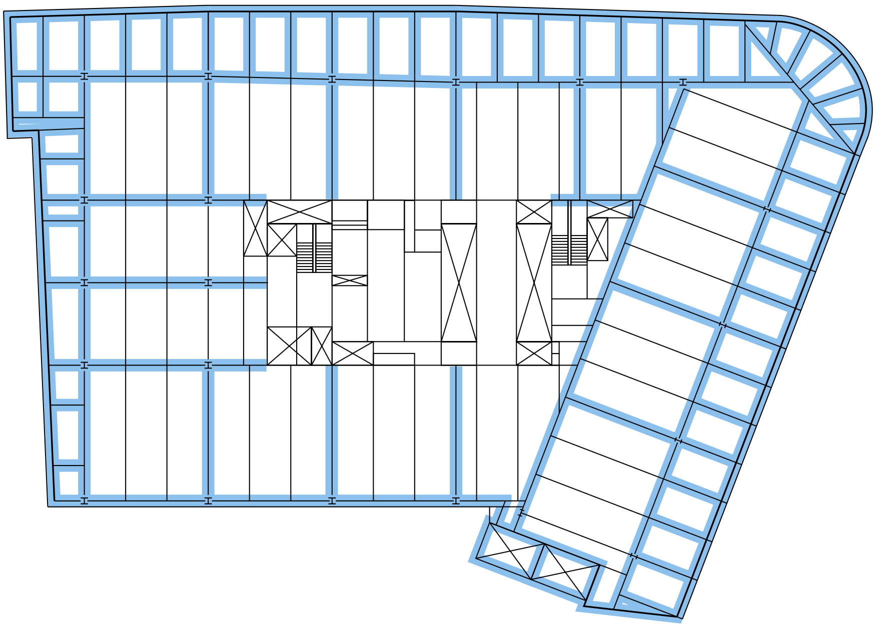


Figure 63 The floorplate at Plantation Place South. Highlighted beams only are fire protected. Courtesy of Arup Fire.

17. Fire damage assessment of hot rolled structural steel

The assessment of fire damaged hot rolled structural steel is an area in which most engineers and architects have little practical experience. On many occasions fire affected steelwork shows little or no distortion resulting in considerable uncertainty regarding its re-usability. This is particularly true in situations where fire has resulted in some parts of the structure exhibiting little or no damage alongside areas where considerable damage and distortion are clearly visible.

The principal source of information on this subject can be found in the Corus Publication 'The Reinstatement of Fire Damaged Steel and Iron Framed Structures[38] (Figure 64). Its main conclusions are summarised here.

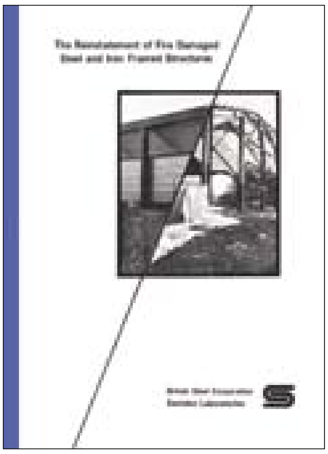


Figure 64 Reinstatement of Fire Damaged Steel and Iron Framed Structures.

Reasons for fire damage

All materials weaken with increasing temperature and steel is no exception. Strength loss for steel is generally accepted to begin at about 300°C and increases rapidly after 400°C. By 550°C steel retains about 60% of its room temperature yield strength (see page 16). This is usually considered to be the failure temperature for structural steel. However, in practice this is a very conservative assumption; low loads, the insulating effects of concrete slabs, the restraining effects of connections etc. mean that real failure temperatures can be as high as 750°C or even higher for partially exposed members.

Behaviour of BS EN 10025 grade S275 steel (formerly grade 43)

A modern grade S275 hot rolled structural steel section, subjected to fire conditions which raises its temperature above 600°C, may suffer some deterioration in residual properties on cooling. In no situation however, whatever the fire temperature, will the room temperature yield stress or the tensile strength will fall further than 10% below their original values. Thus, where it can be safely concluded that the steel members will be utilised to less than 90% of their maximum load bearing capacity or that any loss in strength will not bring the properties below the guaranteed minimum, replacement should not be considered necessary providing the member satisfies all other engineering requirements (e.g. straightness).

Behaviour of BS EN 10025 grade S355 steel (formerly grade 50)

Grade S355 hot rolled structural steel also suffers losses in residual yield and tensile strength when subjected to temperature over 600°C in fire. High strength steels, of which grade S355 is typical, obtain their characteristics as the result of the addition of strengthening elements, typically vanadium and niobium. At high temperatures these elements tend to precipitate out of the matrix creating a coarse distribution.
As a result the reduction in yield strength at room temperature after the steel has been heated to temperatures above 600°C, may be proportionately greater than for unalloyed mild steels.

