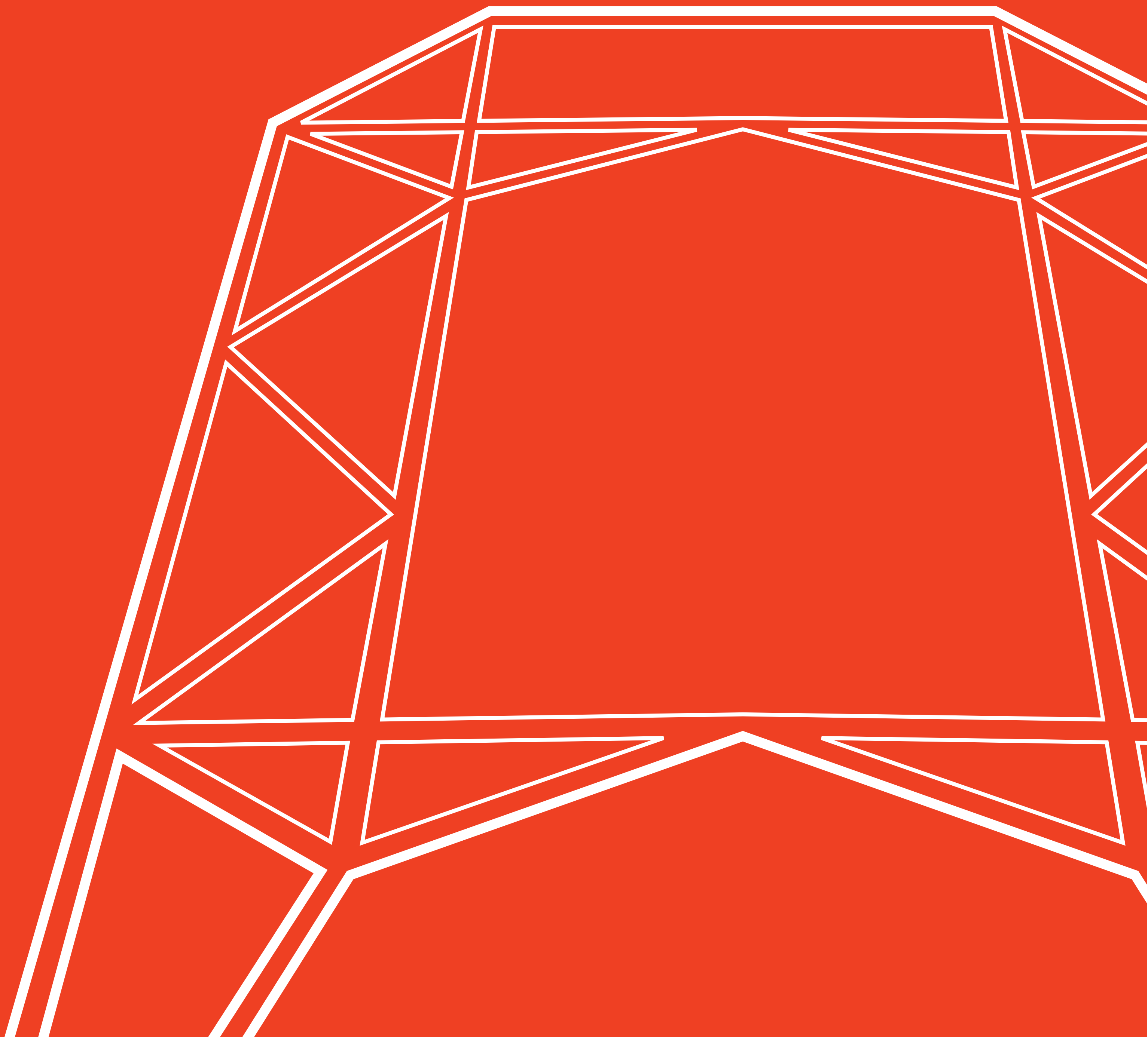
Corus Construction & Industrial

Fire resistance of steel framed buildings

2003 edition



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Foreword

The 1980's and 1990's were a time of rapid change in the field of fire and steel construction. It was a period during which new thinking and research conducted over many years was increasingly put into practice.

The Approved Document approach to satisfying regulatory requirements in England & Wales in the mid 1980's began a recognition of modern practice that continued into the '90's 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 29).

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 21). The pace of change will increase over the next decade as methods are developed to allow design for fire to move away from consideration only of simple elements towards whole building behaviour in fire. (Page 30-31).

This publication is a guide to the latest thinking in the field. It will be updated frequently to ensure its relevance as a source of information on the fire resistance of buildings.

It is concerned primarily with solutions to structural fire protection issues in steel framed buildings.

This brochure may be used in conjunction with the Steel Construction Institute publication; Structural Fire Safety; A Handbook for Architects and Engineers (1).

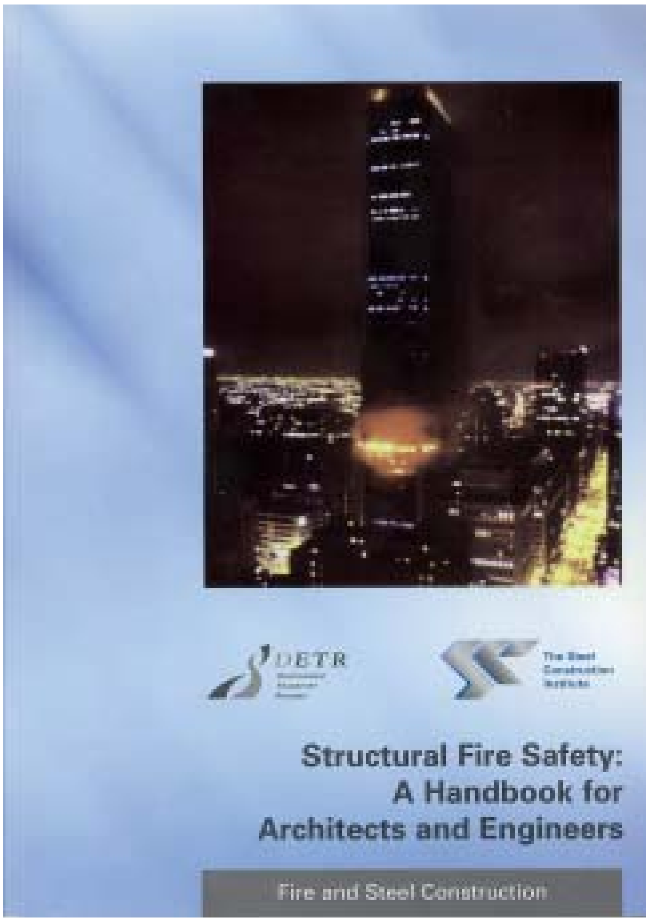


Figure 1

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 1991 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”.

How this requirement might practically be achieved is the subject of this publication.

Approved Document B 2000 (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.

