SCI PUBLICATION 080

Fire Resistant Design of Steel Structures–

A Handbook to BS 5950 : Part 8

Brandsichere Bemessung von Stahlkonstruktionen
Ein Handbuch zu BS 5950: Teil 8

Dimensionnement pour la Résistance au Feu des Structures en Acier.
Un Manuel pour la BS 5950: Partie 8

Diseño de Estructuras de Acero Resistentes al Fuego
Un Manual de la BS 5950: Parte 8

Progetto di Strutture in Acciaio Resistenti al Fuoco
Un Manuale per l'Applicazione delle BS 5950: Parte 8

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FOREWORD

This publication has been prepared to assist designers and others involved in evaluating the fire resistance of steel structures.

It follows the recently published *BS 5950: 'Structural use of steelwork in building, Part 8: Code of practice for fire resistant design'* and describes the background to the clauses in the standard.

The Handbook was written by Dr R M Lawson and Mr G M Newman of the Steel Construction Institute. Much of the information used was provided during the course of preparing the standard, and the authors are grateful to the Chairman of the Drafting Panel of *BS 5950: Part 8*, Mr J T Robinson, and its secretary, Mr A Weller for their assistance at this stage.

The following members of the Committee and other experts commented on the Handbook in its draft form:

Dr G M E Cooke	Fire Research Station
Dr M Edwards	British Steel (Tubes Division)
Dr B R Kirby	British Steel (Swinden Laboratories)
Dr S J Melinek	Fire Research Station
Mr J T Robinson	British Steel (General Steels)

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CONTENTS

SUMMARY

Fire Resistant Design of Steel Structures
A Handbook to BS 5950: Part 8

This publication covers the means of achieving the required fire resistance of steel structures used in building. It follows BS 5950: Part 8 'Code of Practice for Fire Resistant Design' (1990), and describes the background to the Code Clauses. The publication is presented as a Handbook and is intended to be read as a narrative. Cross-references to the Code Clauses are included adjacent to the text.

The main Sections in the Handbook deal with the performance of steel at elevated temperatures, and the means of determining the rise in temperature of steel sections in the standard fire. The evaluation of fire resistance is treated in two ways: by performance based on tests, or alternatively, by calculation methods. The basis of the test approach is presented in terms of 'limiting temperatures' that have been determined for beams supporting floors and columns, as functions of the loads applied to them in fire conditions. The calculation approach used for flexural members, such as beams or floors, is called the 'moment capacity' method.

The method of determining the required thickness of fire protection is new to the UK and is largely based on that put forward in the 'European Recommendations for Fire Safety of Steel Structures'. It permits the calculation of thickness of protection as a function of the thermal properties at elevated temperatures of the materials used.

The other parts of the Handbook deal with particular structural forms, such as: shelf angle floors, composite floors, portal frames, and concrete-filled hollow sections. The concept of 'natural fires' is also reviewed, although this is not strictly included in BS 5950: Part 8. Finally, a number of worked examples is provided.

Brandsichere Bemessung von Stahlkonstruktionen
Ein Handbuch zu BS 5950, Teil 8

Zusammenfassung

Diese Veröffentlichung behandelt die Methoden zur Erzielung der geforderten Feuerwiderstandsdauer von Stahlkonstruktionen in Gebäuden. Sie folgt BS 5950: Teil 8, *'Code of Practice for Fire Resistant Design' (1989) und beschreibt den Hintergrund der Norm. Die Veröffentlichung wird als Handbuch vorgestellt, mit der Absicht als Erzählung gelesen zu werden. Querverweise zu den Paragraphen der Norm werden gegeben.*

Die Hauptabschnitte in dem Handbuch befassen sich mit dem Verhalten von Stahl bei höheren Temperaturen und den Möglichkeiten, den Temperaturanstieg im Stahlquerschnitt unter Normbrandbeanspruchung zu bestimmen. Die Bewertung des Feuerwiderstands geschieht entweder durch das Verhalten im Test oder rechnerisch. Die Grundlage der Test-Methode liegt im Begrenzen der Temperaturen, die für Träger und Stützen als Funktion der im Brandfall vorhandenen Lasten bestimmt wurden. Das Rechenverfahren für biegebeanspruchte Bauteile wie Träger oder Decken wird 'Momentenkapazität'-Methode genannt.

Die Methode zur Bestimmung der erforderlichen Dicke des Brandschutzes (Verkleidung, Anstrich) ist neu im Vereinigten Königreich und stützt sich weitgehend auf den Vorschlag in den 'Europäischen Empfehlungen zum

Brandschutz von Stahltragwerken'. Sie erlaubt die Berechnung der Dicke des Brandschutzmaterials in Abhängigkeit von dessen thermischen Eigenschaften bei erhöhten Temperaturen.

Die anderen Abschnitte des Handbuchs befassen sich mit besonderen Bauteilen wie z.B. 'shelf angle'
Decken, Verbuddecken, Rahmen und betongfüllten Hohlprofilen. Das Konzept des 'Naturbrandes' wird nochmals geprüft obwohl in BS 5950, Teil 8 *nicht ausdrücklich enthalten. Schließlich wird eine Reihe von Beispielen bereitgestellt.*

Dimensionnement pour la Résistance au Feu des Structures en Acier. Un Manuel pour la BS 5950 : Partie 8

Résumé

Cette publication couvre les diverses possibilités permettant de donner une résistance à l'incendie aux structures en acier utilisées dans les bâtiments. Elle est basée sur la partie 8 – Code de bonne pratique pour un dimensionnement à la résistance au feu – de la BS 5950 (1989), *et explique les bases des diverses clauses du code. La publication est présentée sous forme d'un manuel et peut être lue comme une narration. Les références aux articles du Code sont données en face du texte.*

Les principales sections du manuel portent sur le comportement de l'acier à haute température et sur les moyens de déterminer l'élévation de température dans les éléments an acier, lors d'un feu standardisé.

L'evaluation de la capacité de résistance au feu est traitée de deux manières: sur base de résultats d'essais, d'une part, et par des méthodes de calcul, d'autre part. L'approche par résultats d'essais est présentée sous forme de 'températures limités' qui ont été déterminées pour des poutres suppportant des planchers et pour des colonnes, en fonction des charges appliquées dans des conditions d'incendie. L'approche par calcul utilisée pour les éléments fléchis, comme les poutres ou planchers, est appelée méthode de 'capacité en moment'.

La méthode de détermination de l'épaisseur requise de protection à l'incendie est nouvelle pour le Royaume-Uni. Elle est largement basée sur les 'Recommendations Européennes pour la Sécurité au Feu des Structures en Acier'. Elle permet le calcul de l'épaisseur de protection en fonction des propriétés thermiques, aux températures élevées, du matériau utilisé.

Les autres parties du manuel concernent des formes structurales particulières, comme les planchers sur cornières, les planchers composites, les portiques et les profils creux remplis de béton. Le concept d''incendies naturels' est aussi examiné, bien qu'il ne soit pas effectivement considéré dans la partie 8 de la BS 5950. *Enfin, divers exemples sont également donnés dans le manuel.*

Diseño de Estructuras de Acero Resistentes al Fuego Un Manual de la BS 5950: Parte 8

Resumen

Esta publicación recoge los medios de conseguir la resistencia al fuego requerida por las estructuras de acero usadas en edificación. Sigue la BS 5950: parte 8 *'Norma para el Diseño con Resistencia al Fuego' (1989) y describe las bases subyacentes a los artículos de la Norma. La publicación se presenta como un Manual y se pretende de fácil lectura. En los margenes del texto se incluyen referencias a los artículos de la Norma.*

La sección principal del Manual trata del comportamiento del acero a temperaturas elevadas y de los procedimientos para determinar el incremento de temperaturas en las secciones de acero sometidas al fuego tipo. La evaluación de la resistencia al fuego se realiza por dos caminos: mediante ensayos o mediante cálculo. La base de los ensayos se presenta en términos de 'temperaturas límite' que deben ser determinadas, para vigas de piso y columnas, en función de las cargas a ellas aplicadas en condiciones de fuego. El procedimiento basado en cálculos para piezas a flexión, como vigas o forjados, es llamado método de 'capacidad de momento'.

El procedimiento para calcular el espesor de protección frente al fuego es nuevo en el Reino Unido y ampliamente basado en las 'Recomendaciones Europeas para la seguridad al fuego de estructuras de acero'. Permite el cálculo del espesor de protección en función de las propiedades termales, a altas temperaturas, de los materiales utilizados.

Las otras partes del Manual se dedican a formas estructurales diversas: forjados compuestos, estructuras porticadas, secciones huecas rellenas de hormigón, etc. Aunque no estrictamente incluido en la BS 5950: Parte 8, *se revisa igualmente el concepto de 'fuegos naturales'. Finalmente se recogen algunos ejemplos desarrollados.*

Progetto di Strutture in Acciaio Resistenti al Fuoco
Un Manuale per l'Applicazione delle BS 5950: Parte 8

Questa pubblicazione tratta degli strumenti per ottenere che le strutture in acciaio abbiano i requisiti di resistenza al fuoco previsti. Essa fa riferimento alle norme BS 5950: parte 8 *'Specifiche per il progetto di strutture nei confronti della resistenza al fuoco', delle cui raccomandazioni vengono presentate le basi.*

La forma adottata e' quella di un manuale, da leggersi con la facilita' di un romanzo; il riferimento ai vari punti della norma appaiono a lato del testo.

Le principali sezioni nelle quali si articola il manuale trattano delle prestazioni dell'acciaio ad elevate temperature e dei modi per determinare l'aumento di temperatura di profili di accciaio soggetti a incendio standard. La valutazione della resistenza al fuoco viene affrontata secondo due approcci; mediante l'esame di prestazioni basate su risultati sperimentali, o, in alternativa, mediante metodi di calcolo. L'approccio sperimentale si fonda sulle 'temperature limiti' determinate, in funzione del carico applicato, mediante prove su travi di sostegno di solai e su colonne in condizioni di incendio. L'approccio alternativo, valido per elementi inflessi, come travi o solette, prende il nome di metodo del 'momento resistente'.

Il metodo per la determinazione dello spessore richiesto per la protezione al fuoco e' nuovo per il Regno Unito e si fonda in larga parte sulle 'Raccomandazioni Europee per la Sicurezza al Fuoco delle Strutture in Acciaio'. Esso consente il calcolo dello spessore della protezione in funzione delle proprieta' termiche ad elevate temperature dei materiali impiegati.

Le altri parti del Manuale si riferiscono a particolari soluzioni strutturali, quali i solai poggiati su angolari, i solai composti, i portali e i profili cavi riempiti di calcestruzzo. Viene introdotto e discusso anche il concetto di 'incendio naturale', sebbene a questo non si faccia riferimento in senso stretto nelle BS 5950: parte 8.

A conclusione, vengono riportati alcuni esempi completamente svolti.

FIRE RESISTANT DESIGN OF STEEL STRUCTURES – A HANDBOOK TO BS 5950: PART 8

ERRATA

The following important errors have been noted in the text of SCI publication P-080.

Page 47: R x Load multiplier = Steel strength reduction factor
replaces
R = Load multiplier x Steel strength reduction factor

Page 67: Add below Table 9.1:

s – simply supported slab
c – continuous slab

Page 79: Multiplication Factor $\ngtr$ 1.25 in Equation (28)
replaces $>$ 1.25

Page 84: $\eta =$ in Equation (35)
replaces $\eta \leq$

Page 97: 7 lines below Figure 13.3:

2 to 3 m/min
replaces 2 to 3 m/s

Page 102: 10 lines from bottom of page:

... residual strength of cold formed steel members up to grade Z35 may be taken as 70% of the specified strength ....
replaces 90%

Add to bottom of first paragraph of Section 14.1.3:

Nevertheless, cracking can occur in cast iron columns due to rapid cooling or distortion, in which case the sections cannot easily be re-used.

Page 124: 8 lines from bottom of page:

Reduction factor = 0.139
replaces 0.193
Bending capacity = 1.42 kNm per metre
replaces 1.97 kNm per metre

Pages 127 to 132 Reference in Design Example to;
Code Table 9 replaces Table 11
Code Table 10 replaces Table 12
Code Table 11 replaces Table 13

Page 32: Figure 4.7; Reverse directions of convection and radiation arrows.

November 1990

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1. INTRODUCTION

1.1 Introduction to the publication

This publication is intended to be a Handbook to British Standard 5950 '*Structural use of steelwork in building, Part 8: Code of practice for fire resistant design*'. This is the first Code or Standard in the UK dealing specifically with the fire resistance of steel structures designed to *BS 5950: Part 1*[1]. The Code sets out a methodology for determining fire resistance, based on fire tests and analytical methods. The Handbook provides an explanation of the clauses in the Code, and presents additional design information. Design Examples are included as an aid to interpretation and application of the clauses.

The Code presents methods of fire safety design whereby the designer can select an appropriate thickness of fire protection or, in many cases, demonstrate that no protection is needed. It is intended that fire protection manufacturers provide the necessary material data consistent with the use of the Code. There are other forms of steel element where careful consideration of the partial-protection offered by insulating elements such as walls and floors can result in savings in the amount of fire protection needed. These calculation methods are covered in the Code and explained in the Handbook.

It is appreciated that to many designers the question of 'designing' for fire resistance, rather than the use of prescriptive methods, is a new concept. A key reference to the prescriptive approach is the publication *Fire Protection for Structural Steel in Building*[2] which gives thicknesses of fire protection for different proprietary and generic fire protective systems.

It is intended that the Handbook forms a continuous narrative and can be read for general guidance on the methods of fire resistant design and as commentary on the Code. The Code is necessarily a statement of good practice and cannot include all the relevant test or research information. Both the Code and the Handbook refer to other documents for more detailed information. Key references are given later. The Handbook should not be used for design without reference to the source standards and other relevant documents.

1.2 Introduction to fire safety

The purpose of the British Standard 5950: Part 8 *Code of practice for fire resistant design* is to present methods of design to achieve the appropriate fire resistance and hence to avoid premature failure of steel structures in fire. The Code should be read in the context of the current *U.K. Building Regulations*[3] where appropriate periods of fire resistance are presented for different building uses and sizes.

Regulations are concerned primarily with safety in buildings. However, fire resistance periods often exceed those required for safe evacuation and fire fighting. An important report *Fire safety in buildings* [4] endeavours to separate the provisions for life-safety and property protection in the current regulations. So-called 'active' measures of fire protection, such as detectors and sprinklers can improve both life-safety and property protection as they attempt to control the severity of a fire.

Passive measures such as fire protection of the structure and compartmentation are the traditional means of controlling the effects of a fire once ignition has occurred. Compartmentation limits the fire spread throughout the building, and from building to building (via the boundary walls). In many buildings such as shopping malls, factories and sports halls, compartments are necessarily very large and 'active' protection measures may need to be considered as part of the design concept.

Before considering the technical aspects of fire resistant design, it is important to have a measure of the 'cost' of fires in the UK. Insurance statistics(4) suggest that the total number of fires has remained constant at around 350,000 per year and the total loss to the nation has increased steadily to about £1 billion in 1987. The 1000 most severe fires constitute more than 60% of the total loss and the 50 most severe fires (amounting to an individual loss of more than £1 million) constitute more than 30% of the total. The most 'expensive' fires, from a loss point of view, tend to be those in industrial or warehouse buildings, where the contents loss is a major factor. Structural damage is often relatively light in fires (apart from buildings of the above type) and the incidence of collapse in fire protected buildings is negligible.

BS 5950: Part 8 is concerned only with 'passive' protection (i.e. use of fire protective materials) of steel elements to achieve a specified period of fire resistance. 'Active' methods that also contribute to a reduction in the severity of the fire are not included within the scope of the Code. In this context, fire resistance is defined as appropriate to the criteria imposed by the standard fire resistance test to *BS 476:Part 20*(5). This uses a standard time-temperature relationship and is a means of establishing the relative performance of structural systems and materials in fire.

So-called 'natural fires' are ones where the temperature-time history is determined by the fire load (or combustible contents) and ventilation conditions of a compartment. Natural fires are different in nature and in effect and may be considered to be more realistic than the 'standard' fire. In Europe, considerable research effort has gone into the evaluation of natural fires(6). Indeed, steel structures can benefit considerably from this approach as it is often possible to demonstrate that no fire protection is needed for buildings with low fire loads. This relaxation on current Regulations has been justified for many 'special' structures such as sports halls, car parks, railway and airport terminals. These methods are outlined in Section 13 of this publication.

Conventional passive fire protection can add up to 30% to the cost of bare steelwork, potentially accounting for some £200 million 'added-cost' to steel frames in multi-storey buildings. The steel industry in the U.K. has therefore expended considerable research effort into quantifying the inherent fire resistance of unprotected or partially-protected steelwork, and there are many forms of structure where this approach is feasible.

In principle, the designer is concerned with determining the amount of fire protection needed to satisfy the required fire resistance for each steel section, or in some cases, establishing whether bare steelwork alone meets the design criteria. Some steel members which support compartment or boundary walls may need special consideration.

1.3 Scope of the Code

The Code is principally concerned with 'minimizing the risk of structural collapse in fire'. Certain clauses also deal specifically with 'restricting the spread of fire' where there is a direct requirement for stability of the structural members supporting boundary or compartment walls. Two methods of evaluating the fire resistance of steel structures or members are covered:

- Performance derived from fire resistance tests to *BS 476: Part 20* and *Part 21* (5) (formerly to *BS 476: Part 8*)

This method relates to the use of unprotected steel beams and columns, and to the performance of fire protective materials. Manufacturers' fire test data may already be available, and the publication of this Code gives a framework by which further data can be obtained and utilized in design. Some data may have been obtained from Codes previously in operation, such as *BS 476: Part 8* and *BS 449*(7). The use of this data is not precluded provided proper account is taken of the effect of changes in these standards relative to current practice.

