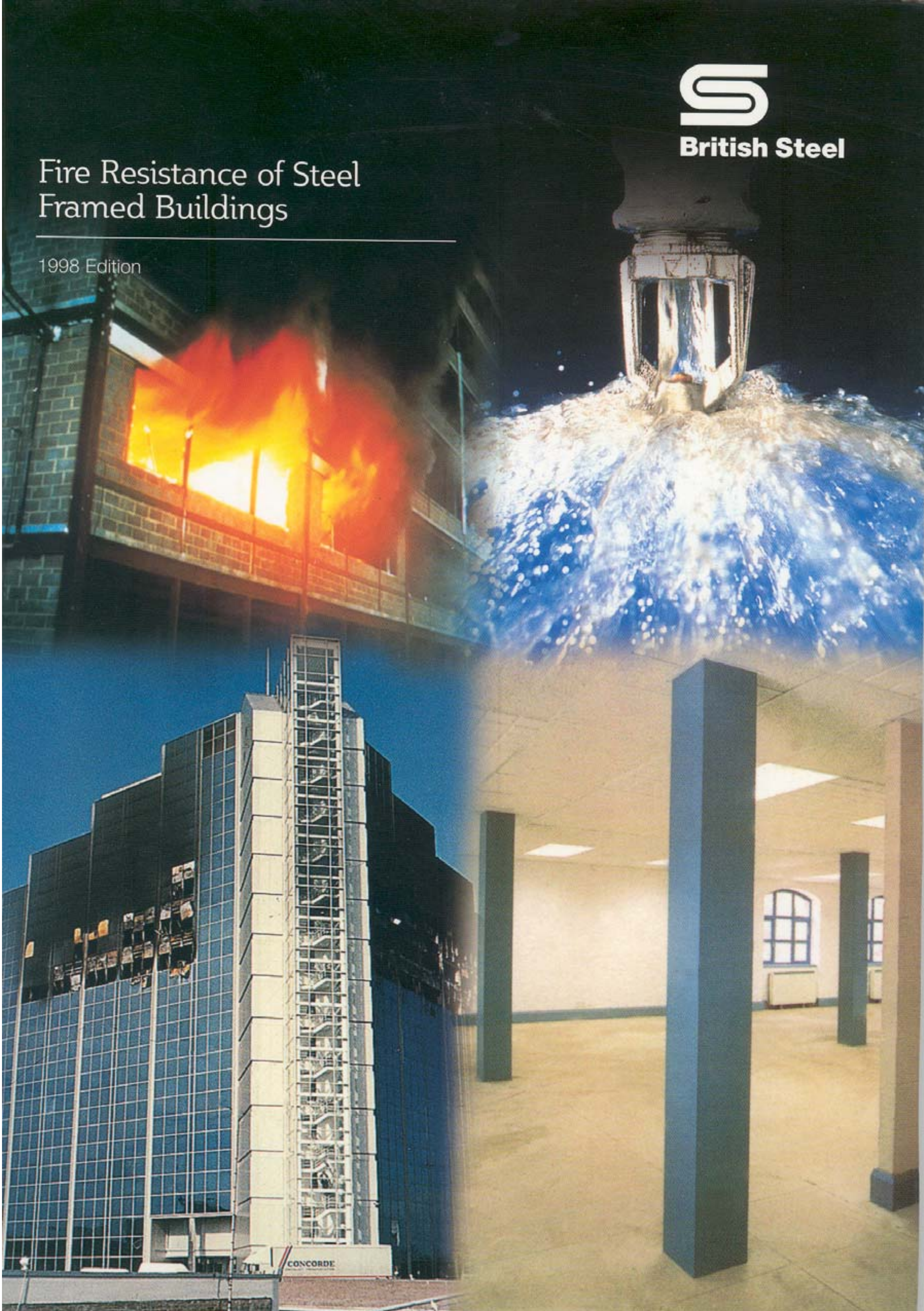
British Steel

Fire Resistance of Steel Framed Buildings

1998 Edition



Foreword

The 1980's and '90's have been a time of rapid change in the field of fire and steel construction. It has been a period during which new thinking and research conducted over many years have been increasingly put into practice. The Approved Document approach to satisfying regulation requirements in England & Wales in the mid 1980's began a process of recognition of modern practice that has continued into the '90's with the introduction of the structural codes for fire resistant design embodied in BS5950 part 8, the draft Eurocodes 1991, 1993 and 1994 and will stretch into the future with the proposed British Standard on Fire Safety Engineering in Buildings.

The results of the research are so far reaching that 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 18). The pace of change is likely to increase into the next century as methods are developed to allow design for fire to move away from consideration only of simple elements towards whole building behaviour in fire.

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

Jef Robinson
Chairman of BS5950 Part 8 Drafting Panel.
Chairman of the Steel Construction Industry Fire Sector Committee.

How to use this book

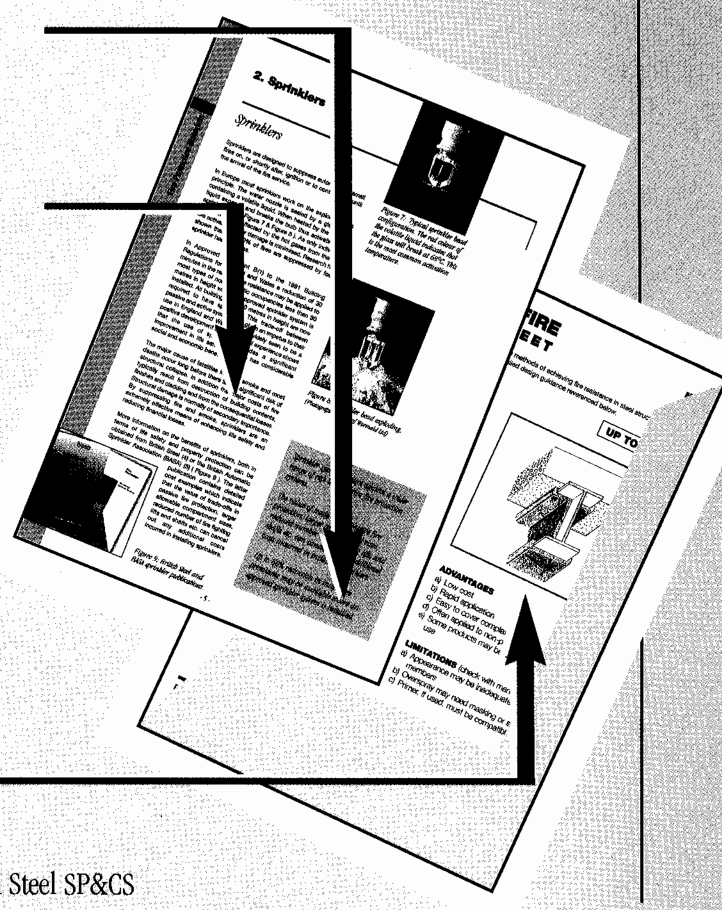
This publication is intended to be used on a number of levels, both as an introduction to the subject of fire resistance of structural steelwork and as a reference for practising designers.

To get a flavour of the book just flick through the pages and read the blue highlight windows in the bottom right hand corners. This is a quick summary of the main points of that page.

For quick reference the main text is arranged in logical sections. It gives a brief but comprehensive description of all of the currently available methods of achieving fire resistance in steel structures. It can be read from cover to cover or used selectively whenever specific information is required.

For more detailed information the book contains references throughout to source documents which deal in greater depth with the subjects in question.

To see what options are available to achieve a specific fire resistance period go to the information sheets in the back pocket.