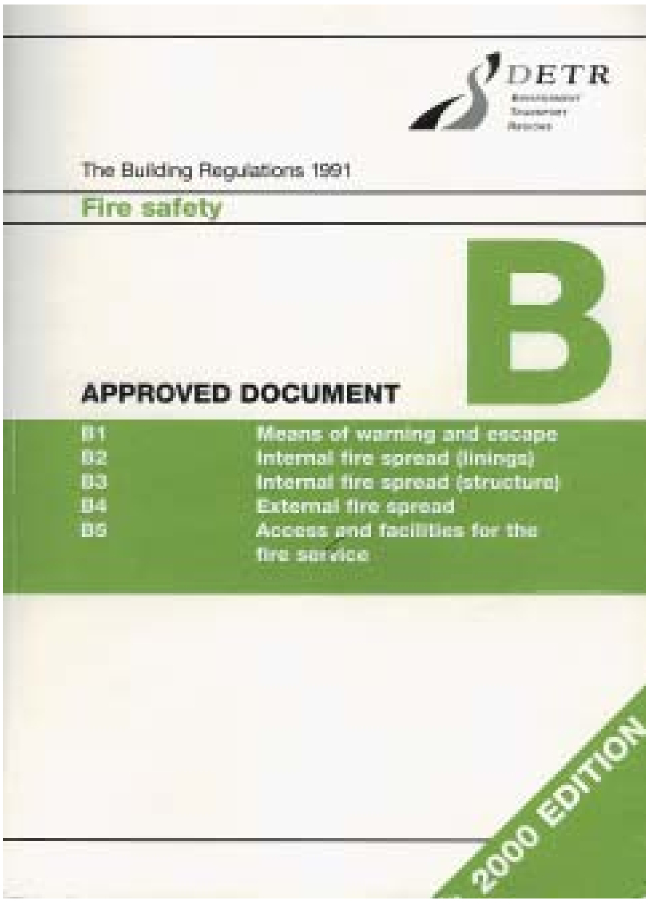


Figure 2
Approved Document B to the Building Regulations for England & Wales, 2000.

Table 1 - Fire resistance in minutes

	Height of top storey - metres			
England & Wales recommendations 2000	**<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*	
Shops, Commercial Assembly	60*	60	90*	120 plus sprinklers
Industrial & Storage	60*	90*	120*	
Car parks - closed	30	60	90	
Car parks - open sided	15	15	15	60

* Reduced by 30mins 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 viable way to a satisfactory standard of fire safety in some large and complex buildings and in buildings containing different uses. Fire safety engineering may also be suitable for solving a problem with an aspect of the building design which otherwise follows the provisions of the document". (See pages 28 and 29).

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

- Fire resistance periods are based on building height only
- 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 superstructure and basements is 120 minutes
- Compartment sizes can be doubled in many instances where sprinklers* are installed
- All non-residential buildings over 30m 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).

Height of top storey

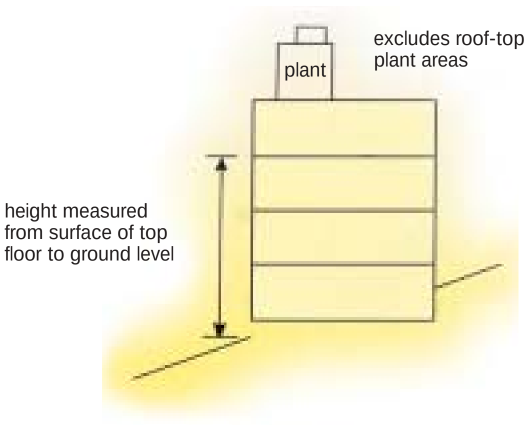


Figure 3
Definition of building height as measured in Approved Document B.

*Sprinkler system means a life safety system

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

Scotland

In Scotland compliance is required with the Technical Standards of the Building Regulations. Approval must be gained before construction takes place; one cannot build at risk.

The Technical Standards underwent major revisions in 2002. The modified documents can be found at www.scotland.gov.uk/build_regs

Fire resistance requirements are contained in Regulation 12 to the Building Standard (Scotland) Regulations 1990 which state that *"every building shall be so constructed that, for a reasonable period, in the event of a fire... its stability is maintained."*

The measures which should be followed to ensure that this regulation is met are contained in Part D2 of the Technical Standards: Structural Fire Precautions (4) (Figure 5). Many of the provisions outlined in Part D are designated as functional standards which contain references to deemed to satisfy standards. These may be descriptive or refer to documents such as British Standards.

The introduction to the Technical Standards contains the following statement:

"Compliance with the Regulations: Regulation 9 stipulates that the requirements of the regulations can be satisfied only by compliance with the relevant standards..... Without prejudice to any other way of meeting the standards, complying with the provisions that are deemed to satisfy the requirements of the regulations, as given in this document, constitutes compliance.

To satisfy the regulations therefore the design, materials and methods of construction must be at least to the standards set in this publication. The provisions deemed to satisfy the standards are provided for the convenience of designers only if they chose to adopt them. There is no obligation to do so, but if used properly deemed to satisfy solutions must be accepted by the local authority."

A relaxation of the requirements given in Technical Standard D is possible where alternative methods of fire protection can be shown to give equivalent levels of safety to those required in the standard. In such situations the local Building Control Officer, often assisted by the Scottish Development Office, may request compensatory features.

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

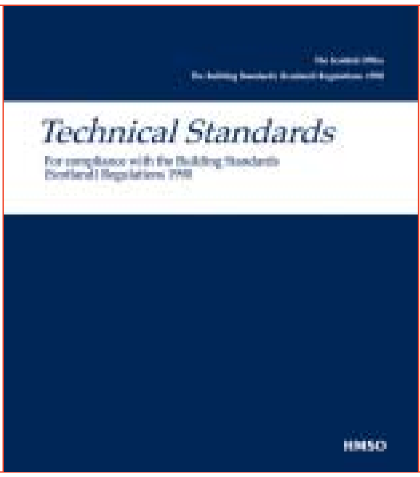


Figure 5
Technical standards D to the Building Standards (Scotland) Regulations, 1990.

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

Figure 7
Technical booklet E to the Northern Ireland building regulations 1994.

Figure 8
Fire safety guide No. 1: Fire safety in section 20 buildings.

Figure 9
HTM 81: Fire precautions in new hospitals.