The Code also presents new information on the fire resistance of composite deck slabs, shelf-angle floors and concrete-filled hollow section columns. The economy of these forms of construction has benefited from large-scale fire tests leading to a better understanding of their behaviour at elevated temperatures.

- Performance derived from calculations

  The Code provides methods of calculation by which the designer can establish appropriate thicknesses of fire protection. There are also many structural forms where the designer can demonstrate by calculation that no fire protection is needed. Calculations are, by implication, conservative and therefore appropriate tests would generally result in greater economy.

The context of these tests and calculations is the standard 'fire-resistance' time-temperature history defined in Section 2.1. Although not covered directly by the Code, these methods may also be applied to 'special structures' where consideration of the effect of 'natural fires' can give a better prediction of the temperatures that may be experienced and the structural response to them.

An Appendix to the Code gives information on the reinstatement of steel structures after moderate fires.

1.4 Definitions

The following key definitions are important in interpreting the Code:

Critical element: The element of a cross section that would reach the highest temperature in fire conditions. 1.2.1

Element of Structure (or building construction): This is the term used in Codes to define a member such as a column or beam and should not be confused with the 'critical element' above. 1.2.3

Design temperature: The temperature of the critical elements (the part of the section that reaches the highest temperature) at the appropriate fire resistance period in the *BS 476: Part 20* test. 1.2.2

Fire resistance: The ability of an element of building construction to withstand exposure to a standard time-temperature and pressure regime without loss of its fire-separating function or load-bearing function or both, for a given time (as defined in *BS 476: Part 20*). 1.2.10

Limiting temperature: The temperature of the critical element at which the member would fail under the given fire and loading conditions. 1.2.7

Load ratio: Ratio of applied forces in fire conditions to those used in the design of the member at room temperature.

Thermal conductivity of a material: Quantity of heat in unit time (Watts) which passes through a unit cross-sectional area of a material for a unit temperature gradient (i.e. 1°C temperature change per unit length).

Specific heat of a material: Quantity of heat stored (Joules) in a unit mass of a material (kg) for 1°C temperature rise.

Standard fire: A term used to define the time – temperature regime in a fire resistance test.

Natural fire: A term used to differentiate fires that may occur in buildings from those in a fire resistance test.

The particular requirements and definitions appropriate to the fire resistance test are described in Section 2.1.

2. FIRE LIMIT STATES

2.1 Fire resistance test

Three criteria are imposed by the standard fire resistance test to *BS 476: Part 20*[5] (formerly Part 8):

- Insulation: A fire on one side of a wall or underside of a floor acting as a compartment boundary should not cause combustion of objects on the unexposed side. Limits of temperature rise of 140°C (average) or 180°C (peak) above ambient temperature are specified in the standard fire resistance test. 1.2.5
- Integrity: A wall or floor acting as a compartment boundary should not allow passage of smoke or flame from one compartment to another as a result of breaks or cracks in the wall on floor. Both the insulation and integrity criteria also apply to members embedded in walls or floors. 1.2.6
- Load-carrying capacity: The members in a structural assembly should resist the applied loads in a fire. Failure criteria for beams and columns are defined in Part 20 as: 1.2.8

  Beams:

  (a) A limiting deflection of span/20 is reached or,

  (b) For deflections greater than span/30, a rate of deflection of span^2/(9000 × member depth) is exceeded. The units of rate of deflection are mm/min when the dimensions are in mm.

  Columns:

  Failure to support the applied load. In practical terms, this corresponds to a rapid rate of increase of vertical deflection (limit undefined) or a maximum lateral deflection of about 120 mm (depending on the column furnace).

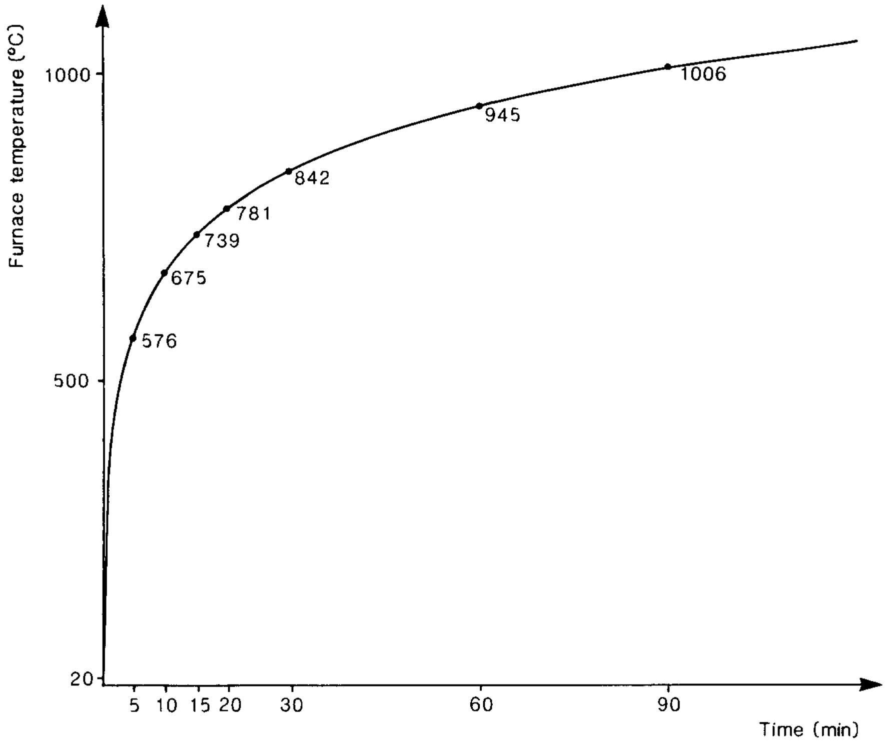


Figure 2.1 *Temperatures in standard ISO fire test*

The fire resistance test follows a temperature-time curve (defined in *BS 476 Part 20* and *ISO 834*[8]). This is presented in Figure 2.1. The curve is described by the formula:

$$T = 345 \log_{10}(8t + 1) + 20 \qquad (1)$$

where T = the furnace temperature (°C)
t = the elapsed time (mins).

The fire resistance test is not intended to reflect the temperatures, and hence structural behaviour that would be experienced in real fires. It is a means of obtaining a measure of the relative performance of structures and materials within the capabilities of standard gas fuelled furnaces.

2.2 Load factors

Fire in buildings is a rare occurrence and for calculation purposes is treated as a form of 'accidental' loading. Partial factors on loads (γ_f) and materials (γ_m) reflect the fact that the structure is required to 'survive' extreme events such as fire but without need for further reserve of strength. This is because the probability of overload and inaccuracies in the method of calculation (or testing) are considered to be small and of less significance than those under normal loading situations.

Partial factors on loads (γ_f) are taken as unity for permanent dead loads, storage loads, and loads on escape stairways and lobbies. These factors are reduced to 0.8 on beams, floors and columns subject to non-permanent imposed loads. The partial factor for wind load is reduced to 0.33 for buildings greater than 8 m in height (to eaves height in pitched roofs). The effect of wind loading may be ignored for smaller structures. Snow loads on roofs may also be ignored. 3.1 Table 2

A fire in a compartment would be expected to 'consume' a major part of the loading in it. However, the floor above a compartment is subject to load deriving from the compartment above which may not be subject to fire. Therefore the loading on the floors in fire conditions would be considerably different in the two cases. The factor of 0.8 is a conservative measure of the loads in the case of floors subject to fire beneath, and takes into account the probability of loading in fire conditions in general working areas. The proposed Eurocode Annex on 'Actions' may suggest a factor of 0.7 on non-permanent imposed loads.

Loads in fire conditions should not be confused with 'fire loads' which is the equivalent weight of wood representing the calorific value of the combustible contents of a compartment or unit of floor area. (see Section 13.3).

2.3 Material factors

The partial factors on material strength (γ_m) at the fire limit state are taken as unity for structural steel and reinforcement, and 1.3 for concrete. On average the actual strength of these materials will be greater than the characteristic values used in normal design. Strength reduction factors for structural steel at elevated temperatures are presented in Section 3.2. 3.3

3. PROPERTIES OF STEEL AND OTHER MATERIALS IN FIRE

3.1 Physical properties of steel

The important properties in determining the performance of steel at elevated temperatures are: strength, deformation, thermal expansion, specific heat and thermal conductivity. All these properties vary with temperature, as is the case for most structural materials.

Steel begins to lose strength at temperatures above 300°C and reduces in strength at an approximately steady rate until around 800°C. The residual strength, which although small, then reduces more gradually until the melting temperature at around 1500°C. This behaviour is similar for hot rolled structural and reinforcing steels, but with cold worked sections and reinforcement there is a more rapid decrease of strength after 300°C. In comparison, temperatures in a fire resistance test do not exceed 1200°C (after 4 hours).

The behaviour of steel in fire is affected by the rate of heating, as there is a component of deformation arising from creep at temperatures above 450°C. The creep behaviour of steel is a complex phenomenon and it is difficult to incorporate this factor into the time-independent design methods, with which the designer is familiar.

For this reason, research, particularly in the UK, has concentrated on the effect of heating rate. Assuming that the rise in temperature of the section is linear and that a limiting temperature of 600°C is attained at failure times of 30 to 120 minutes, then the practical rates of heating are 5°C/minute (for well-insulated sections) to 20°C/minute (for poorly insulated sections). Small-scale tensile tests carried out using rates of heating in this range would therefore be representative of the behaviour of beams and columns in large scale tests. Methods of test are discussed in Section 2.2.

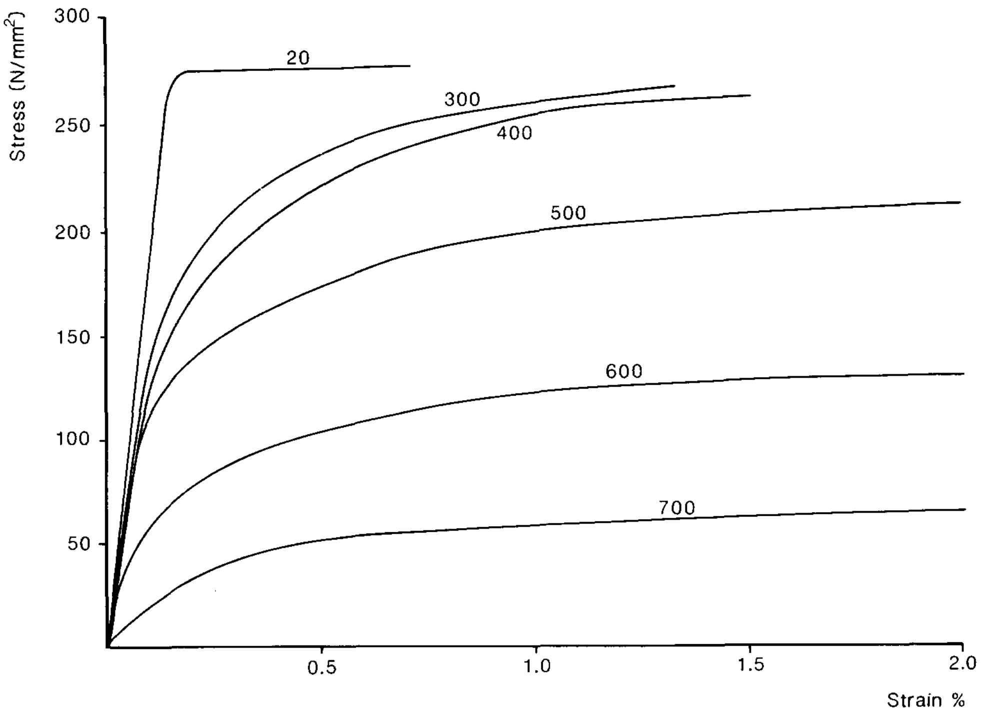


Figure 3.1 *Stress-strain data for grade 43A steel at elevated temperatures*

At temperatures higher than about 300°C, steel does not display a well-defined yield point and there is a more gradual increase of strength with strain. This behaviour is illustrated in Figure 3.1. The capacity of steel to accept high levels of strain increases significantly at higher temperatures (strains of over 5% are often experienced in beam tests). The elastic modulus of steel is therefore more appropriately the tangent modulus at low stress. At higher stresses the effective elastic modulus is simply the stress divided by the strain for the particular heating rate under consideration. The elastic modulus is therefore a rather imprecise property which means that interpretation of the deflection of beam and column tests can be difficult.

The following properties of steel are given in the ECCS Recommendations(9) and other references(10). *BS 5950: Part 8* gives single values for these temperature-dependent properties, which are reasonably accurate for most design purposes.

The coefficient of thermal expansion of steel (α_T increases slightly with temperature. At room temperature α_T is often taken as 12×10^{-6}/°C, but at temperatures in the range of 200 to 600°C, α_T may be taken as 14×10^{-6}/°C. At around 730°C steel undergoes a phase-change and there is a marked change in the expansion characteristics as energy is absorbed and the material adopts a denser internal structure (see Figure 3.2). 2.1 (a)

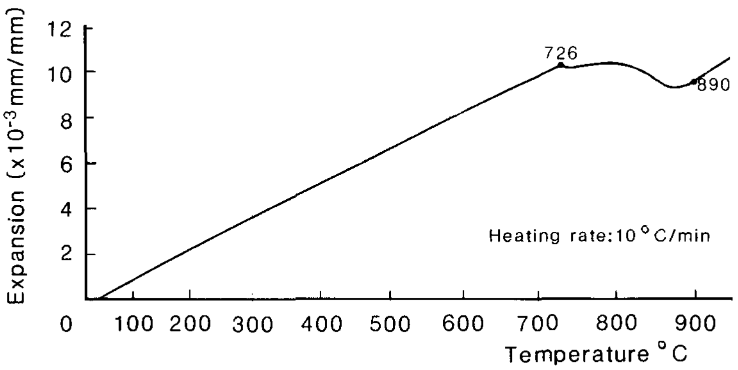


Figure 3.2 *Expansion of grade 43 steel with temperature*

The total extension δ_T from 20°C to a temperature below this phase change is more accurately given by the formula:

$$\delta_T = (0.4 \times 10^{-8}\, t_s^2 + 1.2 \times 10^{-5}\, t_s - 2.42 \times 10^{-4})\, l \qquad (2)$$

where l = the original length of the specimen and t_s is the temperature of the steel (°C).

The coefficient of expansion at a particular temperature is the rate of change of extension with respect to temperature.

The specific heat of steel is the heat stored (Joules) in a unit mass of steel for 1° temperature rise. The greater the specific heat of a material, the smaller its temperature rise for a given amount of heat it absorbs, and the smaller its temperature fall for a given amount of heat it gives out. For most calculations a constant value of specific heat of steel C_s of 520 J/(kg°C) may be used. More accurately for temperatures up to 725°C, the specific heat of steel is: 2.1(b)

$$C_s = 0.47 + 2 \text{x} 10^{-4}\, t_s + 38 \text{x} 10^{-8}\, t_s^2 \ (\text{kJ/(kg°C)}) \qquad (3)$$

At approximately 725°C, the specific heat of steel rises rapidly as a result of changes in the internal lattice structure of steel at this temperature.

Thermal conductivity is defined as the amount of heat in unit time (Watts) which passes through a unit cross-sectional area of a material for a unit temperature gradient (i.e. 1°C temperature change per unit length). This parameter is less important for steel than for fire protective materials. This is because the thermal conductivity of steel is over 50 times greater than concrete and 500 times greater than vermiculite-cement (a typical fire protection material). The thermal conductivity of steel (k_s) is approximately 37.5 W/m°C or more accurately: 2.1 (c)

$$k_s = 52.57 - 1.541 \times 10^{-2}\, t_s - 2.155 \times 10^{-5}\, t_s^2 \text{ (W/m°C)} \qquad (4)$$

Poisson's ratio for steel may be taken as 0.3 and the density of steel may be taken as 7850 kg/m^3. Both may be assumed to be temperature independent. 2.1(d)

3.2 Strength reduction factors for structural steels

An important parameter defining the strength of steel at a particular temperature is the strength reduction factor; that is the residual strength relative to the basic yield strength of steel at room temperature. The determination of appropriate design strengths at elevated temperatures has been the subject of considerable debate in recent years. This is complicated by two key factors; firstly, the method of test and the heating rate used, and secondly, the strain limit at which the steel strength is determined. 2.2

Consider firstly, the method of test:

Isothermal or steady-state tests are ones that have been traditionally used for mechanical engineering applications where the tensile specimen is subject to constant temperature and further strain is applied at a steady rate. The stress-strain curve is therefore appropriate for a given constant temperature.

Anisothermal or transient tests are ones where the specimen is subject to constant load and the rate of heating is set at a pre-determined amount. The resulting strains are measured. The effect of thermal strains are deducted by using 'dummy' unloaded specimens subject to the same temperature conditions. Stress-strain curves at a particular temperature are obtained by interpolation from a family of curves at different stresses. A typical anisothermal test is shown in Figure 3.3.

Both methods of test have been used and the differences between them have been reviewed recently by Kirby and Preston(11). Isothermal tests are carried out at a relatively rapid strain rate and appear to provide more beneficial results than anisothermal tests. However, there is a slight dependence of anisothermal tests on rate of heating. The reference heating rate is taken as 10°C/minute (i.e. 600°C rise in 60 minutes). The faster the rate of heating in anisothermal tests, the lower the resulting strains in the steel for a given temperature and applied stress. This means that for a given strain, higher strengths are recorded at a given temperature for faster rates of heating.

Anisothermal tests result in lower strengths than isothermal tests but can be claimed to be more realistic. This difference between the two methods of test is smaller, but nevertheless significant at higher strains (>1%) than at lower strains. The difference between the methods is apparently less for grade 50 than grade 43 steel.