Document prepared by John Dowling & Jef Robinson, British Steel SP&CS

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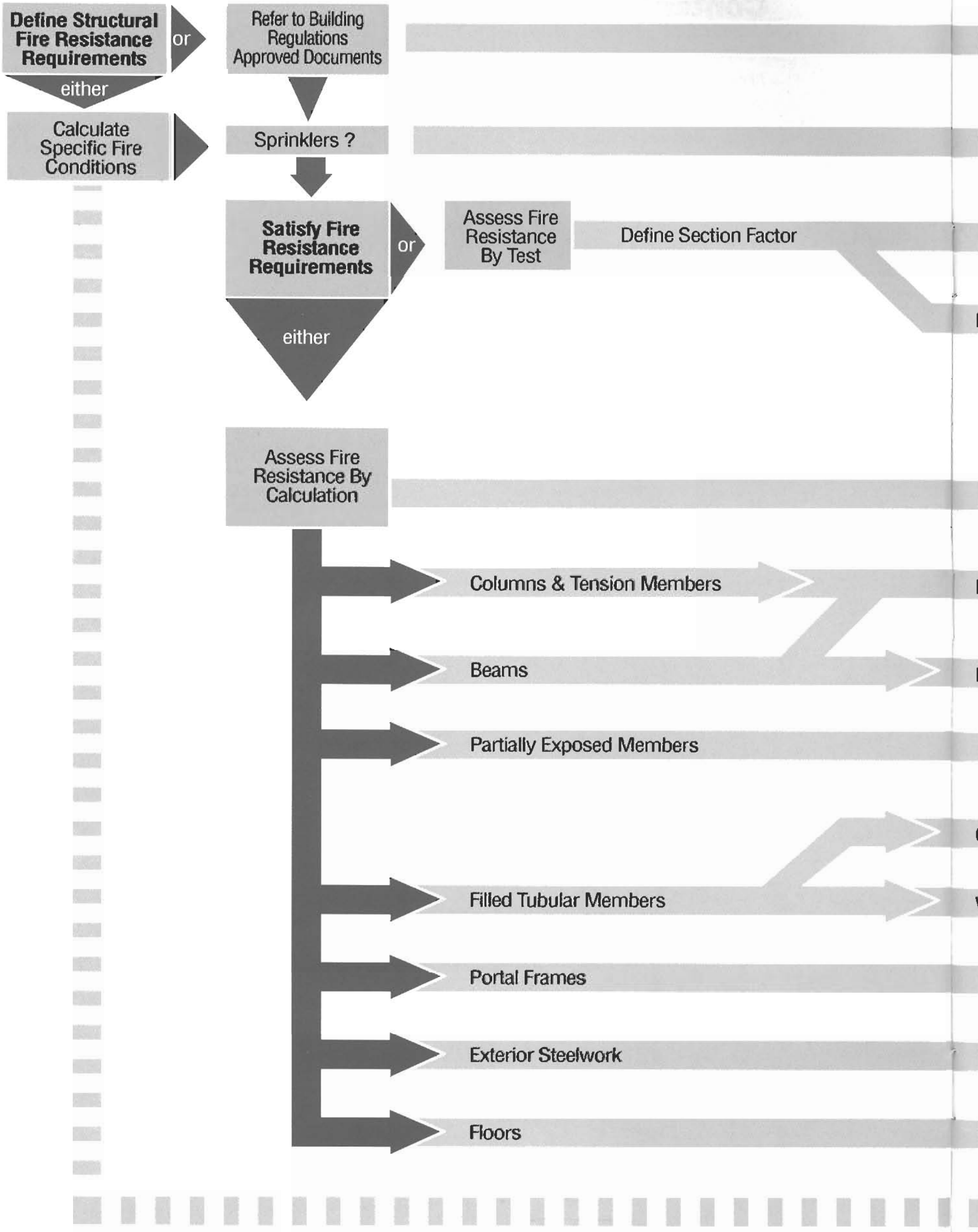

Define Structural Fire Resistance Requirements
or
Refer to Building Regulations Approved Documents
either
Calculate Specific Fire Conditions
Sprinklers ?
Satisfy Fire Resistance Requirements
or
Assess Fire Resistance By Test
Define Section Factor
either
Assess Fire Resistance By Calculation
Columns & Tension Members
Beams
Partially Exposed Members
Filled Tubular Members
Portal Frames
Exterior Steelwork
Floors

1. The Building Regulations and Structural Fire Resistance

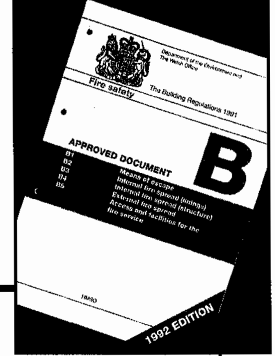


Figure 1: Approved Document B to the Building Regulations for England & Wales, 1991.

1.1 England & 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 1991 (1) (Figure 1) 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.

Fire Resistance	Height of top storey - metres			
England & Wales Recommendations 1992	<5	<20	<30	>30
Approx no. of Storeys	2	5/6	8/9	9+
Residential (non domestic)	30	60	90	120
Offices	30	60*	90*	120 plus sprinklers (floors 90 mins)
Shops, Commercial Assembly	60*	60	90*	
Industrial & Storage	60*	90*	120*	
Car Parks - Closed	30	60	90	
Car Parks - Open sided	15	15	15	60

* Reduced by 30 mins when sprinkled

Table 1 Summary of Structural Fire Resistance Requirements form 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 also 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. However, should a contravention of a requirement be alleged then, if you have followed the guidance in the relevant Approved Documents, that will be evidence tending to show that you have complied with the regulations. If you have not followed the guidance then that will be evidence tending to show that you have not complied. It will then be for you to demonstrate by other means that you have satisfied the requirement".

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

"A fire safety engineering approach that takes into account the total fire safety package can provide an alternative approach to fire safety. It may be the only viable way to achieve a satisfactory standard of fire safety in some large and complex buildings".

The Approved Document lists the parameters that should be included in such a fire safety study.

Approved Document B allows alternative ways of satisfying structural stability requirements in fire.

1. Compliance with Table A2

2. By a Fire Safety Engineering approach.

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

- ❑ **Fire resistance periods are based on 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 2)
- ❑ **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 have 15 minutes inherent fire resistance and thus most steel framed open deck car parks do not now require structural fire protection.** (see Figure 3)

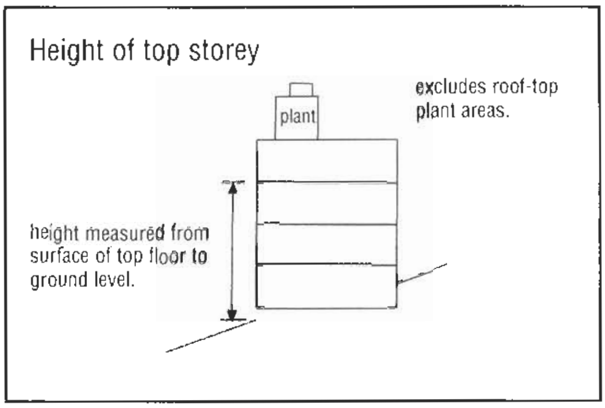


Figure 2: Definition of Building Height as measured in Approved Document B.

Figure 3: Steel in open deck car parks is now predominantly unprotected.

Structural fire resistance periods are based on building height, where height is measured to the floor of the top storey.

Sprinklers are mandatory in non-residential buildings over 30 metres.

In most non-residential buildings below 30 meters in height, fire ratings are reduced by 30 minutes when sprinklers are installed.

Allowable compartment floor areas can be doubled in many cases when sprinklers are installed.

1. The Building Regulations and Structural Fire Resistance ~ *continued*

1.2 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.

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 (2) (Figure 4). Many of the provisions outlined in Part D are designated as functional standards which contain references 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 sets out three ways by which the requirements of the Regulations can be satisfied:

1. by compliance with the relevant standards set out in the supporting Technical Standards; or

2. by conforming with provisions which are stated in the Technical Standards to be deemed to satisfy the relevant standards; or

3. by any other means which can be shown to satisfy the relevant standards."