The most important aspects of the Technical Document 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.
- Where the building or compartment is provided with an approved sprinkler system, allowable compartment sizes can be doubled in most non-residential situations.
- Structural elements of open deck car parks 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

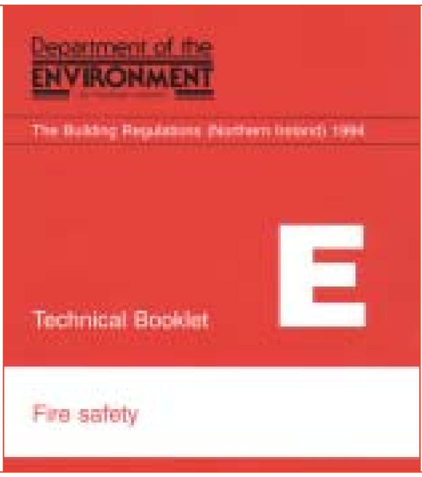
In Northern Ireland new Building Regulations came into force in November 1994. The fire safety requirements for these regulations are supported by Technical Booklet E (5) (Figure 7) which contains provisions regarding structural fire resistance, compartmentation etc. similar to those in the Approved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

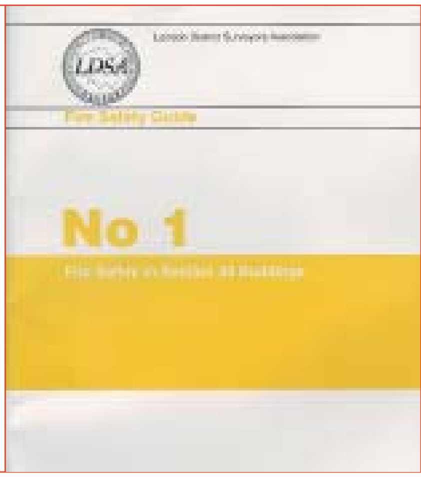
Other sources of information

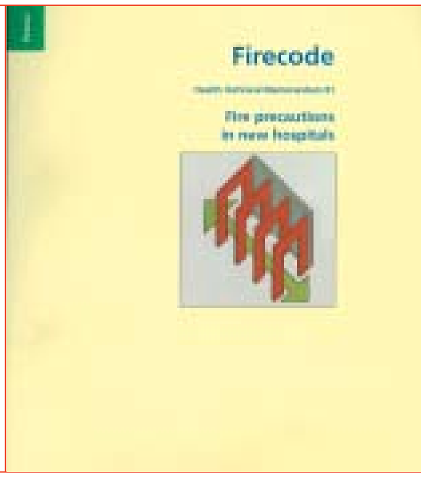
Buildings located within the inner London area are subject to the requirements of the London Building Act 1939. Within this act, precautions against fire in buildings are covered by Section 20. This section ensures that *"proper arrangements will be made and maintained for lessening so far as is reasonably 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. (6) (Figure 8). This document contains detailed information on fire resistance requirements within the Inner London area.

The design of Fire Safety in hospitals is covered by health technical memomanda. For new hospitals the relevant document is HTM 81: Fire precautions in new hospitals. (7) (Figure 9). Approved Document B states that: *"Where the guidance in that document is followed, part B of the regulations will be satisfied"*.







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 (Figure 10 & Figure 11). As only individual sprinkler heads affected by the hot gases from the fire are activated, water damage is minimised. Research has shown that over 90% of fires are suppressed by four sprinkler heads or less.

In Approved Document B to the 1991 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. All non-domestic buildings over 30 metres in height are now required to have sprinklers. 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.

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.

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 12). This publication contains detailed cost examples which indicate that the value of trade-offs in passive fire protection. Larger allowable compartment sizes, reduced number of fire fighting lifts and shafts etc. can 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.
Photograph courtesy of Wormald Ltd.

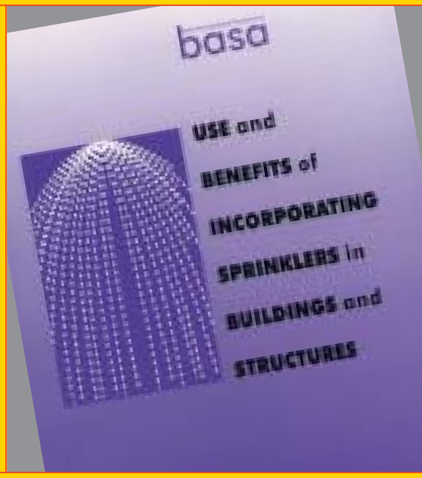


Figure 12
BASA sprinkler publication: Use and benefits of incorporating sprinklers in buildings and structures.

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, which governs the time taken to reach it's failure (or limiting) temperature. This 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 13).

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

An example of this concept is given in Figure 14 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.

Beams supporting 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 $50m^{-1}$

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
b) The cross sectional area of the section - A

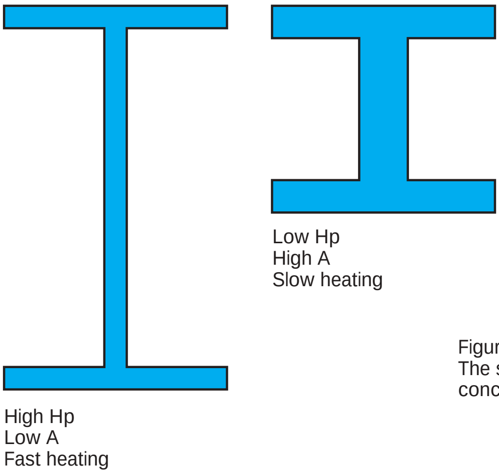


Figure 13
The section factor concept

Unprotected beams - Design temperature

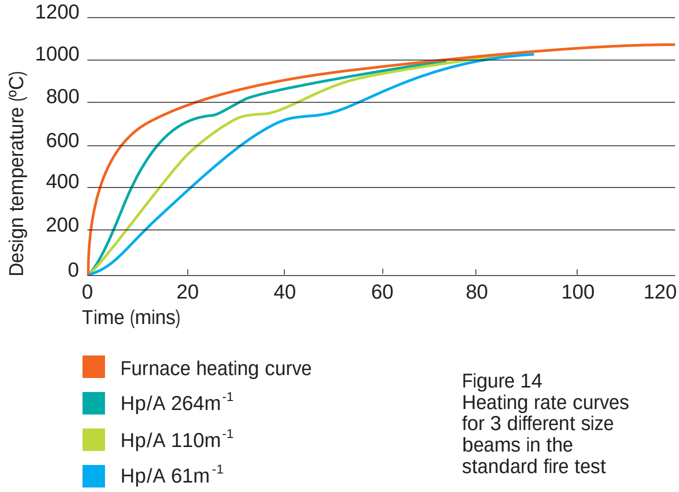


Figure 14
Heating rate curves for 3 different size beams in the standard fire test

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 15) 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.

For typical building construction using universal I and H sections, the value of Hp/A is usually in the range 40-305m^{-1}, the value of 40m^{-1} being associated with the heavy 305 x 305 x 198 kg/m column for three sided box protection (eg. boards), whilst the light 152 x 152 x 23 kg/m column has a Hp/A value of 305 for four sided profile protection (eg. sprayed coatings). In published tables, values of Hp/A are normally rounded to the nearest 5 units.

Figure 13 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 supplied in the Yellow Book, (Figure 17) for a typical product of this type, which shows the required thickness to be 16mm.