Consider, secondly, the strain limit for steel: The ECCS Recommendations(9) use an effective yield strain of 0.5% for temperatures exceeding 400°C, the effective yield strain reducing linearly with decreasing temperature to the 0.2% proof strain at 20°C. The yield strain is traditionally defined as the value consistent with a yield plateau for mild steels, but at elevated temperatures there is a gradual increase of strength with increasing strain (or strain-hardening). In fire tests on beams and columns very high strains are

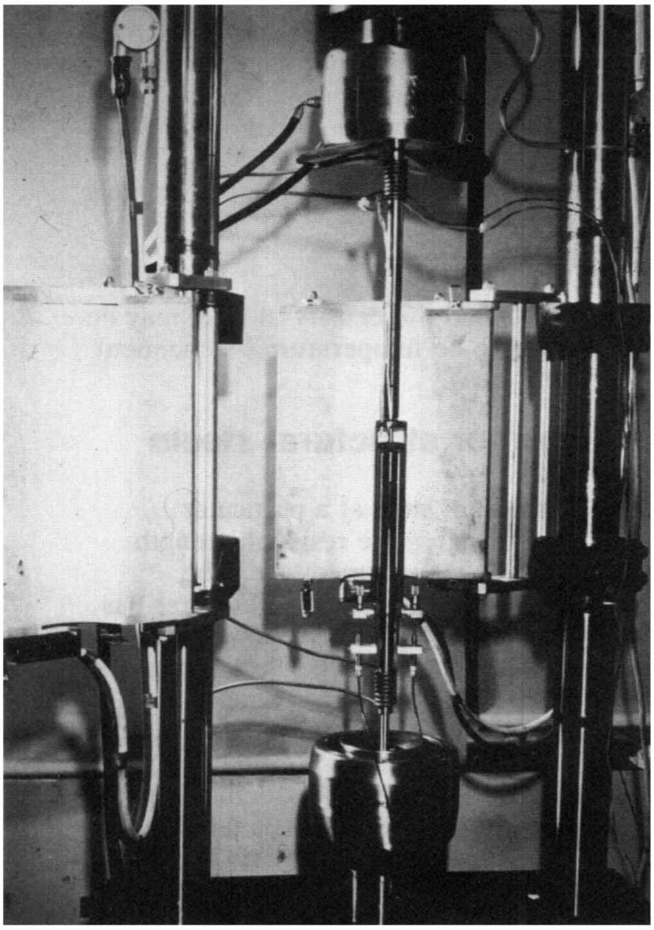


Figure 3.3 *Anisothermal test to determine stress-strain properties of steel*

experienced and this suggests that strengths greater than those at 0.5% strain are developed. Unlike most structural design under normal loading, the deformation of members in fire is of less importance than their strength and so the magnitude of the strain limit is not critical to the safety of the structure.

Such differences in the interpretation of the methods of test and the selection of the strain limit are not generally important for insulated sections because they contribute to a relatively small difference in the required thickness of fire protection. The differences can, however, be significant for unprotected sections where a fire resistance period of 30 minutes is often sought.

Empirical formulae are given in the ECCS Recommendations(9) for the variation of effective yield stress (at the above yield strains) with temperature. The ECCS data is conservative relative to the anisothermal data produced by British Steel, which is now embodied in Part 8 of *BS 5950*. However, in the proposed *Eurocode 3* Annex on fire resistance of steel, it has been accepted that the data produced by British Steel will replace that of the ECCS(9). The proposed EC3 data relates to a 2% strain limit.

The strength reduction factor (more correctly the strength 'retention' factor) defines the strength of steel at a particular temperature and 'mechanical' strain relative to its room temperature yield strength. The strength reduction factors at given steel strains, are presented for comparative purposes in Figure 3.4. The relative importance of the strain limit is apparent from this data.

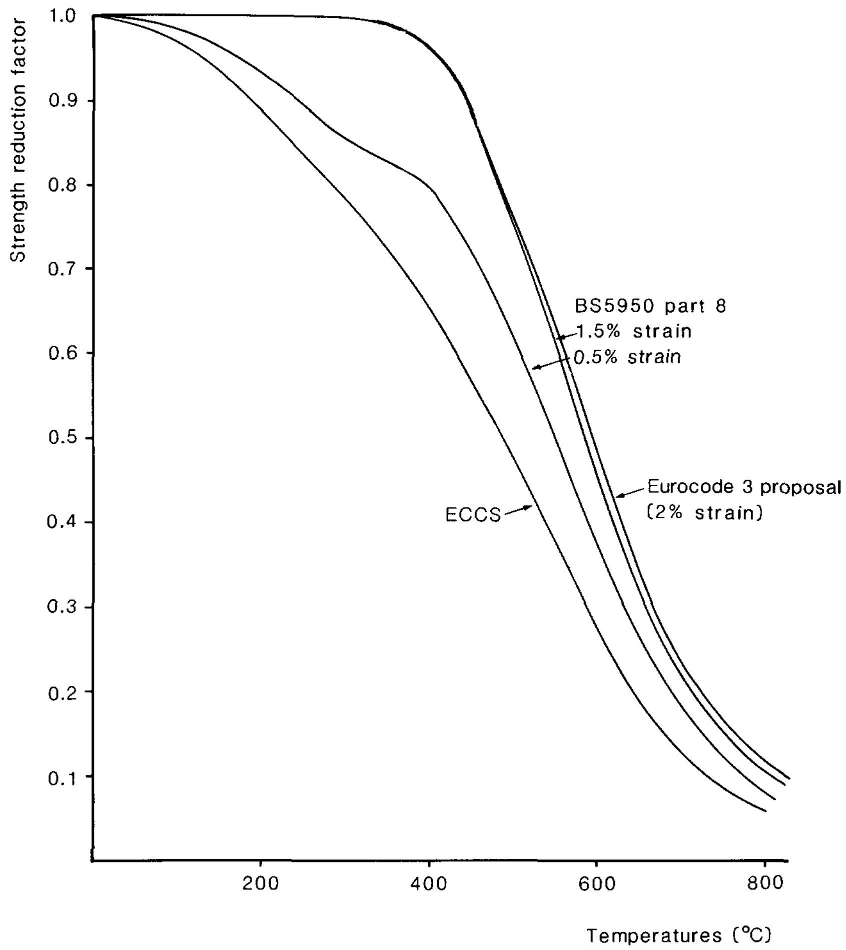


Figure 3.4 *Strength reduction factor for grade 43 steel at elevated temperatures*

The British Steel data is consistent with a heating rate of 10°C/minute. Elevated stress-strain data for *BS 4360: 1979* grade 43A and grade 50B steel are given in Tables 3.1 and 3.2[11]. Full data are presented for strains up to 2%, whereas in part 8, only data for 0.5%, 1.5% and 2% strain are given. The significance of this is explained in the following section. 2.2 Table 1

Similar data can be obtained for other heating rates[11]. Typically, the temperature at which 1% strain is achieved varies relative to a heating rate of 10°C/minute by the following amounts when using Tables 3.1 and 3.2:

20°C/minute	+15°C
5°C/minute	−15°C
2.5°C/minute	−25°C

This means that the faster the rate of heating, the higher the temperature at which a particular steel strength is attained at a certain strain.

For elements such as stocky columns, whose physical behaviour does not change in a fire, the strength reduction factor defines the load that may be applied to the section relative to that at room temperature. This may be termed the 'load-ratio' which is discussed in more detail in Section 5.3. The strength reduction factors for all grades of structural steel are similar (see Section 3.4 for comments on other forms of steel).

The use of the British Steel data in *BS 5950: Part 8* is justified by its better correlation with large scale beam and column tests, both in terms of the heating rates experienced and also the strains developed at the deflection limits imposed by the fire resistance test (designed in Section 3.1).

Table 3.1 *Elevated temperature stress – strain data for grade 43A steel*

Strain %	Temperature in degrees centigrade																			
	20	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	•	*
0.01	18.4	18.4	18.4	17.3	16.5	15.7	15.7	14.6	13.2	11.8	9.3	6.9	5.5	4.1	1.9	1.9	1.6	*	*	•
0.02	36.9	36.9	35.8	35.2	33.0	31.9	31.4	28.9	26.7	23.4	19.0	13.5	11.6	8.3	4.1	3.9	3.3	*	*	*
0.03	55.3	54.4	54.4	53.6	49.8	47.6	46.7	43.5	39.9	35.2	28.6	20.4	17.1	12.4	6.1	5.8	5.5	•	*	*
0.04	73.7	72.9	72.0	70.9	66.3	63.5	62.4	57.8	53.3	46.7	38.2	27.0	22.5	16.5	8.3	8.0	7.4	*	•	•
0.05	92.1	89.9	90.5	88.6	82.8	79.2	78.1	72.3	66.8	58.6	47.6	33.8	28.3	20.6	10.4	9.9	9.3	*	*	•
0.06	110.6	109.7	107.8	106.2	99.0	95.2	93.8	86.9	80.0	70.1	56.9	40.7	34.1	24.8	12.4	11.8	11.3	*	•	•
0.07	128.7	127.3	126.2	123.8	115.5	111.1	109.4	101.2	93.2	82.0	66.8	47.3	39.6	28.9	14.6	13.8	12.9	*	•	•
0.08	147.1	145.5	143.8	142.2	132.3	126.8	124.8	115.8	106.4	93.5	76.2	54.2	45.1	32.7	16.5	15.4	14.3	•	*	*
0.09	165.8	163.9	162.3	159.5	148.8	142.7	133.7	125.4	117.4	105.3	85.5	60.8	48.9	36.0	18.7	16.5	14.6	*	*	*
0.10	184.3	182.3	179.8	177.1	165.3	158.7	140.3	132.0	123.8	112.8	95.2	67.7	52.3	38.5	20.6	17.9	14.9	*	•	*
0.12	221.1	218.3	215.6	212.3	198.3	183.4	152.4	144.6	136.9	122.7	104.5	81.4	58.0	42.9	24.8	19.8	15.4	*	*	*
0.14	256.6	249.7	238.4	230.7	212.6	194.7	161.4	154.3	147.1	131.4	113.6	88.6	63.5	45.9	27.2	21.2	15.4	*	*	*
0.16	264.0	256.9	249.1	241.2	223.3	205.1	169.4	162.3	154.8	138.6	119.9	95.7	67.7	48.7	29.1	22.5	15.7	•	•	•
0.18	271.4	264.5	259.6	248.9	231.6	211.8	176.0	168.9	161.7	144.9	126.2	101.5	71.8	51.1	31.1	23.4	16.0	*	•	•
0.20	275.0	266.8	262.6	251.4	237.1	216.4	182.3	174.9	167.8	149.9	131.4	106.2	75.6	53.1	32.7	24.5	16.2	•	*	*
0.25	275.0	267.3	264.5	255.5	246.7	224.4	195.3	188.1	180.7	161.4	142.4	113.8	82.8	58.3	37.1	26.7	16.8	*	•	•
0.30	275.0	267.6	265.1	258.2	252.2	230.4	206.8	199.4	191.7	171.1	151.3	119.3	89.4	63.0	41.0	28.9	17.3	•	*	*
0.35	275.0	267.6	265.4	260.1	256.0	234.3	215.3	207.9	200.5	179.6	157.8	124.6	94.3	67.4	44.3	30.8	17.9	*	•	•
0.40	275.0	267.6	265.6	261.3	258.0	237.1	223.3	215.3	207.4	186.5	163.4	128.7	98.4	70.4	47.0	32.5	18.4	•	•	*
0.50	275.0	267.9	265.9	263.2	260.1	243.1	234.8	227.1	219.4	198.3	171.1	135.3	104.0	74.0	51.1	34.9	19.5	*	*	*
0.60	275.0	267.9	265.9	263.5	260.7	248.1	243.4	235.7	228.0	207.6	178.5	142.2	108.4	76.4	53.3	36.0	20.6	13.8	*	•
0.70	275.0	267.9	266.2	263.7	261.3	251.9	248.9	242.8	236.8	215.6	185.9	148.2	111.9	78.6	54.7	36.9	21.7	14.9	•	*
0.80	275.0	267.9	266.2	264.0	261.8	256.0	253.8	249.7	245.3	222.2	191.1	153.2	115.8	80.8	55.8	37.7	22.8	15.7	•	•
0.90	275.0	267.9	266.2	264.3	262.1	259.6	257.4	254.1	251.1	227.4	195.5	157.3	118.8	82.5	56.9	38.2	23.9	16.5	•	•
1.00	275.0	267.9	266.2	264.5	262.6	261.8	260.1	258.5	255.5	231.6	198.6	160.6	120.4	83.9	57.8	39.1	25.0	17.3	*	*
1.20	*	*	*	•	•	•	264.8	262.9	259.1	238.7	202.9	164.4	123.2	86.3	59.4	40.2	27.2	18.7	•	•
1.40	*	*	*	*	*	*	*	265.1	261.8	244.2	206.3	167.2	125.4	88.6	60.8	41.3	29.1	19.8	16.0	•
1.60	*	*	•	*	*	*	*	*	264.0	249.4	209.0	169.4	127.3	90.5	61.9	42.1	30.3	20.6	16.5	•
1.80	*	*	•	*	*	*	*	*	265.6	253.8	211.5	171.1	129.0	91.8	63.0	42.9	31.1	21.5	16.8	•
2.00	*	*	•	*	*	*	*	•	•	256.9	213.4	172.4	130.3	92.7	63.8	43.5	31.6	21.7	17.1	14.3

Table 3.2 *Elevated temperature stress – strain data for grade 50B steel*

Strain %	Temperature in degrees centigrade																			
	20	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	*	*
0.01	18.8	18.8	18.8	17.8	16.3	15.6	15.6	14.6	13.1	11.7	9.6	6.7	5.7	4.3	2.1	2.1	1.8	*	*	*
0.02	37.6	37.6	36.6	35.9	33.0	31.6	31.2	28.8	26.6	23.4	18.8	13.5	11.4	8.2	4.3	3.9	3.9	*	*	*
0.03	56.1	55.4	55.4	54.3	49.3	47.2	46.5	43.3	39.8	35.1	28.4	20.2	17.0	12.4	6.0	6.0	5.7	*	*	*
0.04	74.9	74.2	73.1	72.4	65.7	63.2	62.1	57.5	53.3	46.5	38.0	27.0	22.4	16.3	8.2	7.8	7.5	*	*	*
0.05	93.7	92.7	91.9	90.2	82.4	78.8	77.7	72.1	66.4	58.2	47.6	33.7	28.0	20.6	10.3	9.9	9.2	*	*	*
0.06	112.5	111.5	109.7	107.9	98.7	94.8	93.4	86.6	79.5	69.9	56.8	40.5	33.7	24.5	12.4	11.7	11.4	*	*	*
0.07	131.0	129.2	128.5	125.7	115.0	110.4	109.0	100.8	93.0	81.7	66.4	47.2	39.4	28.8	14.2	13.8	13.1	*	*	*
0.08	149.8	148.0	146.3	144.5	131.7	126.4	124.3	115.4	106.1	93.4	76.0	54.0	45.1	32.7	16.3	15.6	14.9	*	*	*
0.09	168.6	166.9	165.1	162.2	148.0	142.0	139.9	129.6	119.3	105.1	85.2	60.7	50.8	36.9	18.5	17.8	16.7	*	*	*
0.10	187.4	185.7	182.8	180.3	164.4	158.0	155.1	143.8	132.4	116.4	94.8	67.4	56.4	40.8	20.6	19.5	18.5	*	*	*
0.12	224.7	222.2	219.4	215.8	197.4	189.6	183.2	171.1	159.0	139.5	113.6	80.9	66.0	49.0	24.5	22.0	19.9	*	*	*
0.14	262.3	259.5	256.0	252.4	230.4	221.2	198.8	188.5	177.9	158.0	132.8	94.4	74.2	53.6	28.8	24.5	19.9	*	*	*
0.16	299.6	296.1	292.5	288.3	263.4	251.0	211.6	201.6	191.3	170.0	145.2	107.9	81.7	58.2	33.0	26.6	20.2	*	*	*
0.18	337.3	333.7	328.4	313.8	287.9	264.8	221.9	212.6	203.1	180.3	156.2	120.3	88.0	62.5	36.6	28.4	20.6	*	*	*
0.20	355.0	345.1	333.7	318.8	300.7	273.0	231.8	222.2	213.0	188.5	165.8	130.6	94.1	66.0	39.8	30.2	20.9		*	*
0.25	355.0	348.6	338.7	326.6	316.3	287.2	250.6	241.8	232.5	206.3	182.8	145.6	105.1	74.5	46.2	33.7	21.7		*	*
0.30	355.0	349.7	341.2	332.6	325.2	296.8	267.0	257.0	247.4	220.5	195.3	154.1	115.0	81.3	51.5	36.6	22.4		*	*
0.35	355.0	350.0	342.6	336.5	330.5	302.5	278.0	268.4	258.8	231.8	203.8	160.8	121.8	87.0	57.2	39.8	23.1		*	*
0.40	355.0	350.0	344.0	338.7	333.0	306.0	288.3	278.0	267.7	240.7	210.9	166.1	127.1	90.0	60.7	41.9	23.8		*	*
0.50	355.0	350.4	345.4	340.8	335.8	313.8	303.2	293.2	283.3	256.0	220.8	174.7	134.2	95.5	66.0	45.1	25.2	*	*	*
0.60	355.0	350.4	345.8	341.2	336.5	320.2	314.2	304.2	294.3	268.0	230.4	183.5	139.9	98.7	68.9	46.5	26.6	17.8	*	*
0.70	355.0	350.7	345.1	341.9	337.3	325.2	321.3	313.5	305.7	278.3	240.0	191.3	144.5	101.5	70.6	47.6	28.0	19.2	*	*
0.80	355.0	350.7	346.5	342.2	338.0	330.5	327.7	322.3	316.7	286.8	246.7	197.7	149.5	104.4	72.1	48.6	29.5	20.2	*	*
0.90	355.0	350.7	346.8	342.6	338.3	335.1	332.3	328.0	324.1	293.6	252.4	203.1	153.4	106.5	73.5	49.3	30.9	21.3	*	*
1.00	355.0	351.1	347.2	343.3	339.0	338.0	335.8	333.7	329.8	298.9	256.3	207.3	155.5	108.3	74.5	50.4	32.3	22.4	*	*
1.20	*	*	*	*	*	*	341.9	339.4	334.4	308.1	262.0	212.3	159.0	111.5	76.7	51.8	35.1	24.1	*	*
1.40	*	*	*	*	*	*	*	342.2	338.0	315.2	266.3	215.8	161.9	114.3	78.5	53.3	37.6	25.6	20.6	*
1.60	*	*	*	*	*	*	*	*	340.8	322.0	269.8	218.7	164.4	116.8	79.9	54.3	39.0	26.6	21.3	*
1.80	*	*	*	*	*	*	*	*	342.9	327.7	273.0	220.8	166.5	118.6	81.3	55.4	40.1	27.7	21.7	*
2.00	*	*	*	*	*	*	*	*	*	331.6	275.5	222.6	168.3	119.6	82.4	56.1	40.8	28.0	22.0	18.5

3.3 Selection of appropriate strain limits

In the UK, loaded fire tests on protected beams have generally been carried out using a 305 × 127 Universal Beam or similar sized section over a 4.5 m span[12]. The ratio of span to depth of such a member is consistent with the range of 15 to 20 found in buildings. A concrete slab is normally laid on the top flange, but precautions are taken to avoid composite action by placing the concrete in segments with small gaps between filled with mineral fibre. The concrete slab partially insulates the upper flange of the beam. The beam is loaded by two or four point loads as a simulated uniform load, as shown in Figure 3.5.