The third of these statements is taken to mean that it is not necessary to follow the requirements of the technical standard if it can be proven that an alternative method meets the provision of the functional standard.

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 stands at the Glasgow Celtic football ground in Parkhead. (Figure 5)

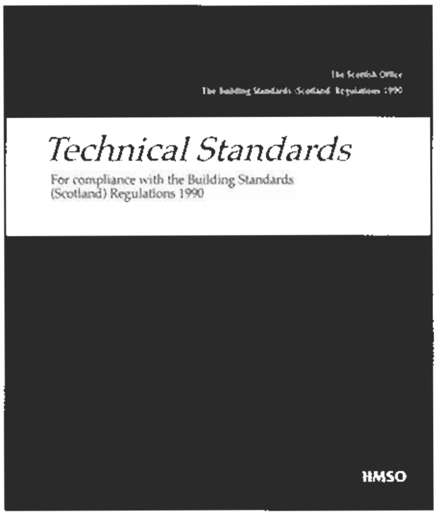


Figure 4: Technical Standards D to the Building Standards (Scotland) Regulations, 1990.

In Scotland approval must be gained before building. One cannot build at risk.

A Distinction is made between low and high hazard in many occupancies with consequent changes in fire resistance requirements.

Scottish fire resistance requirements are based on a combination of height, floor area and cubic capacity.

Where the building or compartment is produced with an approved sprinkler system, allowable compartment sizes can be doubled in most non-residential cases.

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

- **The maximum period of fire resistance required is 240 minutes for certain types of high hazard storage buildings.**
- **Fire resistance requirements are based on a combination of building height, floor area and cubic capacity of a building or compartment.**
- **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).**

Figure 5: New Stand Glasgow Celtic Football Stadium, Parkhead, Glasgow.

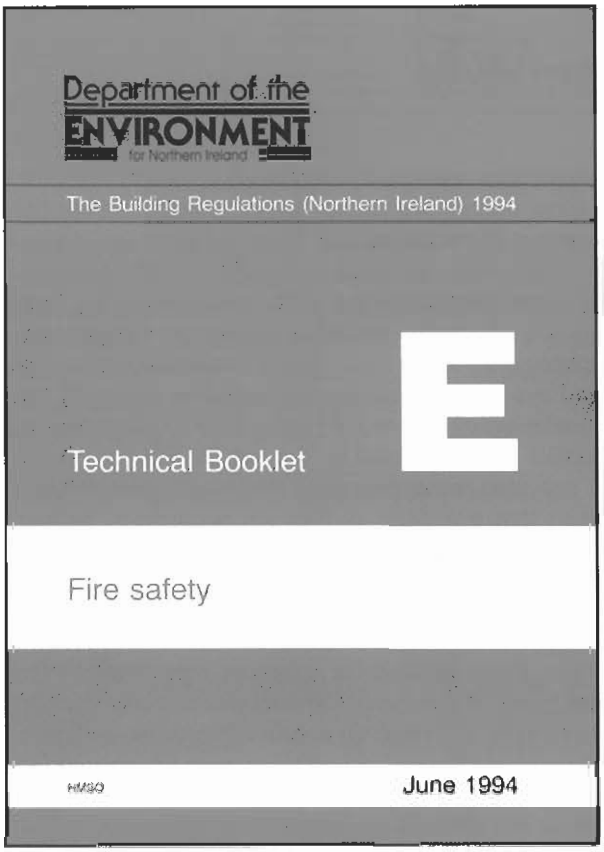


Figure 6: Technical Booklet E to the Northern Ireland Buildings Regulations 1994.

1.3 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 (3) (Figure 6) 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 use of which is regarded as evidence tending to show that the requirements of the Building Regulations have been met, the provisions of Technical Booklet E are deemed to satisfy those requirements. 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.

Compliance with the provisions of Technical Booklet E for Northern Ireland is deemed to satisfy the requirements of the Buildings Regulations.

In all other aspects it is closely modelled on Approved Document B for England and Wales.

2. Sprinklers

Figure 7: 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.

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 7 & Figure 8). 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(1) 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 sprinkler system is installed. All 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 and 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.

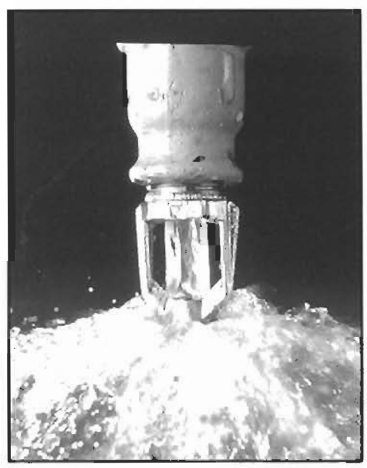


Figure 8: Sprinkler head exploding.
(Photograph courtesy of Wormald Ltd)

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 British Steel (4) or the British Automatic Sprinkler Association (BASA) (5) (Figure 9). The latter 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.

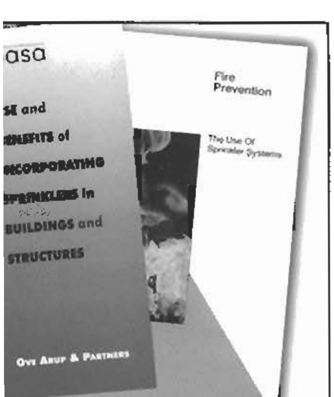


Figure 9: British Steel and BASA sprinkler publications.

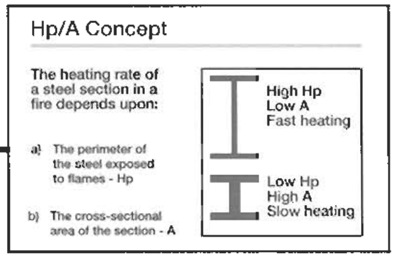
Sprinkler systems protect against a wider range of risk than passive fire protection systems.

The value of trade-offs in passive fire protection, larger compartment sizes, reduced number of fire fighting lifts and shafts etc. can cancel out any additional costs incurred in installing sprinklers.

Up to 60% reduction in insurance premiums may be available where an approved sprinkler system is installed.

3. Section Factor and Protection Thickness Assessment

Figure 10: The Section Factor Concept.



3.1 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 10)

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

An example of this concept is given in Figure 11 which shows the heating rate for three unprotected beams when subjected to the standard fire test. (see Section 6.1)

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 Section 6.2) 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.

For typical building construction using universal I and H sections the value of Hp/A is usually in the range 40 - $300m^{-1}$, 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 300 for four sided profile protection (eg. sprayed coatings). In published tables, values of Hp/A are normally rounded to the nearest 5 units.

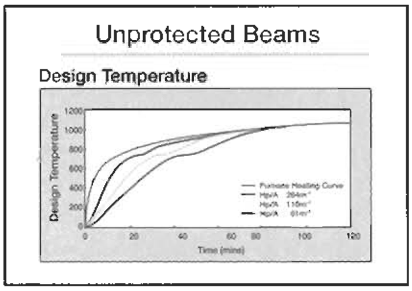


Figure 11: Heating rate curves for 3 different size beams in the Standard Fire Test.

3.2 Hot Rolled H and I Sections

When proprietary passive fire protection is necessary, 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" (6) (Figure 12) 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.

Figure 12: The "Yellow Book".

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 Section 6.1) 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. (see Section 7.2.1)

The main sources of information on lightweight proprietary fire protection materials is Fire Protection for Structural Steel in Buildings, the "Yellow Book".

In this, information is published on the basis of material thickness V's section factor and fire resistance required.