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

Figure 16
The four most common protection configurations for calculation of Hp/A

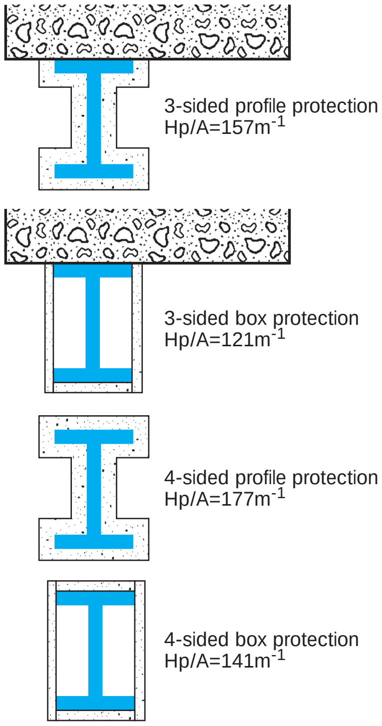


Figure 17
Extract from 'Yellow Book' as it applies to a typical spray fire protection material

Hp/A	**Dry thickness in mm to provide fire resistance of**					
Up to	**1/2hr**	**1hr**	**1.5hr**	**2hr**	**3hr**	**4hr**
30	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		44	

Figure 15
The 'Yellow Book'

Castellated and cellular beams

For castellated or cellular beams or fabricated beams with holes the thickness of the fire protection material should be 1.2 times 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.

At the time of writing, some doubt exists as to whether the 20% factor is correct when applying intumescent coatings. The advice of the intumescent manufacturer should be sought.

Hot rolled unfilled hollow sections

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, $Hp/A < 250m^{-1}$
Thickness = $t (1 + (Hp/A)/1000)$.

For a section factor, $Hp/A > 250m^{-1}$
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. Some suppliers do clearly differentiate between open (H & I) and closed (hollow) sections in their specifications, others do not.

Concrete filled hollow sections are discussed on page 23.

Traditional fire protection materials

For materials such as 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 (Figure 19). (10)

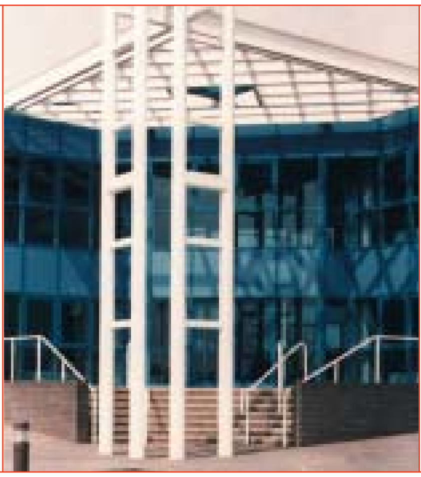


Figure 18
Intumescent coated hollow sections
Photograph courtesy of Carboline Ltd.

Figure 19
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 (Figure 20) are 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.

Sprays

Spray protection systems (Figure 21) have decreased in popularity in the past decade, despite being the cheapest 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 unpainted 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 significantly higher than that assumed using the application costs only.

Thin film intumescent coatings

Intumescent coatings (Figure 22) 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.

Figure 20
Board Protection Systems
Photograph courtesy of Promat Ltd.

Figure 21
Spray Protection System.

Figure 22
Thin Film Intumescent Coating System.

Figure 23
Flexible Blanket Protection System.

Figure 24
Concrete Encasement.

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 and generally comparable with board systems.
Application - is a wet trade which requires suitable atmosphere 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.

Flexible/Blanket systems

Flexible fire protection systems (Figure 23) 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 1970's concrete was by far the most common form of fire protection for structural steelwork (Figure 24). 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 reference 10.

5. Off-site fire protection

Thin film intumescent coatings

Intumescent coatings are described on pages 12 and 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.

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 a 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 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. A second edition is due in early 2004. (Figure 26). (12)

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

Figure 25
Manual application of Off-Site Intumescent Coatings.

Off-site applied spray materials

The use of off-site fire protection to the building industry is not restricted to thin film intumescent coatings. Spray protection materials have also been used although this is generally restricted to the petrochemical and chemical process industries. In these conditions fires are much more onerous than those encountered in most other form of construction and specifications are required of fire protection which cannot generally be met by thin film intumescent coatings. Off-site applied spray protection materials have also been utilised occasionally in standard construction use. (Figure 27).

Figure 26
Design Guidance and Model Specification for use with Off-Site Applied Thin Film Intumescent Coatings.

Figure 27
Off-Site Application of Spray Protection Material. Photograph courtesy of Cafco International.

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 28.

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 550°C structural steel will retain 60% of its room temperature strength (see Figure 29). 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 550°C 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 550°C but varies according to two factors, the temperature profile and the load.

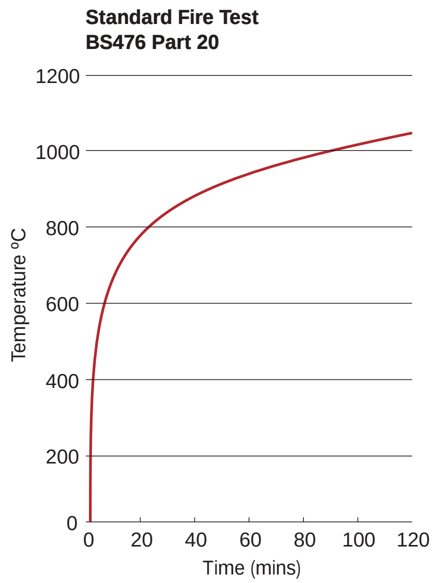


Figure 28
BS476 Part 20 Standard Time-Temperature Relationship for Fire Tests.

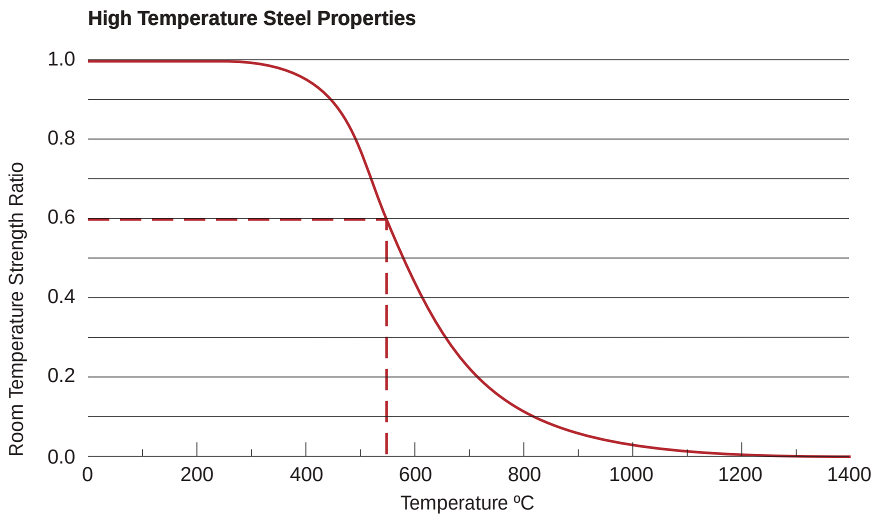


Figure 29
Steel Strength Decreases with Temperature.

Effect of temperature profile

A joint test programme by Corus and the Fire Research Station 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 29 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 concrete floor slab and 60% of its cold load bearing capacity will become plastic at about 620ºC. 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. 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 concrete slabs will not occur until the steel reaches 750ºC, an increase of 130ºC on the limiting temperature in the fully loaded case.