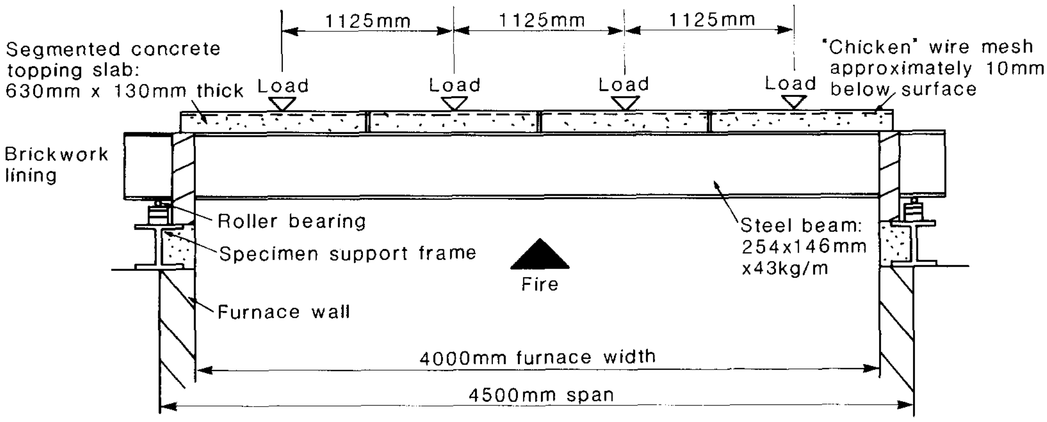


Figure 3.5 *Test arrangement for fire test on simply supported beam*

In tests on bare steel beams, high strains are developed. At a deflection of span/30, strains well in excess of 3% have been measured. However, it should be noted that this deflection also includes a component arising from thermally-induced curvature. Consequently, the load-induced or 'mechanical' strains (consistent with those given in Section 2.2) are smaller, but typically of the order of 2 to 3%.

Therefore, a strain limit of 1.5% has been selected as being representative for the design of bare beams. This limit is less than the strains occurring in the tests and takes account of the use of beams of different proportions other than those tested. The strength reduction factor at this strain is presented as a function of temperature in Figure 3.4. It follows that the strengths appropriate to this strain limit are conservative but give reasonable correlation with simply-supported test results (see Section 6.1). 2.3 (b)

Higher strains are developed in composite beams than in steel beams at a given deflection. Consequently, the strain limit is increased to 2% in composite beams (see Section 9.5). 2.3 (a)

Fire protected beams behave in a similar manner, except that at large deformations (and hence strains) there is a possibility that cracks may open up or, in extreme cases, the protection may become detached. One criterion imposed in the certification of fire-protection materials is an indicative beam test to assess 'stickability'. This uses a similar beam arrangement to that noted above. The test is normally terminated at a deflection of between span/40 and span/30. At a deflection of span/40 it would be reasonable to expect that flange strains exceeding 1.5% had been experienced in a beam of normal proportions (see Section 8.2.3).

Therefore, provided that the fire protective materials have demonstrated their 'stickability' by remaining intact up to the above order of beam deformations, then the 1.5% strain limit may be used in assessing the strength of the steel section.

Failure to meet this stickability criterion implies that the deformation capacity of the protective system is relatively low, leading to lower strain limits in the steel. A strain limit of 0.5% is appropriate in such cases. 2.3 (c)

Manufacturers, therefore, are encouraged to carry out the above beam tests to a deflection of span/30 in order to justify the use of the higher strain limit of 1.5% (rather than 0.5%). This leads to slightly higher strengths in the important temperature range of 400° to 600°C and results in a smaller thickness of fire protection for the same fire resistance (refer to Section 5.). Tests should be carried out on a representative beam section for the span under consideration.

Load tests on columns behave rather differently to beams in that relatively low strains are experienced at failure. In this case, a strain limit of 0.5% is appropriate for all forms of fire protection (Figure 3.4). A discussion of the effect of column slenderness is covered in Section 6.4. 2.3 (c)

A similar strain limit is used for tension members which are subject to uniform axial strain. Higher strains could lead to excessive movements and could effect the overall structural performance of the building where the tension members are used as ties.

3.4 Behaviour of other steels and materials in fire

3.4.1 Cold formed steel

The anisothermal high temperature properties of galvanized cold form steel to *BS 2989* (Z22 to Z35) have been determined by British Steel Strip Products for up to 600°C as in Table 3.3. The fire behaviour, in terms of the stress reduction factor, is similar for all the above grades of cold formed steel. The effect of the loss of cold working strength can be seen from the more rapid loss of strength relative to hot-rolled steel at temperatures exceeding 300°C. Appendix B Table 15

Table 3.3 *Strength reduction factors for structural galvanised steels to BS 2989*

Strain %	Temperature °C								
	200	250	300	350	400	450	500	550	600
0.5	0.945	0.890	0.834	0.758	0.680	0.575	0.471	0.370	0.269
1.5	1.0	0.985	0.949	0.883	0.815	0.685	0.556	0.453	0.349
2.0	1.0	1.0	1.0	0.935	0.867	0.730	0.590	0.490	0.390

However, the data in Table 3.3 obtained for cold formed steel is based on a 95% confidence limit, i.e. a 5% chance that the strength of the steel will fall below this data. The 95% confidence limit is therefore below the mean value. The data obtained for hot rolled steel is not statistically based and may be considered to be more representative of the mean data. In comparison to grade 43A steel, there is a marked difference in the performance of cold formed steels at higher strains in the important temperature range from 450° to 650°C. The limiting temperatures for cold formed steel sections will, in general, be lower than those for hot rolled sections.

3.4.2 Reinforcing steel

The isothermal high temperature properties of hot and cold worked reinforcing bars (to *BS 4449*) and prestressing wires (to *BS 5896*) have been determined by Holmes et. al.[13] in terms of yield strength (defined at 0.2% proof strain) and ultimate strength (consistent with a higher strain of 1 to 2%).

The data presented in this paper[13] are consistent with the simplified approach in the publication *Design and detailing of concrete structures for fire resistance*[14].

The strength reduction factor for hot-rolled reinforcing bars at temperature T (°C) is defined as:

$$S = 0.95(800-T)/500 + 0.05 \quad \text{for } 350 < T < 800°C \qquad (5)$$

A residual strength reduction factor of 0.05 for T>800°C may be used at higher temperatures. The behaviour of cold-worked bars is similar but there is a more marked loss of strength above 600°C. The strength reductions for prestressing wires are much greater (hence lower value of S) than for hot-rolled bars.

3.4.3 Stainless steel

Stainless steel can have varying properties depending on its composition. Modern austenitic stainless steels have the properties given below in Table 3.4. The coefficient of thermal expansion is greater, the thermal conductivity is lower and the specific heat is similar to carbon steels.

The strength reduction factors, determined from isothermal tests, are given in Table 3.4 for the common 316 S31 and 304 S16 grades[15]. This data may be unconservative relative to anisothermal tests (see section 3.2). It is apparent that the loss in strength at temperatures as low as 100°C is considerable, and that the strength reduction factor of 0.5 is reached at temperatures of about 400°C. This suggests that great care should be taken in the design of stainless steel structural elements in areas where they may be exposed to fire.

3.4.4 Cast and wrought iron

Cast iron is a brittle and variable quality material and is not able to undergo distortion in a fire because of its weakness in tension. Failure often occurs as a result of secondary effects such as thermal expansion of a floor, or differential heating or rapid cooling. The elevated temperature strength properties of cast iron in compression are similar to those of mild steel

Wrought iron may also be variable in quality but is relatively ductile. Wrought iron loses strength more rapdily than mild steel at temperatures above 450°C. Data on the performance of cast and wrought iron is given in reference[16].

3.4.5 Bolts and welds

The two main types of bolts are grade 4.6 and 8.8 to *BS 3672*. Grade 4.6 bolts are forged from mild steel, whereas grade 8.8 bolts are manufactured from micro-alloy steel which is quenched and tempered to obtain its higher strength. The margin between the 0.2% proof stress and the tensile strength is much lower in grade 8.8 than grade 4.6 bolts. The behaviour of these bolts at elevated temperatures is described in Section 12.2.2. This is the subject of continuing research at British Steel.

The strength of welds may decrease markedly in the temperature range of 200 to 400°C, but then stabilizes to close to the strength of the parent steel. However, there is relatively little data on the behaviour of welds to give definitive guidance. Consequently, the Code takes the strength of welds as 80% of the equivalent value for structural steel at 0.5% strain.

3.4.6 Normal and lightweight concrete

Concrete loses strength less rapidly at high temperatures than steel but suffers from the phenomenon of spalling, that is the breaking away of the concrete cover to the steel reinforcement. For bending elements, it is the heating up and loss of strength of the reinforcement that determines the fire resistance.

Table 3.4 *Strength reduction factors and physical properties of stainless steels (austenitic grades)*

1. Grade 316S31

Strain %	Temperature °C																			
	20	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000
0.5	1.000	0.773	0.700	0.645	0.600	0.567	0.536	0.518	0.495	0.477	0.467	0.454	0.445	0.436	0.427	0.391	0.309	0.236	0.218	0.127
1.0	1.000	0.787	0.717	0.662	0.625	0.569	0.571	0.554	0.537	0.521	0.504	0.496	0.483	0.471	0.446	0.383	0.300	0.225	0.167	0.125

2. Grade 304S16

Strain %	Temperature °C																	
	20	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900
0.5	1.000	0.752	0.662	0.609	0.571	0.543	0.519	0.500	0.481	0.467	0.452	0.438	0.419	0.400	0.381	0.343	0.305	0.214
1.0	1.000	0.774	0.696	0.639	0.604	0.574	0.543	0.522	0.509	0.500	0.487	0.474	0.452	0.430	0.413	0.348	0.304	0.217

3. Physical Properties Data

	Temperature °C								
	20	100	200	300	400	500	600	700	800
Thermal Expansion $\times 10^{-5}$/°C	–	1.60	1.66	1.72	1.77	1.83	1.87	1.91	1.93
Thermal Conductivity W/m°C	14.3	15.5	17.0	18.4	20.0	21.5	23.0	24.5	26.0
Specific Heat J/kg°C	–	499	510	522	533	544	553	560	563
Poisson's Ratio					0.31				

Hence, it is the insulation and protection offered by the concrete cover to the reinforcing bars that is important. Specified concrete covers increase with fire resistance period and supplementary mesh is required in normal weight concrete when covers exceed 40 mm. This also has implications for the design of concrete encased steel columns.

Design of reinforced concrete is covered by *BS 8110,* and Part 2 deals in more detail with the fire resistance aspects. The strength properties of normal (NWC) and lightweight (LWC) concrete are given in reference (14). This data is needed when considering the behaviour of composite deck slabs (Section 9), and concrete filled hollow sections (Section 11).

In simple terms, the strength reduction factor for NWC is:

$$S = 0.8(800-T)/450+0.2 \quad \text{for } 350<T<800°\text{C} \qquad (6)$$

Lightweight concrete has beneficial properties in fire because of its lower thermal conductivity (and hence better insulating capacity) and higher strength in fire (relative to normal weight concrete). The dry density range of structural lightweight concrete (Lytag) is 1750 to 1900 kg/m^3.

The strength reduction factor for LWC is:

$$S = 0.6(800-T)/300+0.4 \quad \text{for } 500<T<800°\text{C}. \qquad (7)$$

3.4.7 Brick and brickwork

The thermal properties of masonry are covered in BCRA Technical Note 333(17). These properties are important because steel columns are often partially or fully protected by masonry walls (Section 4.4).

The thermal conductivity of most clay bricks is less than normal weight concrete, but the specific heat is similar. There is little loss of strength at high temperatures. The ties between the structure and the brickwork should be well-insulated from the effects of a fire in order to maintain the lateral stability of masonry cladding. This is normally achieved by the inner leaf of a masonary wall.

4. THERMAL RESPONSE OF PROTECTED AND UNPROTECTED STEEL MEMBERS

This Section deals with the rise in temperature of steel members in a fire. Data on the thermal response of steel sections is a prerequisite to the use of the moment capacity and fire engineering methods treated later. Steel sections may be unprotected, in which case the temperature rise is relatively rapid, or protected by materials of low thermal conductivity, so that the temperature rise in the steel is less rapid.

There are also many examples of partial-protection of steel sections, so that part of the section is exposed to the fire and the remaining part is protected. Common examples are beams that support concrete floors, and columns that provide lateral support to masonry walls. Partial protection can be responsible for a considerable enhancement in the fire resistance of the member. (See Section 4.6).

4.1 Section factors

The controlling parameter that determines the temperature rise in a steel member is the ratio of the heated perimeter H_p to the cross-sectional area A of the member. This ratio H_p/A is normally presented in units of m^{-1} and is termed the 'section factor'. Typical values of section factor are between 100 and 250 m^{-1} for the normal size of hot-rolled sections. Sections with low H_p/A factors respond more slowly to heat and therefore achieve higher periods of fire resistance than sections with high H_p/A factors. Some unprotected sections with very low section factor heat up so slowly that they can achieve 30 minutes fire resistance (see Section 5.2). 4.2

The definition of the heated perimeter of an unprotected member is relatively straightforward.

For a fully exposed I section:

$$H_p = (4B + 2D - 2t)$$

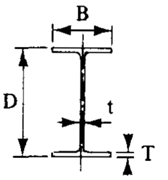
where B and D are the overall breadth and depth of the section and t is the web thickness.

Where a beam supports a floor and it is assumed that the floor material is of such a low conductivity that heat does not pass through the floor into the upper surface of the flange, H_p reduces to $(3B+2D-2t)$. A suitable material would be concrete (for a floor or a wall), or brickwork (for a wall). In the case of timber floors, the heated perimeter for a fully-exposed member should be used.