Re-use of fire damaged steel

An often quoted general rule for fire affected hot rolled structural steels is that if the steel is straight and there are no obvious distortions then the steel is probably still fit for use. At 600°C the yield strength of steel is equal to about 40% of its room temperature value; it follows therefore that any steel still remaining straight after the fire and which had been carrying an appreciable load was probably not heated beyond 600°C, will not have undergone any metallurgical changes and will probably be fit for re-use.

However, where the load in the fire was less than the full design load, and also with high strength steels, this cannot always be held to be true. In such cases it is recommended that hardness tests are carried out on the affected steel. In practice it is recommended that, in all instances, some hardness tests should be carried out. For grade S275 steel, if the ultimate tensile strength resulting from the tests are within the range specified in Table 2 then the steel is reusable.

For grade S355 steel additional tensile test coupons should be taken from fire affected high strength steel members when hardness tests show that:

1 – there is more than 10% difference in hardness compared to non-fire affected steelwork, or

2 – hardness test results indicate that the strength is within 10% of the specified minimum.

Grades	Brinell Hardness Number	Vickers Hardness Number	Ultimate Tensile Strength N/mm^2
Grades S355	187	197	637
Grades S355	179	189	608
Grades S355	170	179	559
Grades S355 / Grades S275	163	172	539
Grades S355 / Grades S275	156	165	530
Grades S355 / Grades S275	149	157	500
Grades S275	143	150	481
Grades S275	137	144	481
Grades S275	131	138	461
Grades S275	126	133	451
Grades S275	121	127	431

Table 2 Brinell and Vickers hardness numbers with equivalent ultimate tensile strength values.

Where deflections are visible, general guidelines on the maximum permissible levels of deflection to ensure satisfactory performance are difficult to specify. The amount of deflection or distortion must be checked so that its effect under load can be calculated to ensure that permissible stresses are not exceeded and the functioning of the building is not impaired. Therefore every building should be considered as a separate case and the structural engineer involved in the reinstatement exercise must decide what level is acceptable to satisfy the relevant Codes.

Connections and foundations

The tensile strength reduction for grade 4.6 bolts is similar to that for S275 steel. For grade 8.8 bolts, which are heat treated in manufacture, the residual strength reduction is more marked if the material temperature has exceeded 450°C. The residual strength of these bolts falls to 80% and 60% after reaching temperatures of 600°C and 800°C respectively.

To err on the side of caution it is recommended that bolts should be replaced if they show any sign of having been heated e.g. blistered paint, smooth grey scaled surface.

Contraction of heated members after the fire can cause distortion of connections. When carrying out an inspection of a fire damaged building it is recommended that special care is taken in inspecting the connections for cracking of welds, end plate damage, bolt failure etc. A number of bolts should be removed to inspect for distortion. Similar care should be taken when inspecting foundations for bolt failure, concrete cracking etc.

18. One Stop Shop for structural fire engineering

One Stop Shop

This document has explained the fundamental principles of structural fire engineering, a discipline which is developing at a significant pace. New design methods are continually being developed based on theoretical and experimental research and designers and clients are increasingly aware of the potential contribution of fire engineering to the construction of economical, robust and innovative buildings.

To assist UK industry in the procurement of efficient and economical construction projects, and to support the application of the latest technology associated with structural fire engineering, the One Stop Shop web site (www.structuralfiresafety.org) has been developed at the University of Manchester, supported by the Department of Trade & Industry. Support was also provided by twelve industrial partners as well as representatives from leading design consultancies, approving bodies, the fire brigade and professional institutions.

The web site provides free practical and impartial advice on all aspects of structural fire engineering allowing the full benefits of previous research, development and experience to be utilised in practice. The information on the site includes sections on how to design for fire, quick solutions for the non-expert, case studies, material behaviour, references and test data.

Figure 65 University of Manchester's One Stop Shop web site: www.structuralfiresafety.org.

A design section covers both prescriptive and performance-based approaches. Available design methods for the performance based approaches are explained for the three components of structural fire engineering: modelling the fire; determining the heat transfer to the structure; and high temperature structural analysis. The background research supporting the design methods is presented and explained.

The web site also contains test data supporting current code provisions for fire resistance. This includes all tests carried out in the UK and the full data set from the Cardington fire tests (see page 32).

The web-site also provides electronic CPD courses to educate designers in the use of structural fire design codes (including the new Eurocodes) for all the common framing materials.

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