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 30) (14) was published in August 1990 and brings together in one document all 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 (See page 16) it may also be used in fire engineering assessments when natural fire temperatures are derived by calculation (page 28).

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 31).

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 pages 9 and 10).

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 is illustrated graphically. (Figure 32). If the limiting temperature exceeds the design temperature no protection is necessary (see page 9).

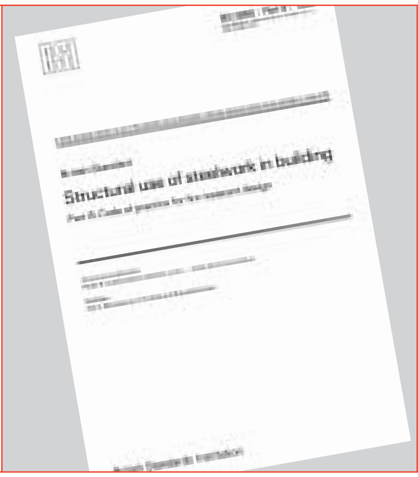


Figure 30
BS5950 Part 8, Code of Practice for Fire Resistant Design.

Figure 31
Fire Resistant Design of Steel Structures: A Handbook to BS5950 Part 8

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 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 20).

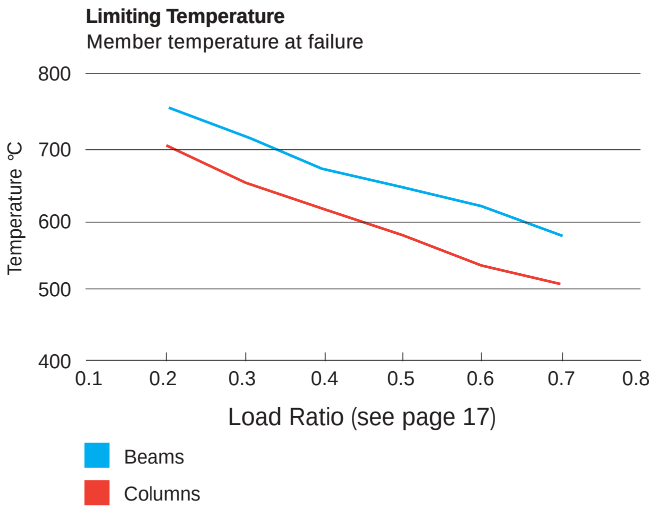


Figure 32

8. 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 pages 9 and 10).

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

Block-infilled columns - (Figure 33) 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 34) 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 35) 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 33
Block-Infilled Column.

Figure 34
Web-Infilled Column Design Guide.

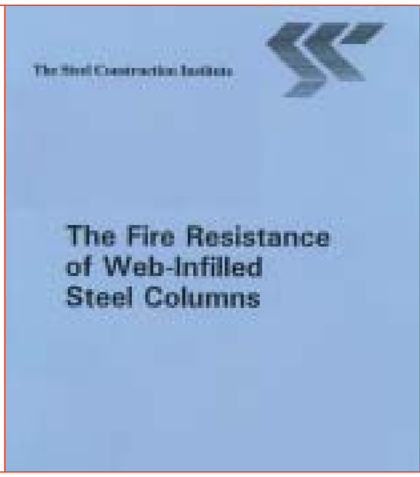
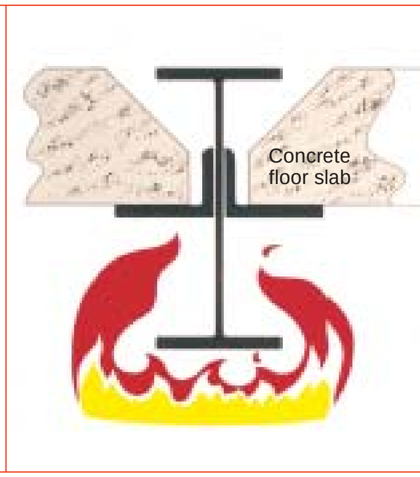


Figure 35
Shelf angle floor beam.



Slim floor beams - (Figure 36 & Figure 37)
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 used deep decking but removes the support plate by using an asymmetric beam (Figure 38). 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.

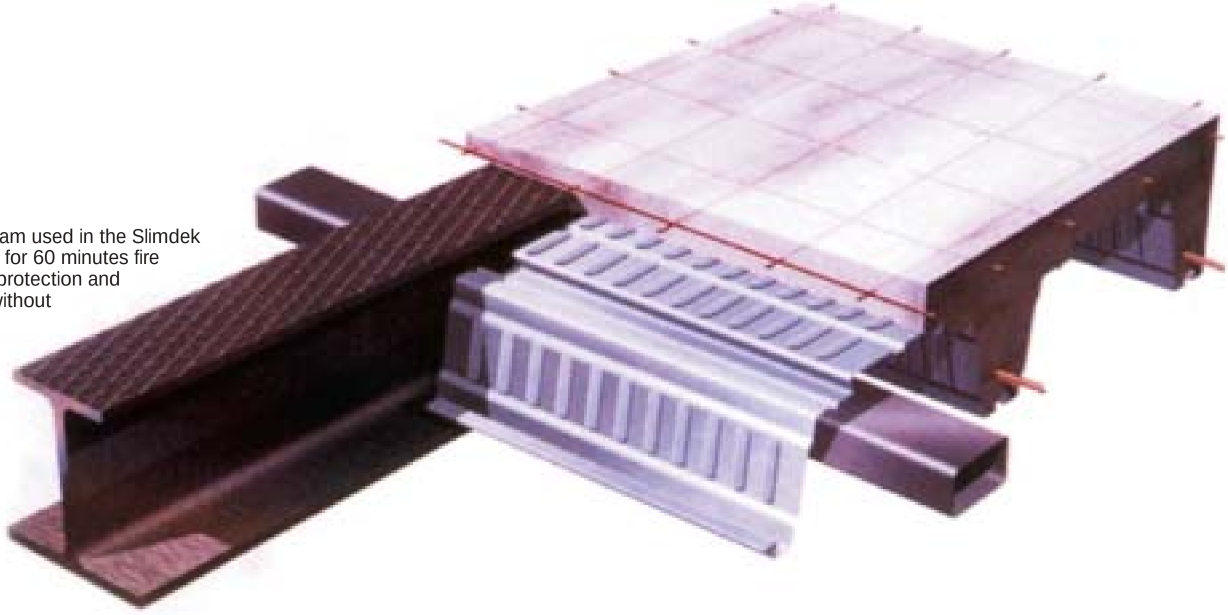


Figure 38
The asymmetric beam used in the Slimdek system is designed for 60 minutes fire resistance without protection and composite action without welded studs.