Formulae for heated perimeters of various protected members are given in of *BS 5950: Part 8*. Published data also take into account the root fillets between the web and flange in rolled sections. The basic data for 3 and 4 sided exposure are given in Table 4.1 (taken from reference (2)). 4.2 Table 3

Table 4.1 (a) Universal beams

Universal beams



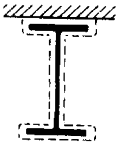
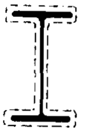
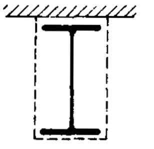
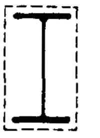
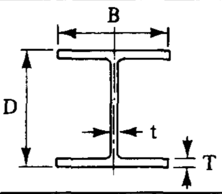
Designation		Depth of section D	Width of section B	Thickness		Area of section	Section factor H_p/A			
							Profile		Box	
							3 sides	4 sides	3 sides	4 sides
Serial size	Mass per metre			Web t	Flange T					
mm	kg	mm	mm	mm	mm	cm²	m^{-1}	m^{-1}	m^{-1}	m^{-1}
914×419	388	920.5	420.5	21.5	36.6	494.4	60	70	45	55
	343	911.4	418.5	19.4	32.0	437.4	70	80	50	60
914×305	289	926.6	307.8	19.6	32.0	368.8	75	80	60	65
	253	918.5	305.5	17.3	27.9	322.8	85	95	65	75
	224	910.3	304.1	15.9	23.9	285.2	95	105	75	85
	201	903	303.4	15.2	20.2	256.4	105	115	80	95
838×292	226	850.9	293.8	16.1	26.8	288.7	85	95	70	80
	194	840.7	292.4	14.7	21.7	247.1	100	115	80	90
	176	834.9	291.6	14	18.8	224.1	110	125	90	100
762×267	197	769.6	268	15.6	25.4	250.7	90	100	70	85
	173	762	266.7	14.3	21.6	220.4	105	115	80	95
	147	753.9	265.3	12.9	17.5	188.0	120	135	95	110
686×254	170	692.9	255.8	14.5	23.7	216.5	95	110	75	90
	152	687.6	254.5	13.2	21.0	193.8	110	120	85	95
	140	683.5	253.7	12.4	19.0	178.6	115	130	90	105
	125	677.9	253	11.7	16.2	159.6	130	145	100	115
610×305	238	633	311.5	18.6	31.4	303.7	70	80	50	60
	179	617.5	307	14.1	23.6	227.9	90	105	70	80
	149	609.6	304.8	11.9	19.7	190.1	110	125	80	95
610×229	140	617	230.1	13.1	22.1	178.3	105	120	80	95
	125	611.9	229	11.9	19.6	159.5	115	130	90	105
	113	607.3	228.2	11.2	17.3	144.4	130	145	100	115
	101	602.2	227.6	10.6	14.8	129.1	145	160	110	130
533×210	122	544.6	211.9	12.8	21.3	155.7	110	120	85	95
	109	539.5	210.7	11.6	18.8	138.5	120	135	95	110
	101	536.7	210.1	10.9	17.4	129.7	130	145	100	115
	92	533.1	209.3	10.2	15.6	117.7	140	160	110	125
	82	528.3	208.7	9.6	13.2	104.4	155	175	120	140
457×191	98	467.4	192.8	11.4	19.6	125.2	120	135	90	105
	89	463.6	192	10.6	17.7	113.9	130	145	100	115
	82	460.2	191.3	9.9	16.0	104.5	140	160	105	125
	74	457.2	190.5	9.1	14.5	94.98	155	175	115	135
	67	453.6	189.9	8.5	12.7	85.44	170	190	130	150
457×152	82	465.1	153.5	10.7	18.9	104.4	130	145	105	120
	74	461.3	152.7	9.9	17.0	94.99	140	155	115	130
	67	457.2	151.9	9.1	15.0	85.41	155	175	125	145
	60	454.7	152.9	8.0	13.3	75.93	175	195	140	160
	52	449.8	152.4	7.6	10.9	66.49	200	220	160	180
406×178	74	412.8	179.7	9.7	16.0	94.95	140	160	105	125
	67	409.4	178.8	8.8	14.3	85.49	155	175	115	140
	60	406.4	177.8	7.8	12.8	76.01	175	195	130	155
	54	402.6	177.6	7.6	10.9	68.42	190	215	145	170
406×140	46	402.3	142.4	6.9	11.2	58.96	205	230	160	185
	39	397.3	141.8	6.3	8.6	49.40	240	270	190	220
356×171	67	364	173.2	9.1	15.7	85.42	140	160	105	125
	57	358.6	172.1	8	13.0	72.18	165	190	125	145
	51	355.6	171.5	7.3	11.5	64.58	185	210	135	165
	45	352	171	6.9	9.7	56.96	210	240	155	185
356×127	39	352.8	126	6.5	10.7	49.40	215	240	170	195
	33	248.5	125.4	5.9	8.5	41.83	250	280	195	225
305×165	54	310.9	166.8	7.7	13.7	68.38	160	185	115	140
	46	307.1	165.7	6.7	11.8	58.90	185	210	130	160
	40	303.8	165.1	6.1	10.2	51.50	210	240	150	180
305×127	48	310.4	125.2	9.9	14.0	60.83	160	180	125	145
	42	306.6	124.3	8	12.1	53.18	180	205	140	160
	37	303.8	123.5	7.2	10.7	47.47	200	225	155	180
305×102	33	312.7	102.4	6.6	10.8	41.77	215	240	175	200
	28	308.9	101.9	6.1	8.9	36.30	245	275	200	225
	25	304.8	101.6	5.8	6.8	31.39	285	315	225	260
254×146	43	259.6	147.3	7.3	12.7	55.10	170	195	120	150
	37	256	146.4	6.4	10.9	47.45	195	225	140	170
	31	251.5	146.1	6.1	8.6	40.00	230	265	160	200
254×102	28	260.4	102.1	6.4	10.0	36.19	220	250	170	200
	25	257	101.9	6.1	8.4	32.17	245	280	190	225
	22	254	101.6	5.8	6.8	28.42	275	315	215	250
203×133	30	206.8	133.8	6.3	9.6	38.00	210	245	145	180
	25	203.2	133.4	5.8	7.8	32.31	240	285	165	210
203×102	23	203.2	101.6	5.2	9.3	29	235	270	175	210
178×102	19	177.8	101.6	4.7	7.9	24.2	265	305	190	230
152×89	16	152.4	88.9	4.6	7.7	20.5	270	310	190	235
127×76	13	127	76.2	4.2	7.6	16.8	275	320	195	240

Table 4.1 *(b) Universal columns*

Universal columns



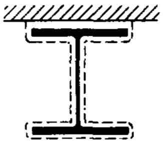
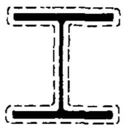
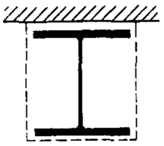
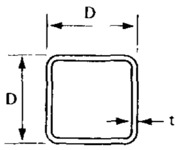
Designation: Serial size	Designation: Mass per metre	Depth of section D	Width of section B	Thickness: Web t	Thickness: Flange T	Area of section	Section factor H_p/A: Profile 3 sides	Section factor H_p/A: Profile 4 sides	Section factor H_p/A: Box 3 sides	Section factor H_p/A: Box 4 sides
										
mm	kg	mm	mm	mm	mm	cm²	m^{-1}	m^{-1}	m^{-1}	m^{-1}
356 × 406	634	474.7	424.1	47.6	77.0	808.1	25	30	15	20
	551	455.7	418.5	42.0	67.5	701.8	30	35	20	25
	467	436.6	412.4	35.9	58.0	595.5	35	40	20	30
	393	419.1	407.0	30.6	49.2	500.9	40	45	25	35
	340	406.4	403.0	26.5	42.9	432.7	45	55	30	35
	287	393.7	399.0	22.6	36.5	366.0	50	65	30	45
	235	381.0	395.0	18.5	30.2	299.8	65	75	40	50
356 × 368	202	374.7	374.4	16.8	27.0	257.9	70	85	45	60
	177	368.3	372.1	14.5	23.8	225.7	80	95	50	65
	153	362.0	370.2	12.6	20.7	195.2	90	110	55	75
	129	355.6	368.3	10.7	17.5	164.9	105	130	65	90
305 × 305	283	365.3	321.8	26.9	44.1	360.4	45	55	30	40
	240	352.6	317.9	23.0	37.7	305.6	50	60	35	45
	198	339.9	314.1	19.2	31.4	252.3	60	75	40	50
	158	327.2	310.6	15.7	25.0	201.2	75	90	50	65
	137	320.5	308.7	13.8	21.7	174.6	85	105	55	70
	118	314.5	306.8	11.9	18.7	149.8	100	120	60	85
	97	307.8	304.8	9.9	15.4	123.3	120	145	75	100
254 × 254	167	289.1	264.5	19.2	31.7	212.4	60	75	40	50
	132	276.4	261.0	15.6	25.3	167.7	75	90	50	65
	107	266.7	258.3	13.0	20.5	136.6	90	110	60	75
	89	260.4	255.9	10.5	17.3	114.0	110	130	70	90
	73	254.0	254.0	8.6	14.2	92.9	130	160	80	110
203 × 203	86	222.3	208.8	13.0	20.5	110.1	95	110	60	80
	71	215.9	206.2	10.3	17.3	91.1	110	135	70	95
	60	209.6	205.2	9.3	14.2	75.8	130	160	80	110
	52	206.2	203.9	8.0	12.5	66.4	150	180	95	125
	46	203.2	203.2	7.3	11.0	58.8	165	200	105	140
152 × 152	37	161.8	154.4	8.1	11.5	47.4	160	190	100	135
	30	157.5	152.9	6.6	9.4	38.2	195	235	120	160
	23	152.4	152.4	6.1	6.8	29.8	245	300	155	205

Table 4.1 *(c) Rectangular hollow sections (square)*

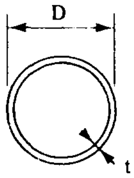
Rectangular hollow sections (square)



Designation: Size D × D	Designation: Thickness t	Mass per metre	Area of section	Section factor H_p/A: 3 sides	Section factor H_p/A: 4 sides
mm	mm	kg	cm²	m⁻¹	m⁻¹
20×20	2.0	1.12	1.42	425	565
	2.5	1.35	1.72	350	465
25×25	2.0	1.43	1.82	410	550
	2.5	1.74	2.22	340	450
	3.0	2.04	2.60	290	385
	3.2	2.15	2.74	275	365
30×30	2.5	2.14	2.72	330	440
	3.0	2.51	3.20	280	375
	3.2	2.65	3.38	265	355
40×40	2.5	2.92	3.72	325	430
	3.0	3.45	4.40	275	365
	3.2	3.66	4.66	260	345
	4.0	4.46	5.68	210	280
	5.0	5.40	6.88	175	235
50×50	2.5	3.71	4.72	320	425
	3.0	4.39	5.60	270	355
	3.2	4.66	5.94	255	335
	4.0	5.72	7.28	205	275
	5.0	6.97	8.88	170	225
	6.3	8.49	10.8	140	185
60×60	3.0	5.34	6.80	265	355
	3.2	5.67	7.22	250	330
	4.0	6.97	8.88	205	270
	5.0	8.54	10.9	165	220
	6.3	10.5	13.3	135	180
	8.0	12.8	16.3	110	145
70×70	3.0	6.28	8.00	260	350
	3.6	7.46	9.50	220	295
	5.0	10.1	12.9	165	215
	6.3	12.5	15.9	130	175
	8.0	15.3	19.5	110	145
80×80	3.0	7.22	9.20	260	350
	3.6	8.59	10.9	220	295
	5.0	11.7	14.9	160	215
	6.3	14.4	18.4	130	175
	8.0	17.8	22.7	105	140
90×90	3.6	9.72	12.4	220	290
	5.0	13.3	16.9	160	215
	6.3	16.4	20.9	130	170
	8.0	20.4	25.9	105	140
100×100	4.0	12.0	15.3	195	260
	5.0	14.8	18.9	160	210
	6.3	18.4	23.4	130	170
	8.0	22.9	29.1	105	135
	10.0	27.9	35.5	85	115
120×120	5.0	18.0	22.9	155	210
	6.3	22.3	28.5	125	170
	8.0	27.9	35.5	100	135
	10.0	34.2	43.5	85	110
	12.5	41.6	53.0	70	90
140×140	5.0	21.1	26.9	155	210
	6.3	26.3	33.5	125	165
	8.0	32.9	41.9	100	135
	10.0	40.4	51.5	80	110
	12.5	49.5	63.0	65	90
150×150	5.0	22.7	28.9	155	210
	6.3	28.3	36.0	125	165
	8.0	35.4	45.1	100	135
	10.0	43.6	55.5	80	110
	12.5	53.4	68.0	65	90
	16.0	66.4	84.5	55	70
180×180	6.3	34.2	43.6	125	165
	8.0	43.0	54.7	100	130
	10.0	53.0	67.5	80	105
	12.5	65.2	83.0	65	85
	16.0	81.4	104	50	70
200×200	6.3	38.2	48.6	125	165
	8.0	48.0	61.1	100	130
	10.0	59.3	75.5	80	105
	12.5	73.0	93.0	65	85
	16.0	91.5	117	50	70
250×250	6.3	48.1	61.2	125	165
	8.0	60.5	77.1	95	130
	10.0	75.0	95.5	80	105
	12.5	92.6	118	65	85
	16.0	117	149	50	65
300×300	10.0	90.7	116	80	105
	12.5	112	143	65	85
	16.0	142	181	50	65
350×350	10.0	106	136	75	105
	12.5	132	168	65	85
	16.0	167	213	50	65
400×400	10.0	122	156	75	105
	12.5	152	193	60	85
	16.0	192	245	50	65

Table 4.1 *(d) Circular hollow sections*

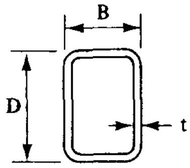
Circular hollow sections

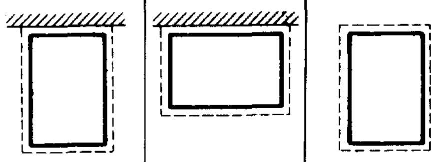


Designation: Outside diameter D (mm)	Designation: Thickness t (mm)	Mass per metre (kg)	Area of section (cm²)	Section factor H_p/A, Profile or Box (m^{-1})
21.3	3.2	1.43	1.82	370
26.9	3.2	1.87	2.38	355
33.7	2.6	1.99	2.54	415
	3.2	2.41	3.07	345
	4.0	2.93	3.73	285
42.4	2.6	2.55	3.25	410
	3.2	3.09	3.94	340
	4.0	3.79	4.83	275
48.3	3.2	3.56	4.53	335
	4.0	4.37	5.57	270
	5.0	5.34	6.80	225
60.3	3.2	4.51	5.74	330
	4.0	5.55	7.07	270
	5.0	6.82	8.69	220
76.1	3.2	5.75	7.33	325
	4.0	7.11	9.06	265
	5.0	8.77	11.2	215
88.9	3.2	6.76	8.62	325
	4.0	8.38	10.70	260
	5.0	10.3	13.2	210
114.3	3.6	9.83	12.5	285
	5.0	13.5	17.2	210
	6.3	16.8	21.4	170
139.7	5.0	16.6	21.2	205
	6.3	20.7	26.4	165
	8.0	26.0	33.1	135
	10.0	32.0	40.7	110
168.3	5.0	20.1	25.7	205
	6.3	25.2	37.1	165
	8.0	31.6	40.3	130
	10.0	39.0	49.7	105
193.7	5.0	23.3	29.6	205
	6.3	29.1	37.1	165
	8.0	36.6	46.7	130
	10.0	45.3	57.7	105
	12.5	55.9	71.2	85
	16.0	70.1	89.3	70
219.1	5.0	26.4	33.6	205
	6.3	33.1	42.1	165
	8.0	41.6	53.1	130
	10.0	51.6	65.7	105
	12.5	63.7	81.1	85
	16.0	80.1	102	65
	20.0	98.2	125	55
244.5	6.3	37.0	47.1	165
	8.0	46.7	59.4	130
	10.0	57.8	73.7	105
	12.5	71.5	91.1	85
	16.0	90.2	115	65
	20.0	111	141	55
273.0	6.3	41.4	52.8	160
	8.0	52.3	66.6	130
	10.0	64.9	82.6	105
	12.5	80.3	102	85
	16.0	101	129	65
	20.0	125	159	55
	25.0	153	195	45
323.9	6.3	49.3	62.9	160
	8.0	62.3	79.4	130
	10.0	77.4	98.6	105
	12.5	96.0	122	85
	16.0	121	155	65
	20.0	150	191	55
	25.0	184	235	45
355.6	8.0	68.6	87.4	130
	10.0	85.2	109	100
	12.5	106	135	85
	16.0	134	171	65
	20.0	166	211	55
	25.0	204	260	45
406.4	10.0	97.8	125	100
	12.5	121	155	80
	16.0	154	196	65
	20.0	191	243	55
	25.0	235	300	45
	32.0	295	376	35
457.0	10.0	110	140	105
	12.5	137	175	80
	16.0	174	222	65
	20.0	216	275	50
	25.0	266	339	40
	32.0	335	427	35
	40.0	411	524	25
508.0	10.0	123	156	100
	12.5	153	195	80
	16.0	194	247	65

Table 4.1 *(e) Rectangular hollow sections*

Rectangular hollow sections



Designation: Size D×B	Designation: Thickness t	Mass per metre	Area of section	Section factor H_p/A: 3 sides	Section factor H_p/A: 3 sides	Section factor H_p/A: 4 sides
						
mm	mm	kg	cm²	m^{-1}	m^{-1}	m^{-1}
50×25	2.5	2.72	3.47	360	290	430
	3.0	3.22	4.10	305	245	365
	3.2	3.41	4.34	290	230	345
50×30	2.5	2.92	3.72	350	295	430
	3.0	3.45	4.40	295	250	365
	3.2	3.66	4.66	280	235	345
	4.0	4.46	5.68	230	195	280
	5.0	5.40	6.88	190	160	235
60×40	2.5	3.71	4.72	340	295	425
	3.0	4.39	5.60	285	250	355
	3.2	4.66	5.94	270	235	335
	4.0	5.72	7.28	220	190	275
	5.0	6.97	8.88	180	160	225
	6.3	8.49	10.8	150	130	185
80×40	3.0	5.34	6.80	295	235	355
	3.2	5.67	7.22	275	220	330
	4.0	6.97	8.88	225	180	270
	5.0	8.54	10.9	185	145	220
	6.3	10.5	13.3	150	120	180
	8.0	12.8	16.3	125	100	145
90×50	3.0	6.28	8.00	290	240	350
	3.6	7.46	9.50	240	200	295
	5.0	10.1	12.9	180	145	215
	6.3	12.5	15.9	145	120	175
	8.0	15.3	19.5	120	95	145
100×50	3.0	6.75	8.60	290	235	350
	3.2	7.18	9.14	275	220	330
	4.0	8.86	11.3	220	175	265
	5.0	10.9	13.9	180	145	215
	6.3	13.4	17.1	145	115	175
	8.0	16.6	21.1	120	95	140
100×60	3.0	7.22	9.20	285	240	350
	3.6	8.59	10.9	240	200	295
	5.0	11.7	14.9	175	150	215
	6.3	14.4	18.4	140	120	175
	8.0	17.8	22.7	115	95	140
120×60	3.6	9.72	12.4	240	195	290
	5.0	13.3	16.9	180	140	215
	6.3	16.4	20.9	145	115	170
	8.0	20.4	25.9	115	95	140
120×80	5.0	14.8	18.9	170	150	210
	6.3	18.4	23.4	135	120	170
	8.0	22.9	29.1	110	95	135
	10.0	27.9	35.5	90	80	115
150×100	5.0	18.7	23.9	165	145	210
	6.3	23.8	29.7	135	120	170
	8.0	29.1	37.1	110	95	135
	10.0	35.7	45.5	90	75	110
	12.5	43.6	55.5	70	65	90
160×80	5.0	18.0	22.9	175	140	210
	6.3	22.3	28.5	140	110	170
	8.0	27.9	35.5	115	90	135
	10.0	34.2	43.5	90	75	110
	12.5	41.6	53.0	75	60	90
200×100	5.0	22.7	28.9	175	140	210
	6.3	28.3	36.0	140	110	165
	8.0	35.4	45.1	110	90	135
	10.0	43.6	55.5	90	70	110
	12.5	53.4	68.0	75	60	90
	16.0	66.4	84.5	60	45	70
250×150	6.3	38.2	48.6	135	115	165
	8.0	48.0	61.1	105	90	130
	10.0	59.3	75.5	85	75	105
	12.5	73.0	93.0	70	60	85
	16.0	91.5	117	55	45	70
300×200	6.3	48.1	61.2	130	115	165
	8.0	60.5	77.1	105	90	130
	10.0	75.0	95.5	85	75	105
	12.5	92.6	118	70	60	85
	16.0	117	149	55	45	65
400×200	10.0	90.7	116	85	70	105
	12.5	112	143	70	55	85
	16.0	142	181	55	45	65
450×250	10.0	106	136	85	70	105
	12.5	132	168	70	55	85
	16.0	167	213	55	45	65