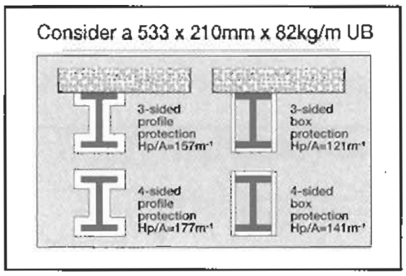


Figure 13: The four most common protection configurations for calculation of Hp/A

Hp/A	Dry Thickness in mm to provide fire resistance of					
Up to	1/2 hr	1 hr	1.5 hr	2 hr	3 hr	4 hr
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 14: Extract from "Yellow Book" as it applies to a typical spray fire protection material.

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 14) 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.

3.3 Castellated and Cellular Beams

For castellated or cellular beams the thickness of the fire protection material should be 1.2 times the thickness determined from the section factor of the original,uncut section. 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.

3.4 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 :-

Figure 15: Intumescent Coated Hollow Sections. (Photograph courtesy of Nullifire Ltd)

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 and water filled hollow sections are discussed in Section 9

3.5 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 16). (7)

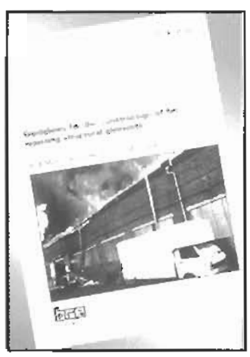


Figure 16: Guidelines for the Fire Protection of Fire Resisting Structural Elements.

The fire protection of castellated and cellular beams is based on that of the parent section plus 20%.

The fire protection requirements for unfilled hollow sections can be derived from those of H or I sections of the same section factor.

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.

4.1 Boards

Board systems (Figure 17) are the most popular type of fire protection in the UK. They are widely used where the protection system is in full view.

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.

4.2 Sprays

After boards, spray protection is the most commonly used fire protection system in the UK (Figure 18). Probably the most common combination of protection materials is to protect beams with sprays and columns with boards.

The principal advantages of sprays 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.

Figure 17: Board Protection Systems (Photograph courtesy of Promat Ltd).

Figure 18: Spray Protection System.

Boards, sprays and intumescent coatings dominate the fire protection market for structural steelwork.

Concrete encasement, once common, is now used mainly where resistance to damage is important.

4. Site Applied Protection Materials ~ *continued*

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.

4.3 Thin Film Intumescent Coatings

Intumescent coatings (Figure 19) 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 of intumescent coatings 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 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 provide 30 and 60 minutes fire resistance. A limited number can be used for longer periods, i.e. 90 and 120 minutes, however the cost increases considerably for periods over 60 minutes.

Figure 19:
Thin Film Intumescent Coating System.

4.4 Flexible/Blanket Systems

Flexible fire protection systems (Figure 20) 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.

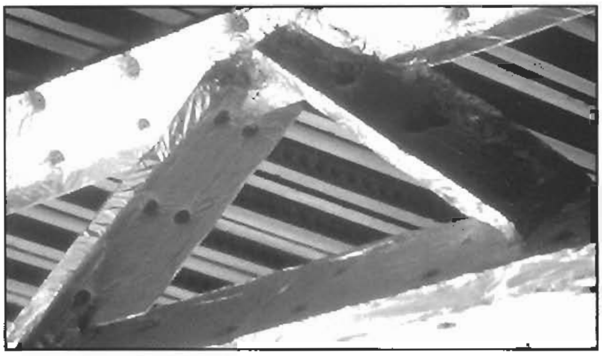


Figure 20: Flexible Blanket Protection System.

Intumescent coatings have been typically chosen for aesthetic reasons.

Their use is however becoming more widespread particularly with the growth of off-site application. (see Section 5).

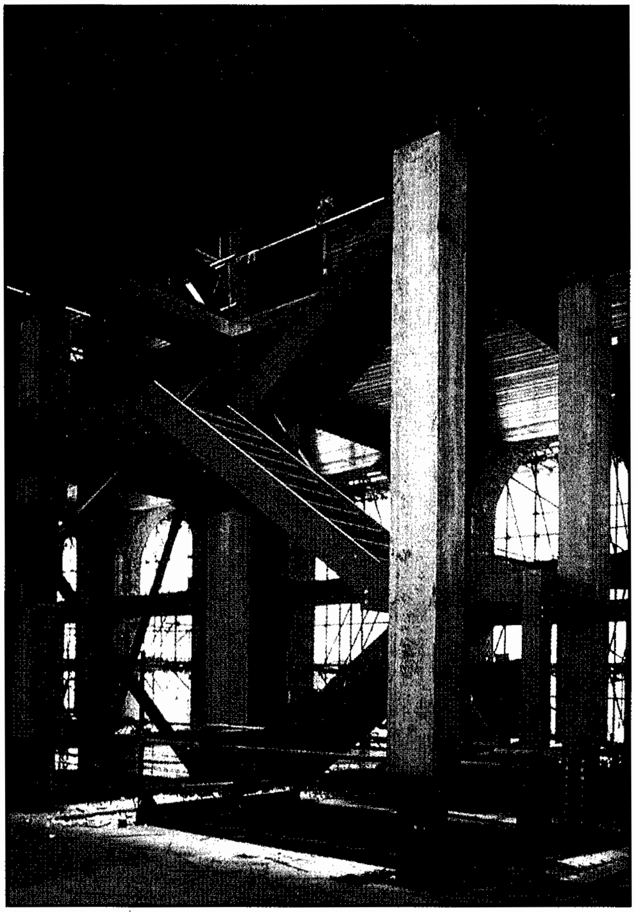


Figure 21: Concrete Encasement.

4.5 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 21). However the introduction of lightweight, proprietary systems such as boards and sprays 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.

5. Off-Site Fire Protection

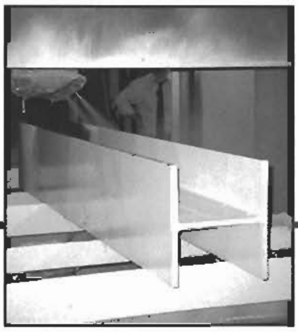


Figure 23: Automated Application of Off-site Intumescent Coatings at the Purpose Built ENOB Treatment Plant, Scunthorpe.

5.1 Thin Film Intumescent Coatings

Intumescent coatings were described in Section 4.3. Of the available fire protection materials it is these which are best suited to large scale off-site application. The coating can be applied either manually, (Figure 22) or automatically, as pioneered by ENOB Treatments at Wishaw and Scunthorpe. (Figure 23)

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

- **Reduced construction time: Fire protection is usually 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. (8)**
- **Reduced overall construction costs .**
- **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. (Figure 24). (9)

The guidance section includes information on the nature of intumescent coatings, environmental compatibility, economics of off-site intumescent coatings, legislative requirements, calculation methods for determining coating thicknesses, composite construction with off-site intumescent coatings and design examples.

The Model Specification covers subjects such as choice of coating and applicator. environmental issues, material specification, application at works, quality control, handling, storage, transport, erection and scheduling of coated steel, damage repair and inspection. It is intended that the information provided will be capable of being included in project specifications.

More general information can be found in the British Steel publication, Off-Site Fire Protection Using Intumescent Coatings. (Figure 25) (10)

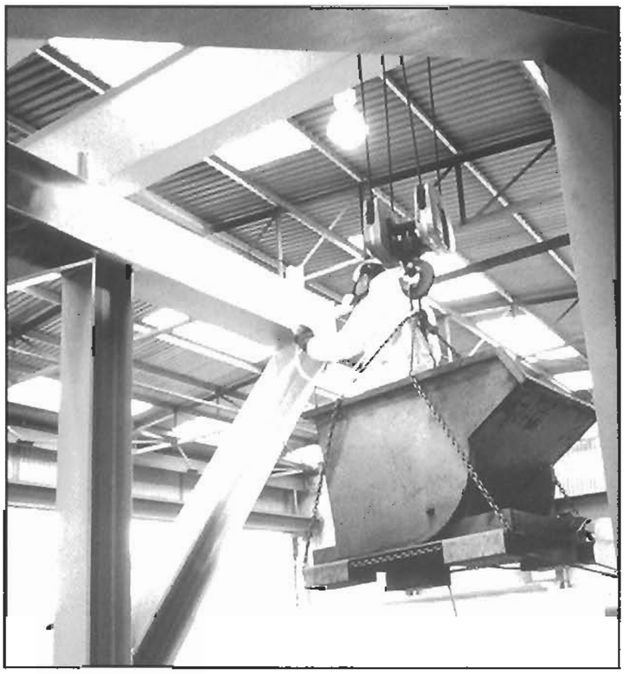


Figure 22: Manual Application of Off-Site Intumescent Coatings.