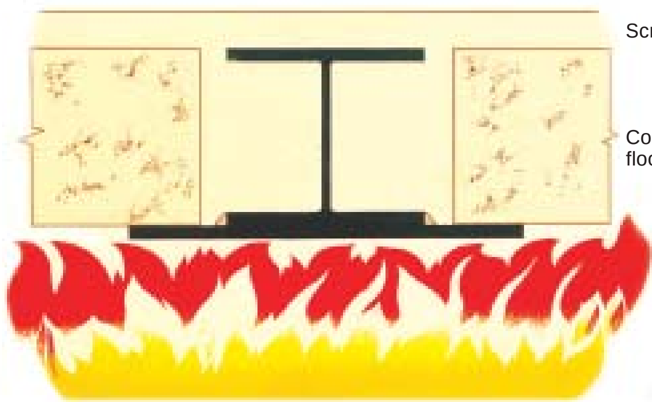


Figure 36
SLIMFLOR with precast slab.

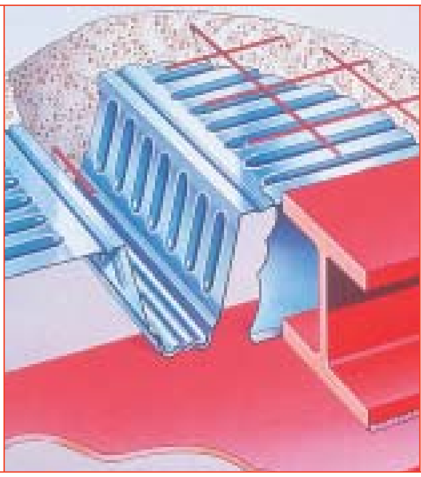


Figure 37
Deep Deck SLIMFLOR System.

9. Combining fire resistant design methods

The innovative design solutions for beams and columns described in Figure 39 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 No 189 - Design of Steel Framed Buildings Without Applied Fire Protection. (23) (Figure 40).

Figure 39
Fire resistance that can economically be obtained for various structural forms.

	Beam type				
Column type	Unprotected beam	Slim floor 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 sections	15	60	60	>60	>60
Protected column	15	60	60	>60	>60

Note: For England and Wales, in accordance with Approved Document B, the requirement for 60 minutes fire resistance may often be reduced to 30 minutes if sprinklers are installed.

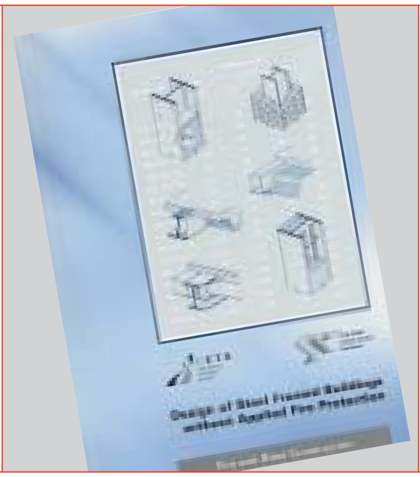


Figure 40
Design of steel framed buildings without applied fire protection.

10. 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 hollow sections, and square and rectangular sections is given in ENV1994 1.2 (24) and CIDECT design guide (25), (Figure 41).

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 moments. As a result, BS5950 Part 8 section 4.6.2.1 requires that two relationships which limit the moments about the major and minor axes must both be met when using plain or fibre reinforced concrete. Compliance with both will 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 4.6.2.2.

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 4.6.3. Similar reductions are also possible with an intumescent coating. However, each individual product must be assessed separately to ascertain these allowable reductions. At present, only one product has been fully evaluated. Further information is available in the Corus Tubes publication Intumescent Coatings and SHS concrete filled columns (Figure 42). (26).

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 BS and for Eurocode Designs, and the Design guide for SHS concrete filled columns, CT26 (27). These publications are available in a scanned version on a CD from Corus Tubes.

Freely available fire design software to Eurocode 4 has now been developed which also includes moments. A copy is available on the new Corus Construction Centre CD. Further information can be obtained from Corus Tubes, Corby, Northants (Telephone 0500 123133).

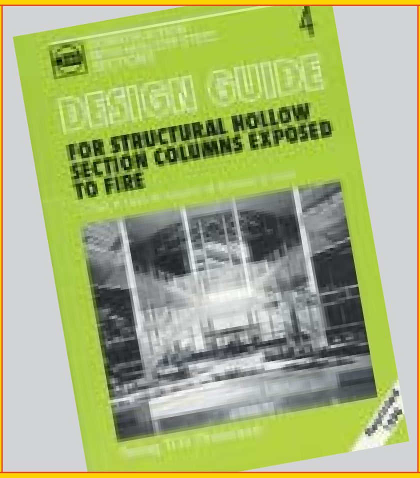


Figure 41
CIDECT Design Guide no. 4.

Figure 42
Intumescent coatings and SHS concrete filled columns.

11. Single storey buildings in fire

In the UK, single storey buildings do not normally require fire protection (Approved Document B, Section 8.4 which excludes from the definition of elements of structure, structure which 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 non-domestic 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 the requirement for adequate space separation between buildings as outlined in Part B of Schedule 1 of the Building Regulations 1991:
"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 may be left unprotected but the stanchion base 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 (Figure 43) (28).

This document is more comprehensive than any of its predecessors and contains guidance not just on plain portal frames but also on portal frames with lean-to structures, two storey sections etc. as well as guidance on 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:
"In England & Wales, the boundary distance for a building with sprinklers may be half that required for a building without sprinklers, or alternatively the unprotected area in the boundary wall can be doubled… the recommendations (of the SCI document) to design the foundation to resist the overturning moment from the collapse of the roof need not be followed.
In Scotland, the Technical Standards do not state explicitly that fitting sprinklers removes the need to design the column foundations to resist overturning. Although the acceptance of the SCI document indicates that this is considered to be a reasonable approach, it is up to the local authorities to grant regulations on an individual basis.
In Northern Ireland, no explicit statement is made as to the need to design the foundations to resist overturning moments when the building is fitted with a sprinkler system.

Figure 43
Single Storey Steel Framed Building in Fire Boundary Conditions.

12. External steelwork

A number of modern steel buildings have being constructed with the steel skeleton on the outside of the structure (Figure 44 & Figure 45). 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 allow the frame members to remain unprotected 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 46). This describes a method to define the design temperature (see page 18) 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.

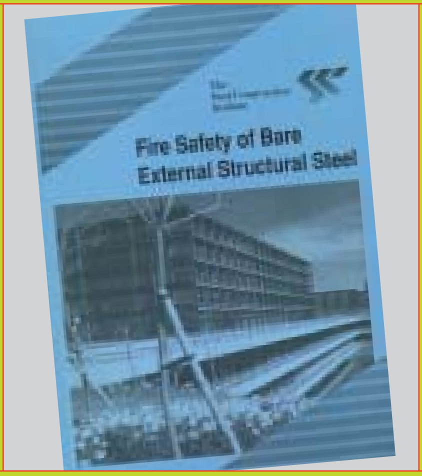
Comparison of the calculated design temperature with the limiting temperature of members calculated from BS5950 Part 8 (see page 18) will indicate whether or not protection is necessary.

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 44
Wills Factory, Bristol.

Figure 45
Hotel de las Arte, Barcelona.

Figure 46
Fire Safety of Bare External Structural Steel.