There are three main types of fire protection that should be considered[2] (Figure 4.1):

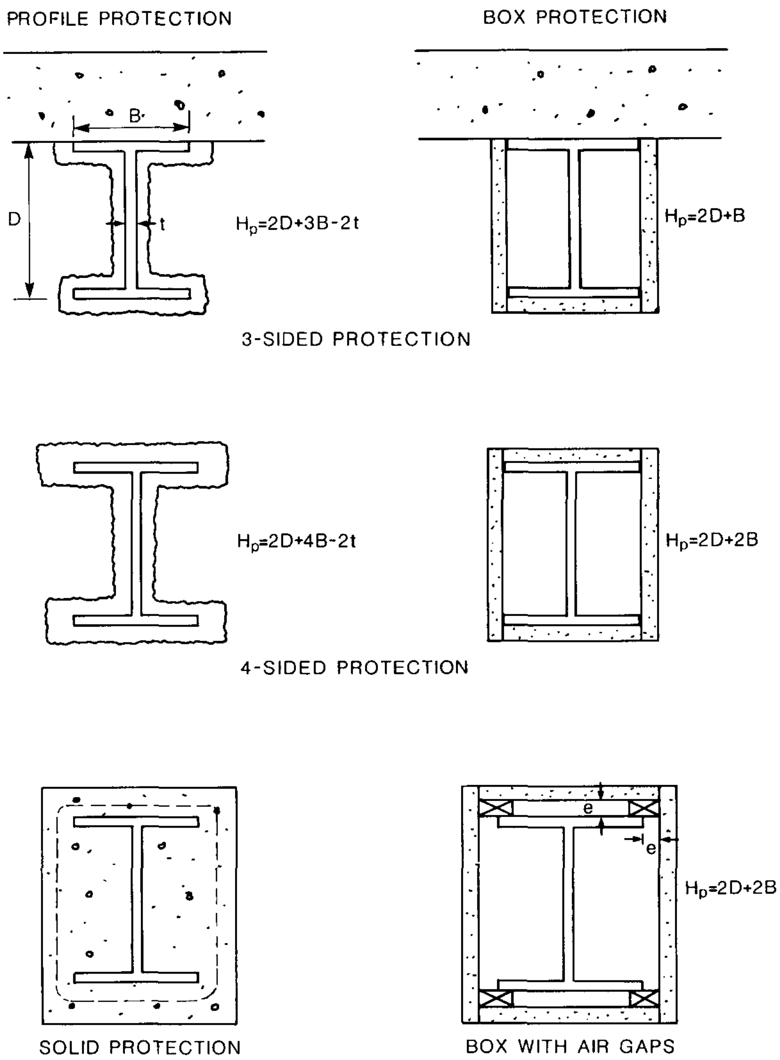


Figure 4.1 *Different forms of fire protection to I section members*

Profile protection is where the fire protection follows the surface of the member. Therefore the section factor relates to the proportions of the steel member.

Box protection is where there is an outer casing around the member. The heated perimeter is defined as the sum of the inside dimensions of the smallest possible rectangle, around the section, neglecting air gaps etc. (refer to Figure 4.1) The cross-sectional area, A, is that of the steel section. The thermal conductivity of the protection material is assumed to be much lower than that of steel and therefore, the temperature conditions within the area bounded by the box protection are assumed to be uniform.

Solid protection is where the member is encased (typically by concrete). This is a more complex case because of the non-uniform thermal profile through the concrete. If only part of the member is exposed (for example the lower flange), then the heated perimeter may be taken around the portion that is exposed. This assumes that the passage of heat through the concrete relative to the steel is small.

Examples of 'profile' and 'box' protection are shown in Figures 4.2 and 4.3.

Figure 4.2 *Profile fire protection to steel members*

Figure 4.3 *Box fire protection to steel members*

There are various other practical cases that are considered in the Code. Sections with openings, such as castellated beams may be considered as having the same section factor as the parent section. Rectangular and circular hollow sections are exposed to heat from only one side and therefore have a relatively low section factor for the same use of steel as an I section. 4.2.3

4.2 Theoretical behaviour of unprotected steel sections in fire

The incremental rise in temperature of a uniformly heated bare steel section in time interval δt is given in the reference (9) as:

$$\delta\theta_s = \frac{\alpha_c + \alpha_r}{C_s \rho_s} \frac{H_p}{A} \,.\, (\theta_f - \theta_s)\, \delta t \qquad (8)$$

where C_s and ρ_s are the specific heat and density respectively of steel. Values are given in Section 3.1.

θ_f = the temperature (°C) of the furnace (or temperature in a natural fire) at a particular time t (secs)

θ_s = the temperature of the steel section, (°C) which is assumed to be uniform, at time t

H_p/A = the section factor (m^{-1})

α_c = the coefficient of heat transfer by convection. (This is normally taken as a constant of 25 W/m^2 °C)

α_r = the coefficient of heat transfer by radiation, where

$$\alpha_r = \frac{5.67\ \varepsilon}{\theta_f - \theta_s} [(\theta_f + 273)^4 - (\theta_s + 273)^4] \times 10^{-8} \qquad (9)$$

The parameter ε is the resultant emissivity and represents the radiation transmitted between the fire and the metal surface and its magnitude depends on the degree of direct exposure of the element to the fire. Elements which are partially shielded from the radiant effects of the heat of the fire would have a lower value of ε. Conservatively, ε may be taken as 0.5.

Equation (9) is in terms of absolute temperature. Clearly, the larger the difference between θ_f and θ_s, the greater the heat transmitted by radiation between the fire and the steel surface.

This equation for the temperature rise may be used to determine steel temperatures by incremental integration, if the variation of the fire (or furnace) temperature with time is known. The rise in temperature of an unprotected section is of the form shown in Figure 4.4.

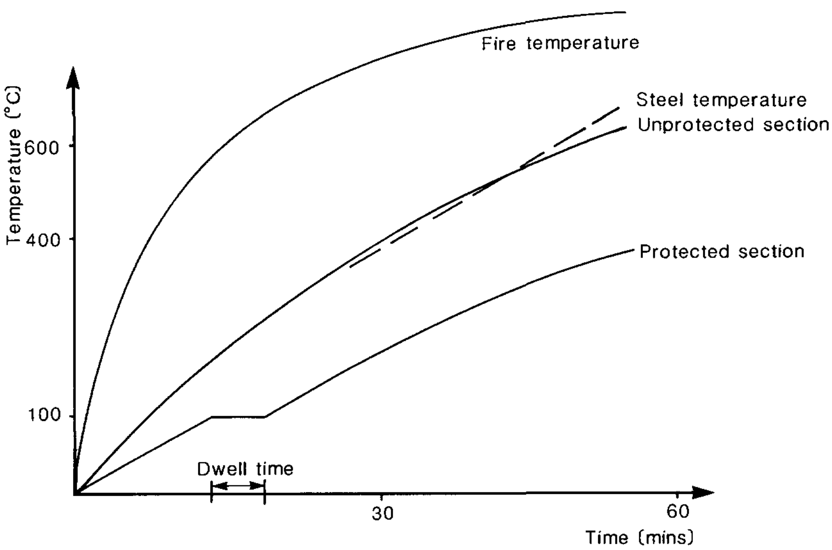


Figure 4.4 *Rate of heating in unprotected and protected sections*

An approximate formula for the time t_θ (now expressed in minutes) at which a limiting steel temperature of θ_s is achieved in an unprotected section is given in the ECCS Recommendations[9] as:

$$t_\theta = 0.54\,(\theta_s - 50)\,(H_p/A)^{-0.6} \tag{10}$$

This linear variation of θ_s with time is valid in the range of steel temperatures from 400° to 600°C. (See Figure 4.4).

4.3 Design temperatures in unprotected columns and beams

4.3.1 Temperatures in columns and beams

The thermal response of I section columns and beams has been measured in test furnaces and correlated with analytical methods. For reference purposes these temperatures may be presented in terms of the section factor (H_p/A) or, alternatively, flange thickness t_f.

Column members exposed to heat from all four sides may be considered to be uniformly heated. Therefore, the section factor gives a good measure of the average temperature rise in the member (following the approach of Section 4.2).

Beam members supporting concrete floors are subject to differential heating because the top flange is connected to the floor slab which acts as a 'heat-sink'. The behaviour of members in bending is more influenced by the temperature of the lower flange, and hence by the thickness of the lower flange than the section factor which is averaged over the cross-section.

The temperatures in the lower and upper flanges and at mid-height of the web of a 356 × 171 × 67 kg/m Universal Beam section supporting a concrete floor are presented in Figure 4.5. Both measured and computed temperatures are shown, but the standard furnace temperatures are omitted (see Figure 2.1). The section factor for this beam is 142 m^{-1} for three-sided heating (see Table 4.1).

The computed temperature of the upper flange in an unprotected member is generally some 200°C lower than that of the lower flange at 30 minutes in a fire resistance test, although this difference reduces with further exposure to the fire (compare Figures 4.5 (a) and (b)). Web temperatures are relatively constant below mid-height of the section and are the same as the lower flange temperature, but temperatures decrease above mid-height (compare Figures 4.5(a) and (c)). As an approximation to this effect, the temperature of the upper 100 mm of web may be considered to be 50°C lower than the remaining portion which is taken as at the same temperature as the lower flange.

The same form of temperature distribution occurs in protected beams, and indeed, the relative temperature difference between the flanges can be greater than in unprotected beams. This justifies the general use of the limiting temperature method (see Section 6) which is entirely based on the results of unprotected members.

4.3.2 Design temperatures of unprotected sections

Design temperatures are defined as those that the critical element (lower flange for beams) of an unprotected steel section would reach at the appropriate period in a standard fire resistance test. In the Code, design temperatures are presented as a function of flange thickness. For beams, in particular, flange thickness gives a slightly better measure of performance than the 'average' section factor. This data is presented in Table 4.2. for unprotected columns and tension members, and in Table 4.3. for unprotected beams supporting concrete floors.

4.4.3
Table 6
Table 7

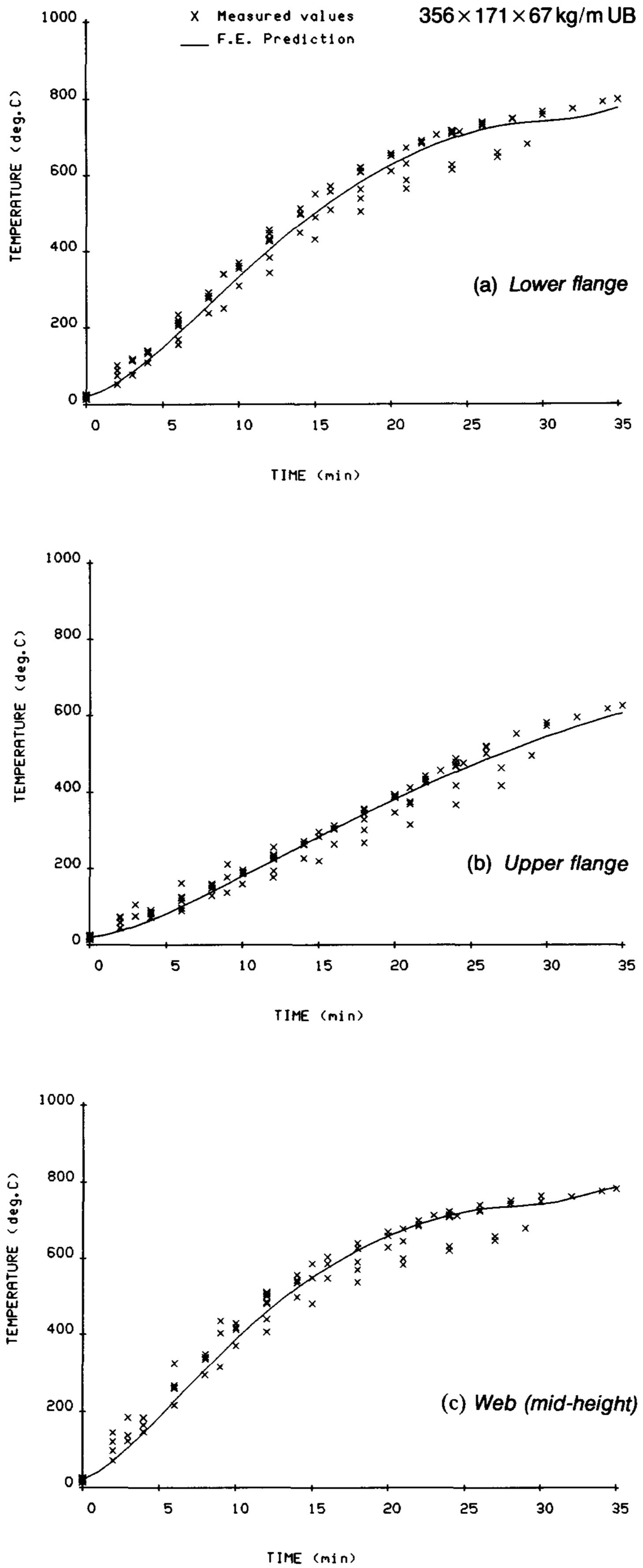


Figure 4.5 *Computed and measured temperatures in lower flange, upper flange and web of a typical beam supporting a concrete floor*

(Courtesy, British Steel (Swinden Laboratories))

BS 5950 Part 8

Table 4.2 *Design temperatures for I section columns*

Table 6

Flange thickness (mm)	Temperatures 30 mins	60 mins
≤6.8	841	945
9.4	801	911
11.0	771	900
12.5	747	891
14.2	724	882
15.4	709	877
17.3	689	869
18.7	676	864
20.5	661	858
21.7	652	854
23.8	637	848
25.0	630	844
27.0	618	839
30.2	601	832
31.4	595	829
36.5	574	820
37.7	569	818
42.9	552	810

Table 4.3 *Design temperatures for I section beams supporting concrete floors*

Table 7

Flange thickness (mm)	Temperatures 30 mins	60 mins
≤6.8	810	940
8.6	790	939
9.7	776	937
10.9	767	937
11.8	755	936
12.7	750	935
13.2	746	936
14.8	741	936
15.7	740	934
17.0	739	935
17.7	736	933
18.8	728	931
19.7	722	928
20.2	719	929
22.1	716	928
23.6	694	920
25.4	688	919
26.8	676	914
27.9	665	908
32.0	619	878
36.6	586	849

This data has been determined partly from computer analyses and partly from correlation with fire tests. The computer analyses have been carried out using FIRES–T (see Section 4.7) and have covered a wide range of representative beam sizes. These analyses were checked against temperatures measured in tests and were shown to be accurate. Differences between steel temperatures in column and beam tests arise partly because of the different convection and radiation characteristics of the two types of furnace.

Design temperatures for I section beams supporting concrete slabs are shown in Figure 4.6 as a function of section factor (as an alternative to Table 4.3). The phase change of steel at a temperature of around 730°C causes an absorbtion of energy and decrease in the rate of increase of temperature of the section. This leads to a 'plateau' in the relationship between temperature and section factor as is apparent from Figure 4.6.