The principal advantages of off-site applied intumescent coatings are

1. Faster, cheaper construction

2. Reduced number of on-site trades

3. Improved quality

4. Site access and segregation problems are reduced.

5.2 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.

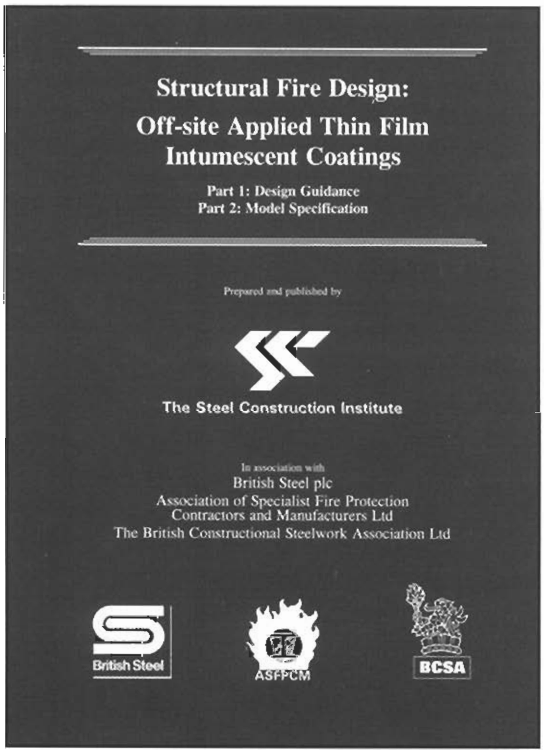


Figure 24: Design Guidance and Model Specification for use with Off-Site Applied Thin Film Intumescent Coatings.

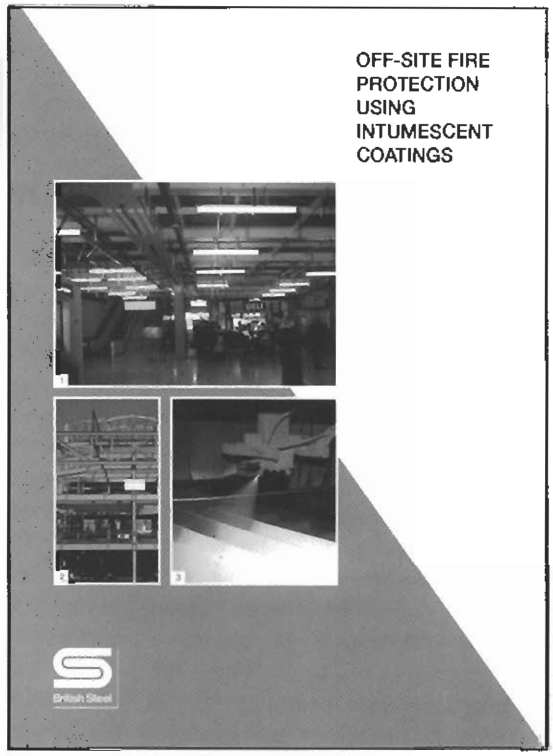


Figure 25: Off-Site Fire Protection using Intumescent Coatings.

Figure 26: Off-Site Application of Spray Protection Material. (Photograph courtesy of Mandoval Ltd).

6. Steelwork Fire Resistance

Steelwork Fire Resistance

Fire resistance is usually expressed in terms of compliance with a test regime outlined in BS476 Part 20 and 21 (11). 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 27).

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 28). 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.

6.1 Effect of Temperature Profile

A joint test programme by British Steel 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 it's performance.

The basic high temperature strength curve shown in Figure 28 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 it's 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.

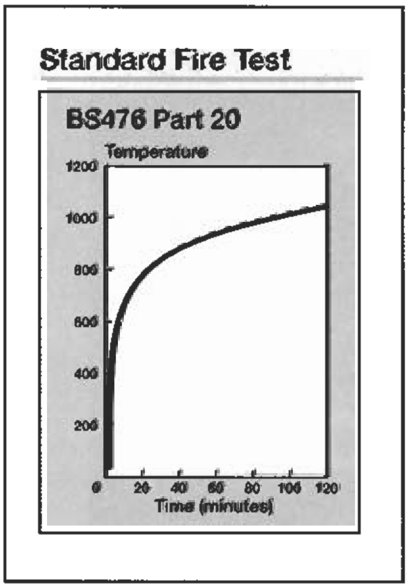


Figure 27: BS476 Part 20 Standard Time-Temperature Relationship for Fire Tests.

The fire resistance of a steel member changes according to

- ***It's temperature profile***
- ***The applied load***

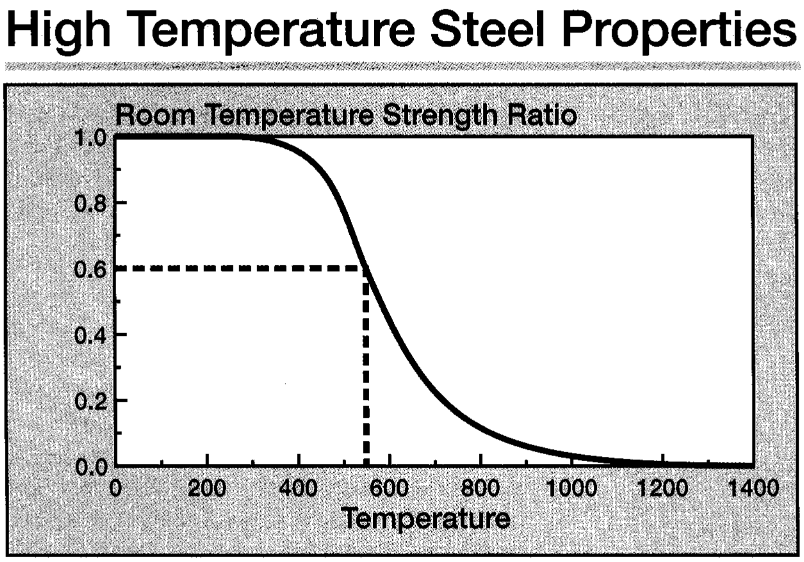


Figure 28: Steel Strength Decreases with Temperature.

6.2 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 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 (12) (See Section 7) load is expressed in terms of the 'Load Ratio' where

$$\textit{Load Ratio} = \frac{\textit{the load at the fire limit state}}{\textit{the load capacity at } 20^{0}C}$$

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 750°C, an increase of 130°C 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 (12)

BS5950 Part 8 (Figure 29) (12) was published in August 1990 and brings together in one document all of the methods of achieving fire resistance of structural steelwork. Although it is based on evaluation of performance of structural steel members in the BS476 Part 20 (11) Standard Fire (See Section 6) it may also be used in fire engineering assessments when natural fire temperatures are derived by calculation (Section 13).

BS5950 Part 8 (12) 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. (11)

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 (13) (Figure 30) .

7.1 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" (6) published jointly by the Association of Specialist Fire Protection (ASFP) and the Steel Construction Institute (see Section 3.2).

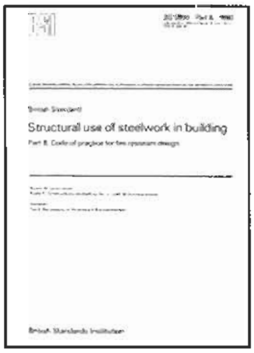


Figure 29: BS5950 Part 8, Code of Practice for Fire Resistance Design.