13. Composite steel deck floors in fire

Assessment of composite slabs

A composite steel deck floor 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 improving 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 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 47). 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. Typically the use of the 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.

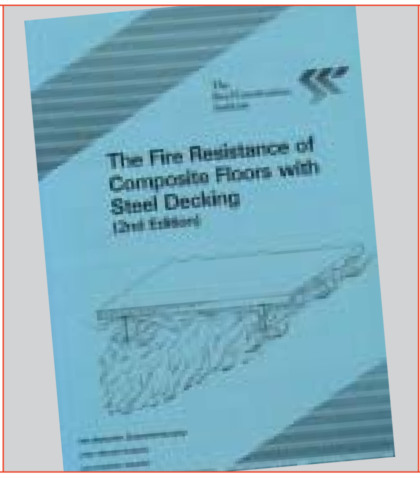


Figure 47
The Fire Resistance of Composite Floors with Steel Decking.

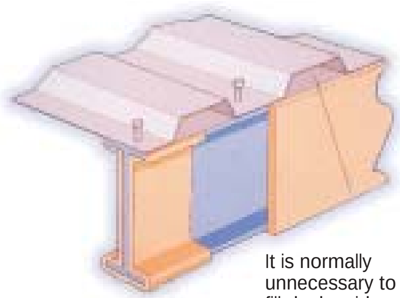
Deck voids

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 48) Details are given in the Steel Construction Institute publication The Fire Resistance of Composite Floors with Steel Decking. (Figure 47) (31)

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.

Composite Beams with spray, intumescent or board protection



It is normally unnecessary to fill deck voids for up to 1.5 hrs fire resistance.

Figure 48
Composite Steel Deck Floor with Unfilled Voids.

14. 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 the Approved Documents, Technical Standards and Technical Booklets, (see pages 4-7). Recognition of this, and also increased knowledge of how real structures behave in fire, made possible by a wide ranging and intensive program 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 to the Building Regulations for England and Wales, 1991 states that:

"Fire safety engineering can provide an alternative approach to fire safety. It may be the only viable way to a satisfactory standard of fire safety in some large and complex buildings and in buildings containing different uses. Fire safety engineering may also be suitable for solving a problem with an aspect of the building design which otherwise follows the provisions in this document."

Fire safety engineering can be seen as an integrated package of measures designed to achieve the maximum benefit from the available methods for preventing, controlling or limiting the consequences of fire. In terms of structural stability, fire safety engineering is aimed at adopting a rational scientific approach which ensures that fire resistance/protection is provided where it is needed and expense is not incurred needlessly in giving an illusion of safety. This is achieved by the following three stage process :-

1) Predicting the heating rate and maximum temperature of the atmosphere inside the compartment.
This involves assessing the fire load (the quantity and type of combustible material) in the compartment, the ventilation available and the thermal characteristics of the compartment linings. For example when the walls are insulated, the fire temperature will be higher than if they were constructed from a non-insulated material.

The draft Eurocode ENV 1991-1-2 (32) gives a method to define the time equivalent (the time in a standard BS476 fire that would have the same effect as the natural fire in the compartment under consideration) as:-

$$te = q.k.w$$

Where:-
q = the fire load in Kg of wood /m²
k = a conversion factor relating to the thermal characteristics of the compartment linings
w = a factor taking into account the degree of ventilation

This is one of a number of time equivalent equations available.

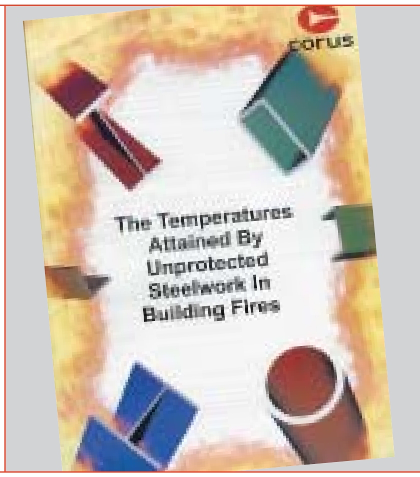
2) Predicting the temperature of the steel member.
This depends on the location, the section factor and any protection applied. The Corus publication, Temperatures Attained By Unprotected Steelwork in Building Fires may be used. (Figure 49) (33).

Figure 49
Temperatures attained by unprotected steelwork in building fires.

Figure 50
Twickenham North Stand.

Figure 51
Windsor Park.

Figure 52
Stansted Airport.



3) Predicting the stability of the structure.

The stability of the member depends not only on the temperature it reaches during the fire but also on the applied load and the effects of any composite action, restraint and continuity from the remainder of the structure.

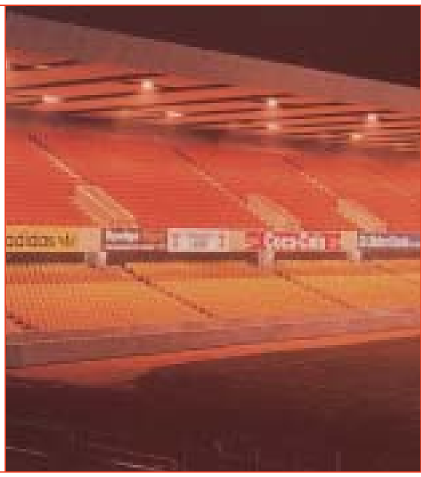
Consideration of these factors permits fire severity, heating rates and the stability of the steel structure to be predicted. Consequently, protection requirements can be specified to meet the fire hazard. This design concept proves most cost effective when it can be shown that the structure, or parts of the structure, has sufficient inherent fire resistance to the temperatures generated by the fire load to avoid the need to apply any fire protection.

Typical of the situations 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 stands can no longer be described as simple bare steel, concrete and blockwork structures for the sole purpose of watching sport. Instead, they are buildings for mixed occupancies, often containing shops, restaurants and gymnasia, which creates difficulties in developing fire safety policies consistent with those laid out in the Approved Documents.
A solution can often be found for such situations using fire engineering.

Fire engineering is not restricted to large prestige buildings. The method is also widely used for partial analysis and for smaller buildings, in particular where the cost of structural fire protection per unit floor area is high.

In addition, a full fire engineering assessment is not always needed. Small changes which can be introduced at the design stage can often be effective in simplifying the process of meeting the fire resistance requirements and reducing costs. Examples of this would be using a slightly larger member than required from cold design to reduce the load ratio (see page 17) or block filling of columns rather than using standard fire protection materials.
Structures designed using fire engineering include sports stadia (Figure 50 & Figure 51), office and industrial buildings, atria, airport terminals (Figure 52), leisure centres, hospitals, shopping centres and car parks.

Detailed guidance on the process for carrying out a Fire Engineering Assessment are now contained in BS7974 (34).



15. Cardington fire tests and Level 1 Design Guidance

Between 1994 and 1996 a series of six fire tests were carried out on an eight storey composite building with metal deck floors at the Building Research Establishment test facility at Cardington in Bedfordshire. The test programme was divided into two parts; the first, compromising a single beam test and three large compartment tests was sponsored by the European Coal and Steel Community and Corus Plc. and was carried out by the Corus Technology Centre. A complementary programme, compromising two compartment tests, was sponsored by the Department of the Environment, Transport & the Regions and was carried out by the Building Research Establishment.