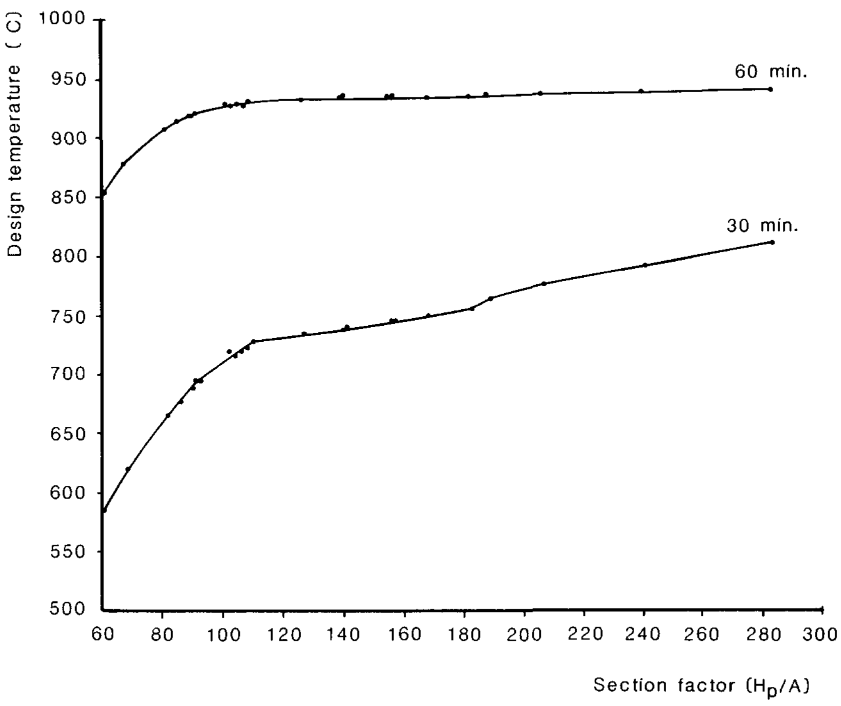


Figure 4.6 *Design temperatures for beams at 30 and 60 min (expressed in terms of section factor)*

An additional beneficial effect on the behaviour of beams in fire is that the web and inside faces of the flanges are partially shielded from the direct effects of the fire, which reduces the rate of heating. This is defined in terms of an aspect ratio (member depth/member width). This has already been taken into account in the design temperatures for standard UB sections, but contributes to a slight (up to 10°C) temperature reduction for UC sections. However, this effect is of greater benefit to partially-protected members such as shelf angle floors. 4.4.3.2 Table 8

4.4 Theoretical behaviour of protected steel sections in fire

The one-dimensional passage of heat through a thin fire protective material into a steel section is defined by the following equation in reference (9):

$$\delta\theta_s = \frac{1}{C_s \rho_s} \frac{H_p}{A} \frac{k_i}{d_i} (\theta_f - \theta_s)\, \delta t \qquad (11)$$

These parameters are defined in Section 4.2, except as follows:

k_i is the thermal conductivity of the protection material (W/m°C) (see definitions in section 1.4).

d_i is the thickness of the protection material (m).

Time dependent properties for k_i and C_s can be introduced in an incremental integration for θ_s knowing the variation of the fire or furnace temperature θ_f with time, t. The temperature rise of the steel section is of the same basic form but at a lower rate compared to an unprotected section (see Figure 4.4).

Equation (11) ignores certain beneficial factors. Firstly, there is a certain component of heat transfer by convection and radiation between the fire and the outer surface of the protection. Secondly, thicker heavier insulation materials have some thermal capacity (they store heat). Thirdly, most protective materials have some natural moisture content and a certain amount of heat is required to vapourize this moisture. This causes, a 'dwell' in the rise of temperature at approximately 100°C.

The generalized equation for the temperature rise in a protected steel section is:

$$\delta\theta_s = \frac{\dfrac{H_p}{A}\dfrac{(\theta_f - \theta_s)}{(1+\xi)}\delta t}{C_s\rho_s\left(\dfrac{1}{\alpha_c+\alpha_r}+\dfrac{d_i}{k_i}\right)} - \frac{\delta\theta_f\xi}{(1+\xi)} \qquad (12)$$

where $\xi = \dfrac{C_i d_i \rho_i H_p}{2C_s\rho_s A}$

C_i = specific heat and
ρ_i = density of the fire protective material

This one dimensional behaviour is illustrated diagrammatically in Figure 4.7.

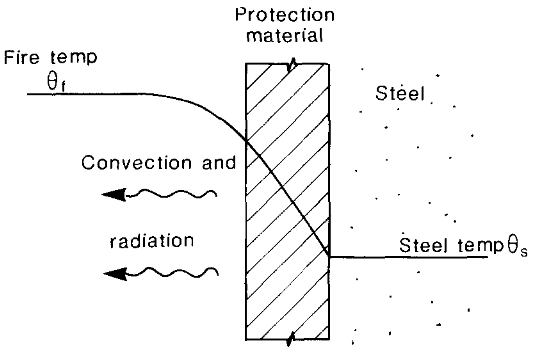


Figure 4.7 *Temperature distribution through a section of a protected member*

The term ξ is determined on the assumption that the average temperature in the fire protective material is $(\theta_f+\theta_s)/2$, and defines the relative amount of heat stored in the protective material. Moisture content effects are considered in Section 8.3.2.

It is normally found that the term $(\alpha_c+\alpha_r)^{-1}$ representing the surface radiation and convection effects is small in comparison with the term representing the insulating capacity of the fire protective material, and can be neglected. Similarly, the term in $\delta\theta_f$ is also small.

Equation (12), incorporating the above simplifications, is used in developing the design formula for the required thickness of fire protective material, as explained in Section 8.3.

4.5 Traditional and modern fire protective materials

Examination of the Equation (12) reveals that the most important parameter defining the performance of an appropriate fire protective material is its thermal conductivity k_i. There are a number of readily available materials of relatively low thermal conductivity and hence high insulating value. As with most good insulating materials it is the presence of voids distributed within the structure of the material that largely contribute to the insulating performance.

Examples of traditional fire protective materials are concrete (both normal and lightweight), brick- and blockwork and plaster board. These materials are not necessarily the best insulants but combine robustness and practicality, and have stood the test of good performance in service.

Examples of modern fire protective materials are those based on mineral fibre vermiculite, and lightweight cementitious materials, in spray or board form. Expanded vermiculite, in particular, is one of the best insulants but can be soft and easily broken away from the steelwork. Mixtures of cementitious materials and vermiculite are more robust.

Intumescent coatings are a separate category of materials which although originally applied thinly (as little as 1 to 2 mm thick), expand on heating in a fire. The foam-like expansion products offer the appropriate insulation to the steel. Some spray applied intumescent coatings are thicker and can provide over 1 hour fire resistance.

There are many proprietary materials of the above types that are marketed, each with different properties and methods of installation. Most of these are presented in reference (2). The performance of traditional and proprietary fire protective materials is described in Section 8.

4.6 Partial protection to bare steel beams and columns

Partial protection to steel members is often provided by the other elements to which the steel is attached. Examples are concrete floor slabs and masonry walls. In simple terms, their effect is to insulate a certain portion of the steel member from the direct effects of the fire.

Consider a steel column that is embedded in a cavity wall so that half of its perimeter is exposed to the fire; this member may be conservatively treated as having a section factor (heated perimeter/cross-sectional area) of half its original value. This assumes that the heat gained by the member is spread uniformly throughout the section. However, the temperature distribution throughout a partially protected member is far from uniform. The steel temperature in the insulated portion may be 300°C to 500°C lower than in the exposed portion, despite the relatively high conductivity of the steel.

The cooler portions of the cross-section have higher strength retention, and help to 'support' the hotter weaker portions. These effects contribute to a significant enhancement in the fire resistance of steel columns located within the inner leaf of masonry walls. Tests suggest that 30 minutes fire resistance can be achieved for such partially protected steel members that alone would have a fire resistance of barely 15 minutes.

Another practical example of partial protection is that of columns with 'blocked-in' webs using concrete blocks bonded between the flanges of the section[18]. The effective reduction in section factor is by some 70% relative to that of the exposed member. This is described further in Section 5.2.

A possible adverse effect of partial protection to columns is that of 'thermal bowing' or temperature induced deflection which occurs because of the temperature variation across the section. This happens before significant structural weakening of the section. Indeed close to failure, the movement of columns is often in the opposite sense to that of initial thermal bowing.

The potential amount of thermal bowing can be calculated from first principles. The deflection of a cantilever column (with no top restraint) is theoretically 13 times greater than the same column with top restraint. In practice, most columns are restrained by the supporting structure, leading to reduced deflections. The evidence of distress of masonry walls resulting from this effect is limited. Nevertheless it may be necessary to consider the effect of thermal bowing in the design of tall walls incorporating steel columns designed as cantilevers (see Section 12.4).

4.7 Computer methods for predicting thermal response

The analysis method presented in Sections 4.2 and 4.3 dealt with one-dimensional heat flow through uniform materials. There are many protected sections where heat-flow occurs in two dimensions because of the non-uniform thickness of protection, or because of the shape of the section. An example of this is concrete encasement to an I section.

Various computer programs have been developed to analyse generalized heat flow through sections comprising different materials. Examples of these are FIRES–T2, prepared by the University of California[19], CEFICOSS, prepared by the University of Liege, Belgium, (under European Commission funding)[20] TASEF, prepared by the University of Lund, Sweden, and PC based programs such as TEMPCALC.

In principle, the cross-section is modelled as a series of rectangular or trapezoidal elements and the basic heat-flow equations are solved between the elements for each time interval. The surfaces exposed to the fire are subject to the precise time-temperature relationship. Other surface emission properties take into account the local heat transfer by radiation. This method can incorporate time dependent material properties. Some more complex methods can take account of the moisture content of the materials.

The comparison of the temperature prediction by FIRES–T2 with test results for an unprotected member is shown in Figure 4.3. This comparison is generally good, given the scatter of the temperatures at different locations on the test beams which may arise from variations in the furnace heating. FIRES–T2 has also been used to compute some of the design temperature data in Tables 4.2 and 4.3.

5. EVALUATION OF FIRE RESISTANCE

This Section deals with the methods of evaluating fire resistance, either based on test data or calculations. Ideally, the results of fire tests should give the optimum fire resistance. Calculations should be conservative as they are generally not able to include some of the factors contributing to improved performance in a fire test. To this extent reliance is placed on test data for much of the design information on unprotected members. The results are extended to other design cases by the use of appropriate analytical methods based on physical behaviour in fire.

5.1 Performance derived from fire tests

Fire tests are an appropriate means of determining fire resistance provided the structural configuration and the test loads are representative of the structural use of the member under normal design loading divided by a representative load factor. For general application of the test information, the test loads should be consistent with the member being fully stressed under normal loading. The load factor is determined from the information in Section 2.2, and typical values (depending on the proportion of dead and permanent and non-permanent imposed loads) are between 1.5 and 1.8[1]. The inverse of the load factor is then the load-ratio (see Section 6.2). 4.3

Tests for appraisal of fire protective materials are less sensitive to the test load actually used. In principle, tests should be carried out at the highest load that may be encountered under fire conditions. However, in practice these tests are carried out at loads corresponding to the beam capacity divided by a load factor of 1.7[2]. The use of this data is not precluded provided that account is taken of the variation of load factor to cover other design cases. This is treated in the load ratio method of Section 5.3.1. 4.3.1

All fire tests should be carried out by an approved testing station and the results appraised and the recommendations made by a suitably qualified person or organization.

5.2 Performance of unprotected members derived from fire tests

5.2.1 Unprotected beams and columns

This section of the Code refers to empirical data on the fire behaviour of unprotected beams and columns. Comprehensive test data is presented in a British Steel publication *Compendium of UK Standard Fire Test Data on Unprotected Structural Steel*[12].

A series of beam tests have been carried out principally on 254 × 146 × 43 kg/m UB sections spanning 4.5 m as in Figure 3.5. Loads were applied through a 130 mm deep concrete slab. The test variable was the bending stress in the member, which has been used in deriving the load-ratio method outlined in Section 5.3.1. The effect of load ratio on the fire resistance of this beam section is shown in Figure 5.1. Further beam tests have been carried out on deeper UB sections, the heaviest with a minimum section factor (H_p/A) of 140 m^{-1}.

The scatter of results in terms of fire resistance is often considerable, as indicated in Figure 5.1. This is because the failure criterion is in terms of deflection, and hence strain, which can vary considerably for a small change in

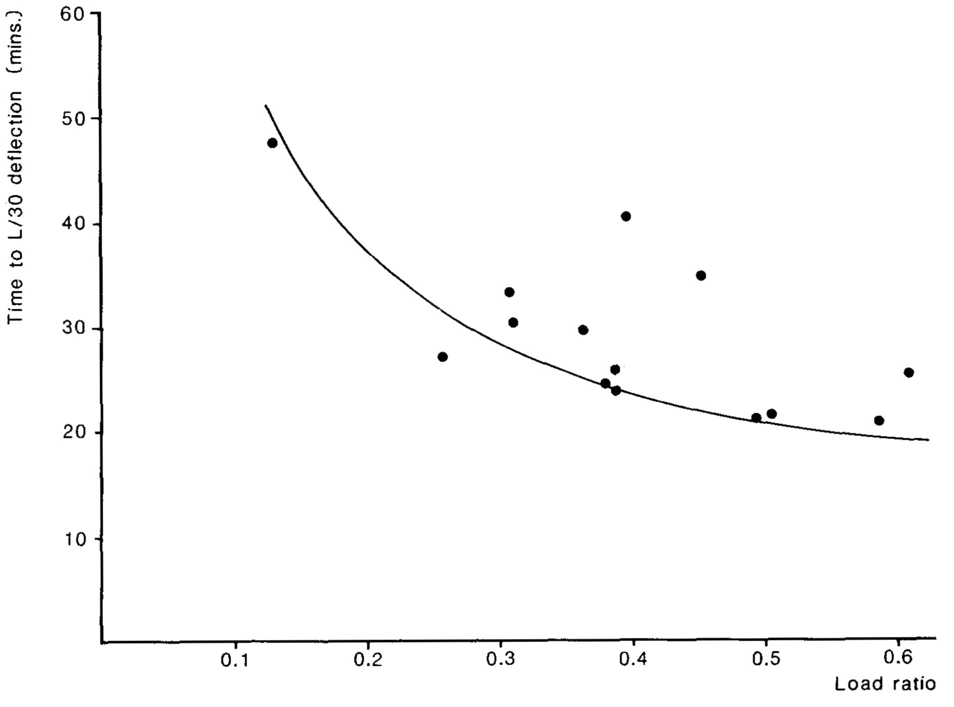


Figure 5.1 *Effect of load ratio on the fire resistance of a simply supported 254 × 146 × 43 kg/m bare beam*

strength (see Section 6). However, the limiting temperatures corresponding to these tests is less variable and this is the basis of the approach in *BS 5950: Part 8.*

A typical load-deflection curve for an unprotected beam in a fire test is shown in Figure 5.2. Although a deflection limit of span/30 was formerly imposed in

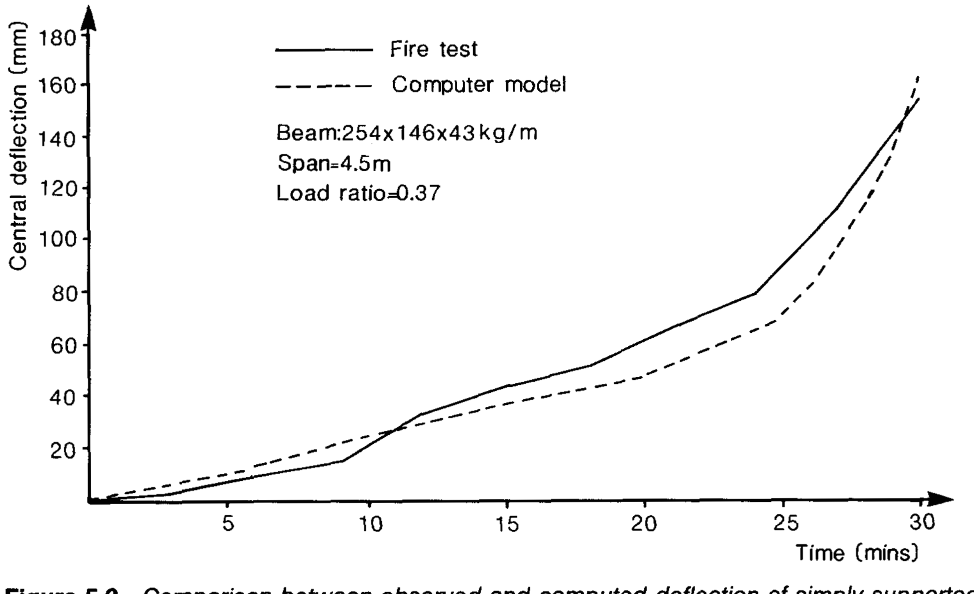


Figure 5.2 *Comparison between observed and computed deflection of simply supported beam in fire test*

BS 476: Part 8, the rate of deflection of unprotected beams increases rapidly close to failure. The increased fire resistance of an unprotected beam at a deflection limit of span/20 would be only 2 to 5 minutes depending on the section size. However, most beam tests would exceed the rate of deflection limit prior to reaching this deflection.