7.2.1 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 (12) this is done via a set of prepared tables and is illustrated graphically (Figures 11 and 31). If the limiting temperature exceeds the design temperature no protection is necessary.

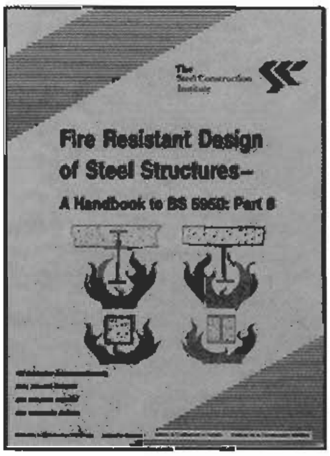


Figure 30: Fire Resistant Design of Steel Structures: A commentary.

The BS5950 Part 8 Limiting Temperature calculation 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 member temperature at the required fire resistance time (the design temperature).

An example of calculations based on the BS5950 Part 8 (12) Limiting Temperature method is provided in the British Steel publication, Design Examples to BS5950 Part 8 (14) (Figure 31).

7.2.2 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 (15). 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 section 8.3).

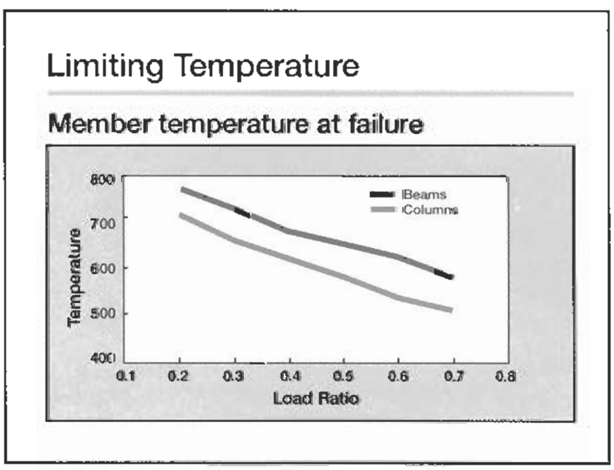


Figure 31: Limiting Temperature varies according to the Load Ratio.

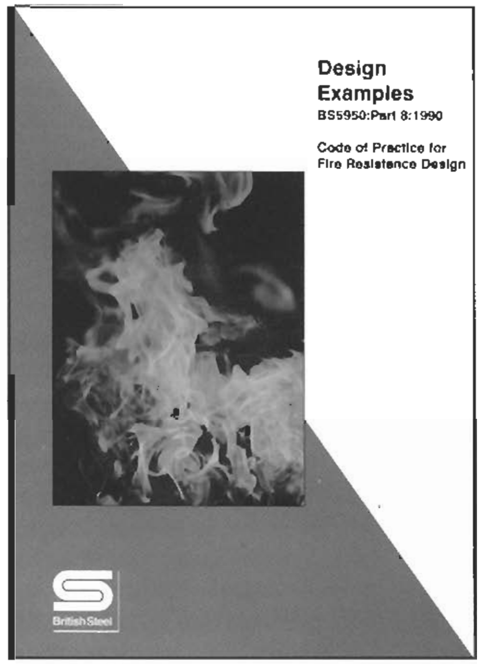


Figure 32: Design Examples to BS5950 Part 8.

The BS5950 Part 8 Moment Capacity 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.

8. Partially Exposed Steelwork

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 Section 3).

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

8.1 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. (16)

8.2 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. (17)

8.3 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 many instances. (18)

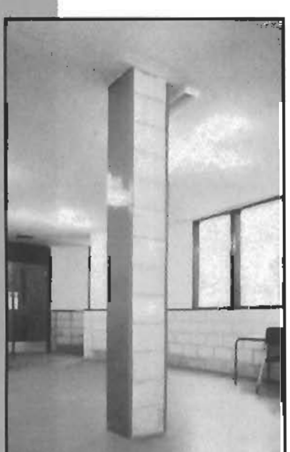


Figure 33: Block-Infilled Column

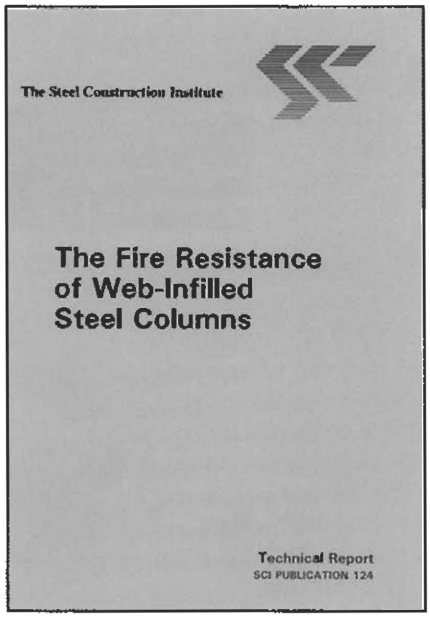


Figure 34: Web-Infilled Column Design Guide.

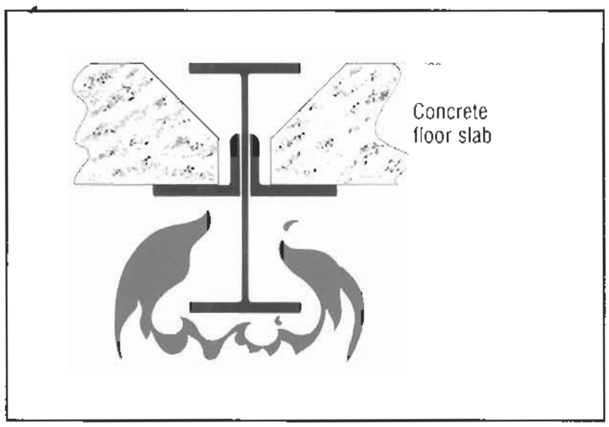


Figure 35: Shelf Angle Floor Beam.

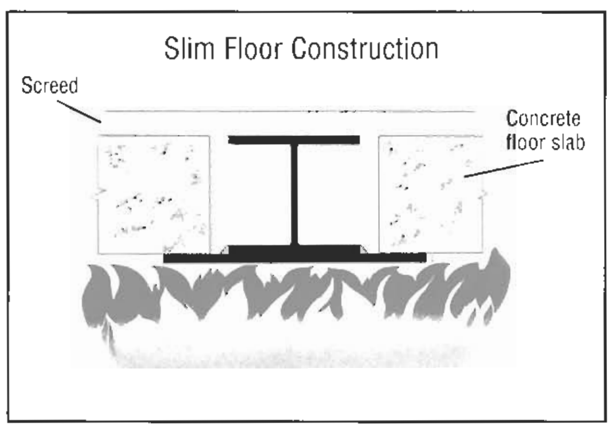


Figure 36: Slim Floor with Precast Slab.

8.4 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. (19,20)

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 British Steel under the trade name SLIMDEK. (21)

The shape of the asymmetric beam is uniquely designed to give optimum performance in fire. The thick web / thin flange configuration gives maximum capacity under the non-uniform temperature distribution at the fire limit state.

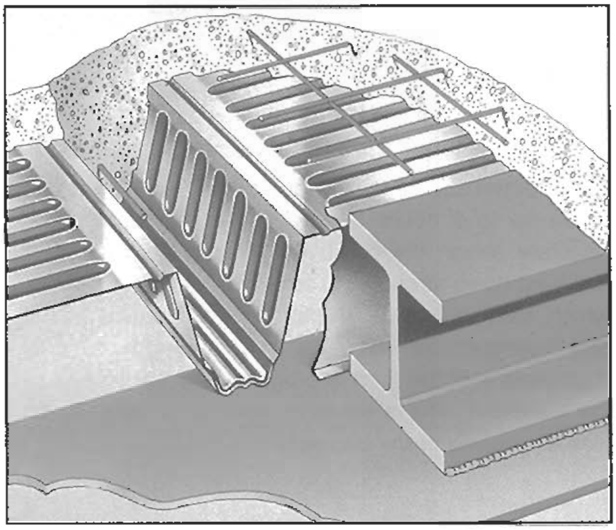


Figure 37: Deep Deck System Slimflor.