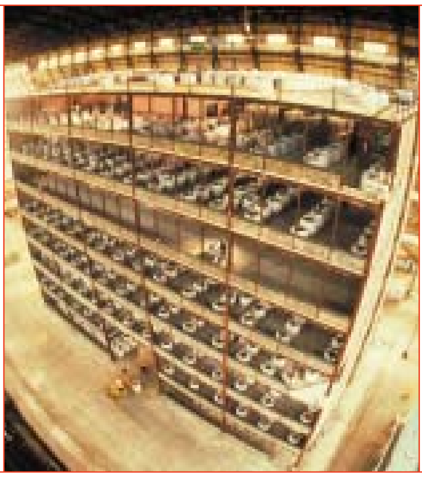
Cardington fire tests

The tests were carried out to determine if the fire performance of real buildings of this type is better than is suggested by tests on individual elements of construction. Evidence that this is the case had been provided by studies of actual fires in real buildings (35), tests carried out by BHP in Melbourne 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.

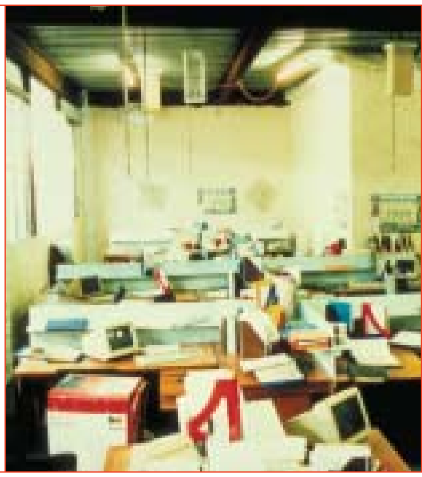
In order to determine a direct comparison, the first test was carried out on a single unprotected beam and surrounding area of slab. The results demonstrated that a failure deflection (normally considered to be Length/30) would have occurred at approximately 1000°C, far greater than the temperature of 700°C at which the beam 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 loading provided by gas, wooden cribs or standard office furniture. Columns were protected but beams were not. However, despite atmosphere temperatures of almost 1200°C and steel temperatures on the unprotected beams in excess of 1100°C in the worst cases, no structural collapse took place.

Figure 53
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 Level 1 Design Guidance applies only to buildings of this type.



Figures 54, 55, 56
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

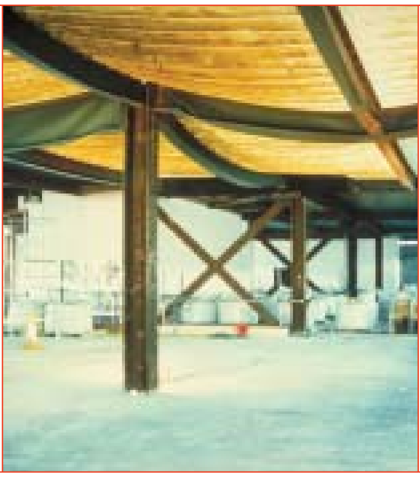
Observations from the Cardington fire tests and other large building fires have shown that the behaviour of the composite floor slab plays a crucial role in providing enhanced fire resistance. Where significant numbers of beams are not protected, this has the effect of greatly increasing the distance which the floor slab spans in the fire condition. The Cardington tests demonstrated that, in these conditions, the slab acts as a membrane supported by cold perimeter beams and protected columns. As the unprotected steel beams lose their load carrying capacity, the composite slabs 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. In the case of simply supported edges, the supports will not anchor these tensile forces and a compressive ring will form around the edge of the slab. Failure will only occur at large displacements with fracture of the reinforcement.

Level 1 Design Guidance

The Building Research Establishment has developed a simple structural model which combines the residual strength of the steel composite beams with the slab strength calculated using a combined yield line and membrane action model designed 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 were published in September 2000 in Fire Safe Design: A New Approach to Multi-storey Steel Framed Buildings. (37). (Figure 58). Use of these tables allow the designer to leave large numbers of secondary beams unprotected in buildings requiring 30 and 60 minutes fire resistance although some compensation features, such as increased mesh size and density, may be required. The publication also contains design examples and considerable information of the background to the tests. The recommendations of the guidance can be seen as extending the fire engineering approach outlined on pages 28 & 29. It is intended that designs carried out in accordance with these recommendations will achieve at least the levels of safety required by regulations.

Of necessity, design tables are restricted in the range of loads and spans which can be addressed. To increase the scope, the programme to generate the tables has been made available on www.steel-sci.org/it/software/fire

The process of creating design tables has resulted in some simplifications. Use of the BRE calculation method from first principles may lead to additional economies. The BRE calculation method may be used for fire resistance periods of up to 120 minutes.



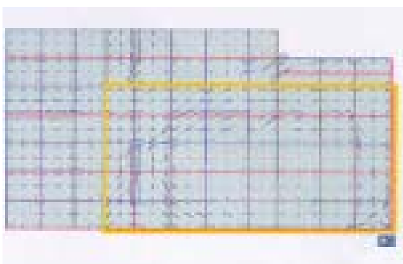


Figure 57
Computerised simulation of the development of tensile membrane action in a corner fire test at 900°C. The magnitude and direction of the blue lines indicate compressive forces, those of red lines indicate tensile forces.

Figure 58
Fire Safe Design: A new approach to multi-storey steel framed buildings.

16. 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 whose 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 in the Corus Publication ‘The Reinstatement of Fire Damaged Steel and Iron Framed Structures (38) (Figure 59). It’s main conclusions are summarised here.

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 300ºC and increases rapidly after 400ºC, by 550ºC 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 750ºC 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 it’s temperature above 600ºC, 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 600ºC 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 600ºC, may be proportionately greater than for unalloyed mild steels.

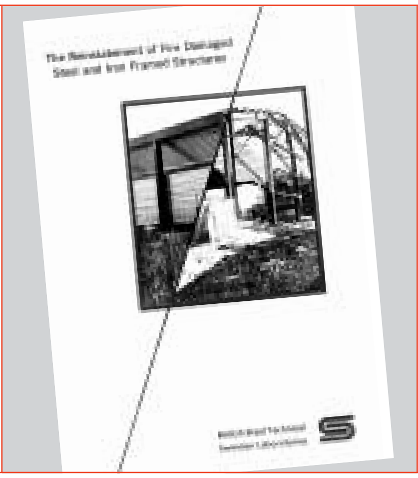


Figure 59
Reinstatement of fire damaged steel and iron framed structures.

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 steel 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 600ºC, would 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 instance, 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.

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 450ºC. The residual strength of these bolts falls to 80% and 60% after reaching temperatures of 600ºC and 800ºC 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.

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

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

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www.corusgroup.com

Corus Construction & Industrial
PO Box 1
Brigg Road
Scunthorpe
North Lincolnshire
DN16 1BP
T: 01724 404040
F: 01724 405600

English language version

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