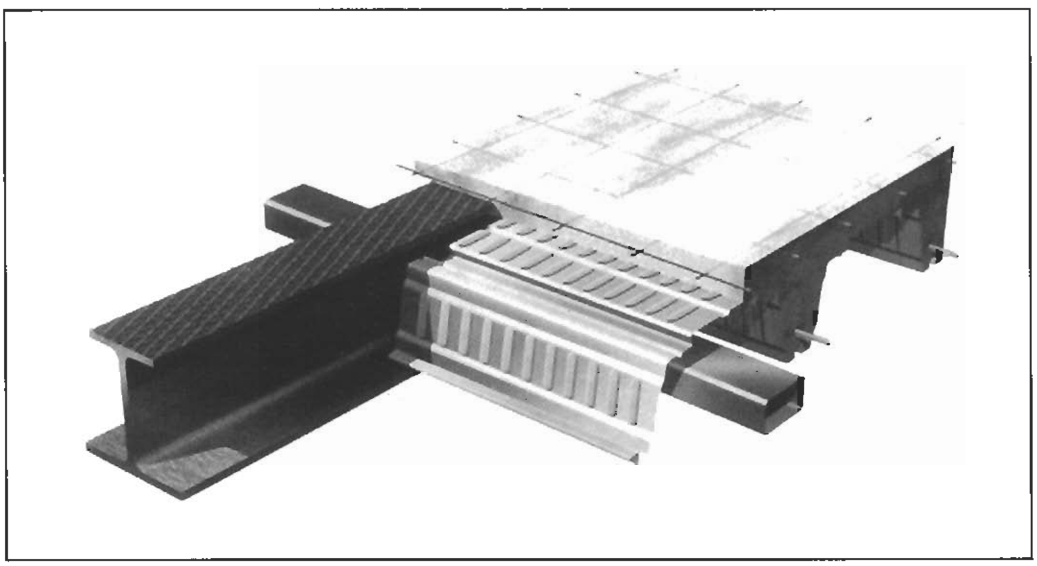


Figure 38: The asymmetric beam used in the Slimdek system is designed for 60 minutes fire resistance without protection and composite action without welded studs.

Fire protection periods of up to 60 minutes are possible without additional applied protection when steel members are partially embedded in walls and floors.

9. Filled Hollow Sections in Fire

Concrete Filled Columns

Unprotected square or rectangular 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 it's 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.

Three possibilities exist for concrete filling, plain, fibre reinforced or bar reinforced concrete. Plain and/or fibre reinforced concrete perform well under compression loading but perform less well when the column is subject to moments. As a result, BS5950 part 8 (12) section 4.6.2.1 requires that two relationships, which limit the moments about both the major and minor axis, must be met. Compliance with both will ensure that the column remains in compression under combined fire limit state axial load and moment.

When moments above these limits are present, the capacity of the concrete filled section can be enhanced by the addition of bar reinforcement. The calculation method for checking the axial and moment capacities is given in BS5950 part 8, (12) section 4.6.2.2.

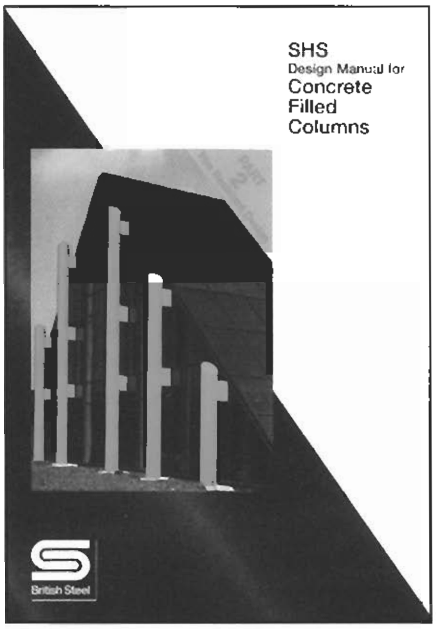


Figure 39: British Steel Design Manual for Hollow Sections in Fire.

As an alternative to bar reinforcement, the column can be designed compositely and protected by a board, spray or intumescent coating system. In this case it is possible to exploit the improved thermal properties of filled columns to reduce the level of external protection used. This can be determined by calculating the passive protection requirement based on the hollow section only and multiplying the resultant thickness by a modification factor. This modification applies to passive protection materials only and is not applicable to intumescents. It can be found, together with information on the advantages, limitations and methodologies of achieving fire resistance using concrete filled tubes in The Design Manual for Concrete Filled Columns Part 2 Fire Resistance Design. (Figure 39) (22)

There is no published U.K. guidance on the fire resistance of concrete filled circular hollow sections. However such guidance, prepared in accordance with Eurocodes 3 and 4 is available in a CIDECT Design Guide. (23)

Queries should be directed to British Steel Tubes Division, Corby, Northants: (Telephone 0500 123133).

Significant periods of fire resistance can be achieved with hollow sections by filling with concrete.

Enhanced periods of fire resistance for concrete filled sections can be achieved by the addition of fibre or bar reinforcement.

10. Combining Fire Resistant Design Methods

The innovative design solutions for beams and columns described here can of course be combined so that whole buildings with fire ratings up to 1 hour can be realised without recourse to site applied protection. (Figure 40) summarises the options. Further details can be found in SCI publication No 189 - Design of Steel Framed Buildings Without Applied Fire Protection. (24)

COLUMN TYPE \ BEAM 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: Fire resistance that can economically be obtained for various structural forms.

11. Single Storey Buildings in Fire

Single Storey Buildings

In the UK, single storey buildings do not normally require fire protection (Approved Document B (1) clause 7.4, See Section 1.1).Exceptions may occur where the structural elements form part of

- **a separating wall.**
- **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).**
- **supports a gallery or supports a roof which also performs 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 in a boundary condition.

In a boundary condition when fire resistance is necessary it has been widely accepted (Approved Document B clause 12.4) that it is sufficient for only the stanchions supporting the walls designated as forming the boundary 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 rafter collapse 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 document, The Behaviour of Steel Portal Frames in Boundary Conditions (Figure 41) (25) An abridged version of the method is also outlined in BS5950 part 8, Appendix F. (12)

Most authorities expect engineers to design portal frame buildings in this way. In England, Wales and 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 a basis for design. A class relaxation is available in Scotland provided that the Steel Construction Institute design method has been used.

The SCI guide also advises on the use of sprinklers in portal frames;

"A sprinkler system is often installed in portal frame buildings which are to be used for retail or warehouse purposes. In the event of a fire their effective operation would control the size of the fire and may even extinguish some fires.The resulting risk of fire spreading beyond the building of origin is greatly reduced. It follows that in a sprinklered building rafter collapse would be unlikely and the dependence on base fixity would be greatly reduced."

In England and Wales the Department of the Environment has indicated that they consider this to be a reasonable approach.

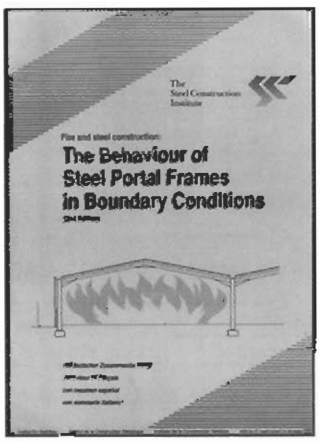


Figure 41: Portal Frames in Boundary Conditions.

Single storey buildings do not normally require fire protection. The most common exception to that rule is where a boundary condition exists.

For portal frames in this situation, the most widely accepted solution is to protect only the stanchion in the boundary walls and to design the bases to maintain stanchion stability in the event of rafter collapse.

12. External Steelwork

External Steelwork

A number of modern steel buildings have being constructed with the steel skeleton on the outside of the structure (Figure 42 & Figure 43). 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 (26) (Figure 44) This describes a method to define the design temperature (see section 7.2.1) 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 (12) (see section 7) 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 British Steel design guide. (27) In addition design against brittle fracture should also be considered and design guidance is given in BS5950 Part 1. (15)

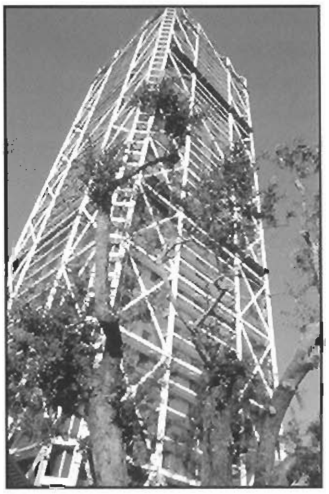


Figure 43: Hotel de las Artes, Barcelona.

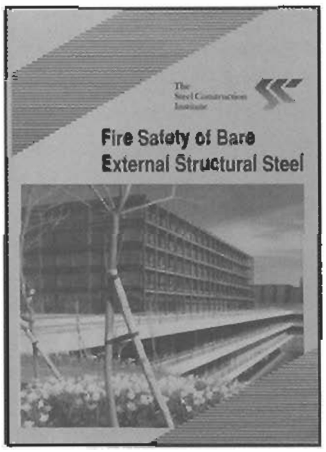


Figure 44: Fire Safety of Bare External Structural Steel.

Figure 42: Wills Factory, Bristol.

Where the steel frame is built outside the structure, it may be possible to position the steelwork in relation to openings in the walls such that it will not require fire protection.

13. Composite Steel Deck Floors in Fire

13.1 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 (28) (Figure 45) 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.

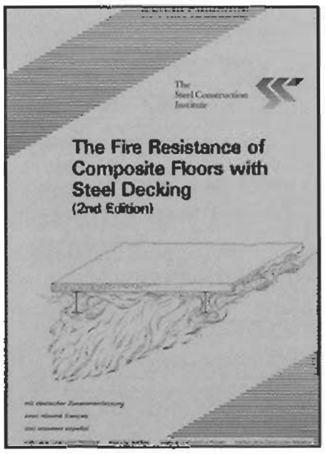


Figure 45: The Fire Resistance of Composite Floors with Steel Decking.

It is not normally necessary to fire protect the soffit of steel deck composite floor slabs.

Two methods are available to design such slabs for fire, the simplified and the fire engineering method.

13.2 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 46) Details are given in the Steel Construction Institute publication The Fire Resistance of Composite Floors with Steel Decking. (Figure 45) (28) and in the information sheets which accompany this publication.

Designers should take care that gaps are filled where the beam forms part of the compartment wall to ensure the integrity of the compartment.

Composite Beams with spray, intumescent or board protection

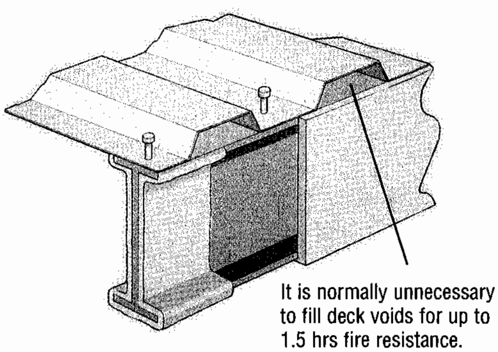
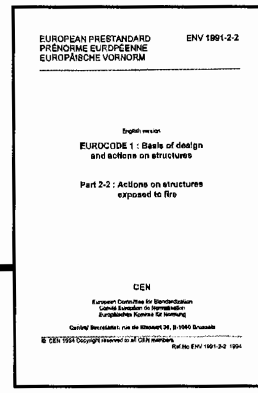


Figure 46: Composite Steel Deck Floor with Unfilled Voids.

In steel deck composite floor construction it is often unnecessary to fill the gaps between the deck profile and the top flange of the beam.

This does not apply when the beam forms a part of a compartment wall.

14. Structural Fire Engineering


EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM
ENV 1991-2-2

English version

EUROCODE 1 : Basis of design and actions on structures

Part 2-2 : Actions on structures exposed to fire

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Figure 47: Eurocode 1 Part 2.2

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 Section 1). 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 (1) states that:

"a fire safety engineering approach that takes into account the total fire safety package can provide an alternative approach to fire safety. It may be the only viable way to achieve a satisfactory standard of fire safety in some large and complex buildings".

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 1991-2-2 (Figure 47) (29) 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:-

$$t_e = q.k.w$$

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.

A Structural Fire Engineering approach can provide an acceptable method of determining fire resistance requirements.

The method proved 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.

Figure 48: Twickenham Main Stand

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 etc. 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 section 6.2) or block filling of columns rather than using standard fire protection materials.

Figure 49: Windsor Park.

Structures designed using fire engineering include sports stadia (Figure 48 & Figure 49), office and industrial buildings, atria, airport terminals (Figure 50), leisure centres, hospitals, shopping centres and car parks.

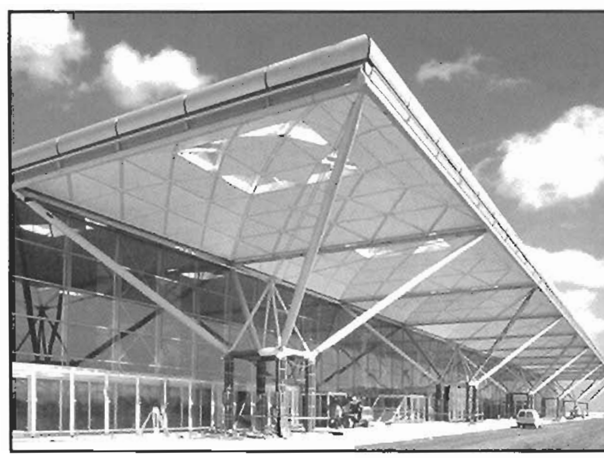

Figure 50: Stansted Airport.

A structural fire engineering study attempts to match the fire protection to the fire threat and may result in significant reductions in fire protection requirements.

The method had been successfully used for sports stadia, airport terminals. leisure centres, shopping centres and car parks.

15. Fire Damage Assessment of Hot Rolled Structural Steel

Re-Use of Fire Damaged 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 British Steel Publication 'The Reinstatement of Fire Damaged Steel and Iron Framed Structures (30) (Figure 51). It's main conclusions are summarised here.

15.1 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 section 6). 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.

15.2 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 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).

15.3 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.

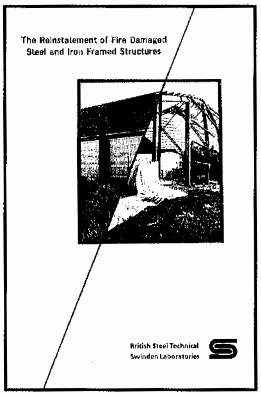

Figure 51: Reinstatement of Fire Damaged Steel and Iron Framed Structures.

Structural steel will lose some of its residual strength after a fire only if the steel temperature has exceeded 600°C

Steel which has been affected by a fire should be checked for residual strength loss. This can generally be done on site using portable hardness testers.

15.4 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 600°C, 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 are 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.

15.5 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 eg. 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/mm²
	187	197	637
	179	189	608
	170	179	559
Grades S355	163	172	539
	156	165	530
	149	157	500
	143	150	481
Grades S275	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.*

If the steel is still straight after a fire then the likelihood is that its residual strength properties have not been significantly affected by the fire. Connections and foundations should be examined for cracking and suitability for re-use.

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