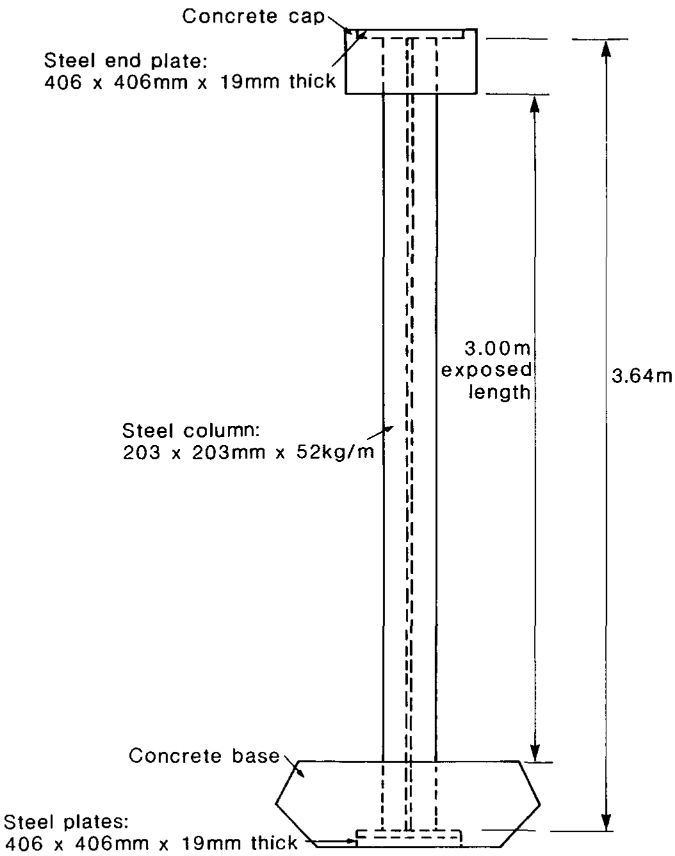


Figure 5.3 *Vertical section of an unprotected column test assembly (UK)*

A limited series of fire tests has been carried out on bare steel columns, standardized to a 3 m length (Figure 5.3). The column ends are treated as being fixed for consideration of lateral stability. The column is subject to nominally concentric axial load and end moments are negligible. A typical load-deflection relationship is shown in Figure 5.4. Failure occurs when the

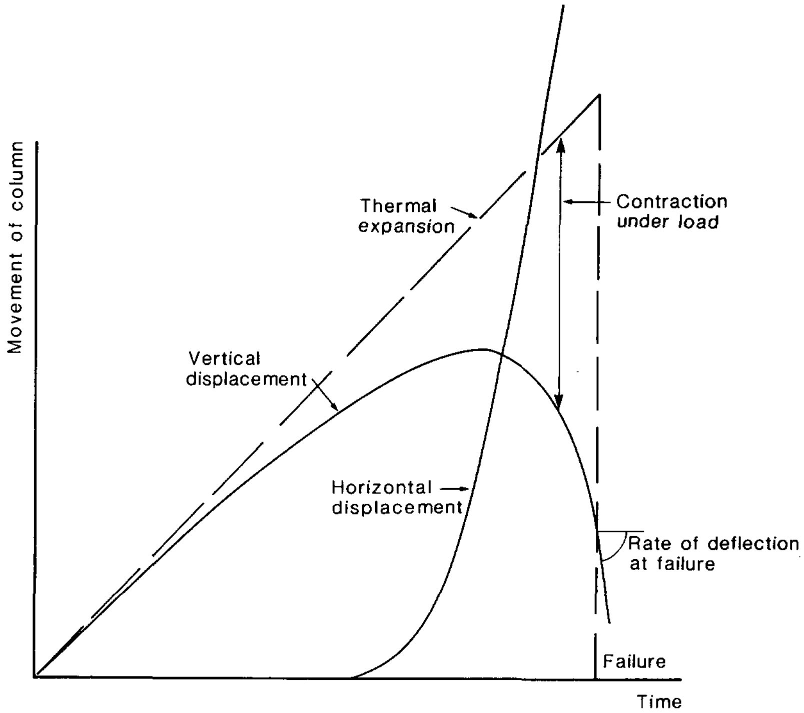


Figure 5.4 *Typical behaviour of steel column under axial load in fire test*

rate of vertical or horizontal deflection is so great that the load cannot be sustained.

Fire resistances of 30 minutes can be achieved for beams and columns of certain proportions. The following maximum section factors have been determined by extrapolation from the above tests: 4.3.2 Table 4

(a) Members in bending supporting concrete or composite floors
$Hp/A \leq 90\ m^{-1}$

(b) Columns in simple construction (as defined in *BS 5950: Part 1)*
$Hp/A \leq 50\ m^{-1}$

This data applies to members loaded in fire conditions to the *BS 449* permissible stress limits(7). This corresponds to a load ratio of approximately 0.6, taking account of the actual material strengths in the tests. In practical terms, the steel members satisfying the above limits are very heavy, and it would not normally be feasible to take advantage of this simplified approach. However, by reducing the loading to which the member is subject, more sections can achieve 30 minutes fire resistance.

A series of 10 fire tests has been carried out on nominally simply-supported beams with different degrees of end restraint(12). This would be the case where the beam size was determined on the basis of simple bending, but the beam-column connections were capable of developing rotational restraint in fire conditions. The restraint moment offered in the tests varied between 10% and 100% of the simply-supported moment.

Although no guidance has been put forward in the Code, the tests indicated that members with section factor of 170 m^{-1} could achieve a fire resistance of 30 minutes, provided the end connections were capable of resisting the applied moment in fire conditions. This would be typical of the behaviour of end plate or similar connections. The fire resistance of beam to column connections is the subject of continuing research (see Section 12.2).

5.2.2 Partially protected members

As noted in Section 4.6, partial protection to otherwise unprotected steel members can greatly enhance the fire resistance of the member. Typical structural forms which deliberately take advantage of this are shelf angle and slim floors. The design of shelf angle floors is covered in Section 10.

Partial protection to columns can be achieved by building the column into a cavity wall so that only one flange and a proportion of the web is exposed(12). Two fire tests have been performed on unprotected columns shielded in this way. A column with section factor of 77 m^{-1} (based on the exposed portion of the section) achieved a fire resistance of 30 minutes when subject to a load ratio of 0.6. Reducing the column load by 50% increased the fire resistance to over 90 minutes.

Two further tests have been performed on protected columns, with only one flange exposed (i.e. the web was protected). A column of this type achieved a fire resistance of 60 minutes when subject to a load ratio of 0.6.

A simple practical method of enhancing the fire resistance of a column is to fill in the gaps between the flanges with lightweight concrete blocks (see Figure 5.5). Four fire tests have shown that 30 minutes fire resistance can be achieved for columns designed to be fully stressed under normal conditions. This is now embodied in BRE Digest 317(18). The maximum section factor of the column (based on the exposed portion of the section) is 69 m^{-1}.

Examples of the cases of partial protection of beams and columns that can readily achieve 30 minutes fire resistance are shown in Figure 5.6. 4.3.2 Table 4

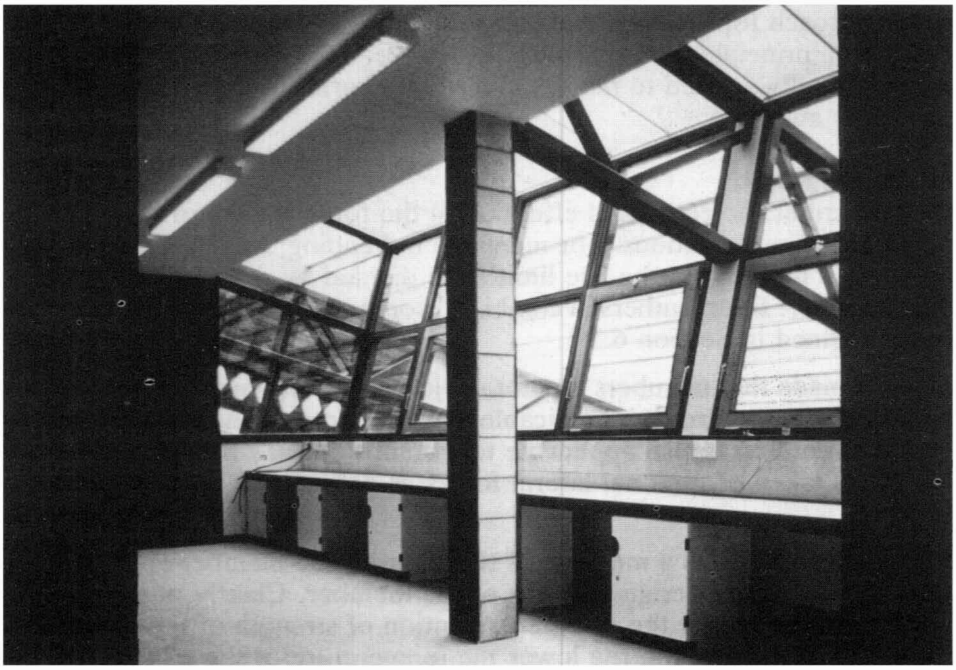


Figure 5.5 *Typical blocked-in column used in school building*

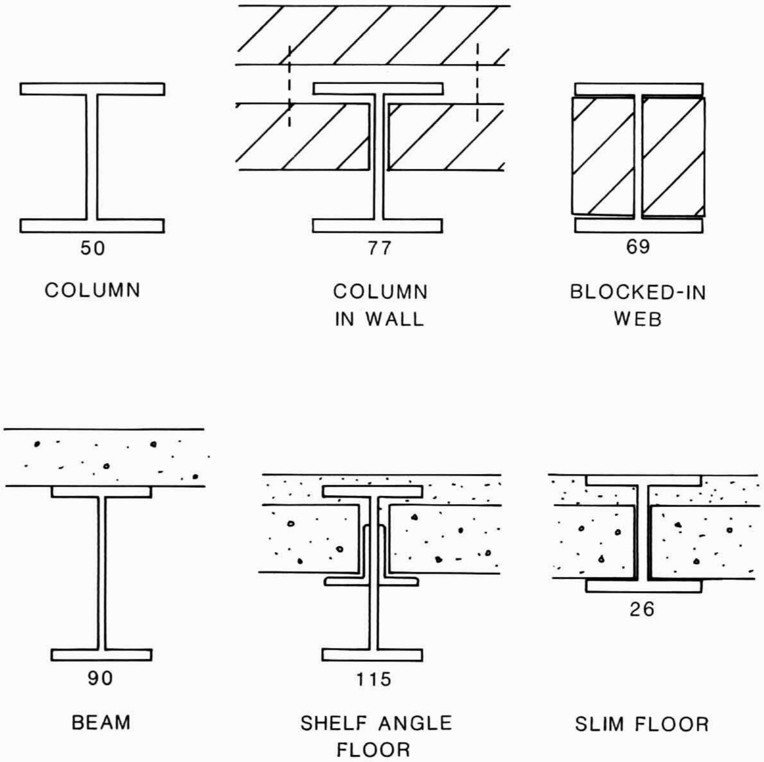


Figure 5.6 *Maximum ratios of Hp/A (m^{-1}) of exposed steel to give 30 minutes fire resistance*

5.3 Calculation methods for evaluating fire resistance

The principal method of determining the fire resistance of members subject to bending, axial compression and tension is the 'load ratio' method. This is independent of the form of fire protection as the method simply relates the temperature in a critical part of the member to its reduced strength under fire conditions. The temperature itself is determined from the design temperatures in Section 4.3 or on the basis of a thermal model or from fire test data.

An alternative approach for members subject to bending is the 'moment capacity' method. In principle, this is a more refined version of the load ratio method, and is generally applied to more complex structural forms for which temperature data is available.

5.3.1 Load ratio method

The load ratio describes the combined effect of all the force actions on the structural member in fire conditions. For members in bending, the load ratio is simply the applied moment at the fire limit state divided by the moment capacity of the member. For members in combined compression and bending, the load ratio is defined in Section 6.3.1.

The assumption is made that members are not subject to second-order effects resulting from deflections in fire. It is applicable to braced frames and columns in simple construction, and is also applicable to columns in sway frames provided account is taken of notional lateral forces as defined in *BS 5950: Part 1.*

The load ratio, therefore, gives a measure of the stress in the member at the fire limit state relative to the design strength of the member. Clearly, the higher the load ratio the higher the required retention of strength of the member in fire, and consequently, the lower the temperature of the critical element to resist the applied loads. This is known as the *limiting temperature* which is a function of the load ratio.

The limiting temperature method for beams and columns is presented in Section 6.

5.3.2 Moment capacity method

The load ratio method is applicable to normal structural members in bending, where the critical element determining the fire resistance is the lower flange. There are other structural forms where a different approach is required. It is relatively inexpensive to obtain thermal profile data by carrying out a small-scale fire test on a section of a more complex structure. Using the temperature data obtained, it is possible to evaluate the fire resistance of the member by considering the strength properties of the materials in the section at these elevated temperatures.

This is known as the 'moment capacity' method and may be applied to composite beams, shelf angle and slim floors. Indeed, it is the basis of the method used in determining the fire resistance of composite floors. The method is described in Section 7.

5.4 Fire resistance provided by generic forms of fire protection

The effect of fire protective materials on enhancing the fire resistance of steel members is covered in Section 8. Modern fire protective materials go through a vigorous appraisal procedure(2). This procedure is not covered in the Code. There are also many 'generic' forms of fire protection such as brick and blockwork and lath and plaster which have been used for many years and have performed satisfactorily in service.

The use of such generic materials are not presented in the Code, but reference is made to the BRE publication, *Guidelines for the construction of fire resisting structural elements*(21). The data contained in this publication is effectively 'deemed to satisfy' the requirements of this Code.

The data for minimum thickness of protection is reproduced in Tables 5.1 and 5.2. These are applicable to the section sizes given or to heavier sections. Solid protection means that the member is completely encased without intervening cavities. Hollow protection means that there is a void between the protective material and the steel. All hollow protection to columns should be effectively sealed at their ends.

Guidance on the design of hollow steel sections and castellated sections is given in Section 11.1 and 11.2 respectively.

Table 5.1 *Encased Steel Beams. 406 mm × 178 mm × 60 kg/m (Hp/A = 130) (Loaded in accordance with BS 449: Part 2(7))*

Nature of construction and materials (Protection applied on three sides)	Minimum thickness (mm) of protection for a fire resistance of: ½h	1h	1½h	2h	3h	4h
(A) **Hollow protection** (without an air cavity beneath the lower flange)						
A1 Expanded metal lath* with:						
(a) cement/lime/sand render (1:2:6 by volume) – thickness of render	13	25				
(b) lightweight aggregate gypsum plaster (metal lathing grade) – thickness of plaster	13	13	15	19	32†	50†
A2 Plasterboard with 1.2 mm steel wire binding at 100 mm pitch (if spiral) or 100 mm centres (if bands)						
(a) 9.5 mm plasterboard (minimum) finished with lightweight aggregate gypsum plaster – thickness of plaster	10	13				
(b) 19 mm plasterboard finished with						
(i) gypsum board finish plaster – thickness of plaster	5	5				
(ii) lightweight aggregate gypsum plaster – thickness of plaster			15	19		
(B) **Hollow protection** (with an air cavity beneath the lower flange)						
B1 Plasterboard on 44 mm × 44 mm (nominal) timber cradles‡, finished with lightweight aggregate gypsum plaster						
(a) 9.5 mm plasterboard (minimum – thickness of plaster	10	13				
(b) 19 mm plasterboard – thickness of plaster	10	10	15	19		
(C) **Solid protection**						
C1 Dense concrete, not leaner than 1:2:4 mix (unplastered)						
(a) concrete not assumed to be loadbearing, reinforcedø	25	25	25	25	50	75
(b) concrete assumed to be loadbearing, reinforced in accordance with BS 5950: Part I(1)	50	50	50	50	75	75
C2 Lightweight concrete, not leaner than 1:2:4 mix (unplastered) concrete not assumed to be loadbearing, reinforcedø	25	25	25	25	40	60

Table and guidance notes as in reference (21).

*Provided in accordance with guidance note 3.
†To be reinforced in accordance with guidance note 4.
‡Spaced and supported in accordance with guidance note 5.
øReinforced in accordance with guidance note 6.

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Table 5.2 *Encased Steel Columns 203 mm × 203 mm × 52 kg/m (Hp/A = 125) (Loaded in accordance with BS 449: Part 2[(7)])*

Nature of construction and materials (Protection applied on four sides)	Minimum thickness (mm) of protection for a fire resistance of:					
	½h	1h	1½h	2h	3h	4h
(A) **Hollow protection** (without an air cavity over the flanges)						
A1 Expanded metal lath* with						
(a) cement/lime/sand render (1:2:6 by volume) – thickness of render	13·	25				
(b) lightweight aggregate gypsum plaster (metal lathing grade) – thickness of plaster	13	13	15	19	32‡	50‡
A2 Plasterboard with 1.2 mm steel wire binding at 100 mm pitch (if spiral) or 100 mm centres (if bands)						
(a) 9.5 mm plasterboard (minimum) finished with lightweight aggregate gypsum plaster – thickness of plaster	10	13				
(b) 19 mm plasterboard finished with						
(i) gypsum board finish plaster – thickness of plaster	5	5				
(ii) lightweight aggregate gypsum plaster – thickness of plaster			15	19		
A3 Solid bricks of clay composition or sand lime, reinforced in every horizontal joint, unplastered	50	50	50	50	75	100
A4 Solid blocks of foamed slag or pumice concrete, reinforced in every horizontal joint, unplastered	50	50	50	50	60	75
A5 Autoclaved aerated concrete blocks, density 475–1200 kg/m^3	60	60	60	60		
A6 Solid blocks of lightweght concrete	50	50	50	50	60	75
(B) **Hollow protection** (with an air cavity over the flanges)						
B1 Plasterboard on 44 mm × 44 mm (nominal) timber cradles‡, finished with lightweight aggretate gypsum plaster						
(a) 9.5 mm plasterboard (minimum) – thickness of plaster	10	13				
(b) 19 mm plasterboard – thickness of plaster	10	10	15	19		
(C) **Solid protection**						
C1 Dense concrete, not leaner than 1:2:4 mix (unplastered)						
(a) concrete not assumed to be loadbearing, reinforcedø	25	25	25	25	50	75
(b) concrete assumed to be loadbearing, reinforced in accordance with BS 5950: Part I[(1)]	50	50	50	50	75	75
C2 Lightweight concrete, not leaner than 1:2:4 mix (unplastered) – concrete not assumed to be loadbearing, reinforcedø	25	25	25	25	40	60

Table and guidance notes as in reference (21).

*Provided in accordance with guidance note 3.
†To be reinforced in accordance with guidance note 4.
‡Spaced in accordance with guidance note 5.
øReinforced in accordance with guidance note 6.

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6. LIMITING TEMPERATURE METHOD

6.1 General

The limiting temperature of a member provides a simple way of safely relating the failure temperature to the load that the member supports in fire conditions. 4.4.2

In Section 3, the way in which steel loses strength with increasing temperature was described. All steel members lose strength in a similar manner but because of aspects such as temperature gradients and instability effects at elevated temperatures it is not appropriate to apply the same strength reductions to structural members as are applied to the steel itself.

A steel beam supporting a concrete floor will be subject to greater exposure to heat on the bottom flange than on the top flange. This will result in a temperature variation across the section. The colder, upper parts of the section, will then be able to offer some 'support' to the hotter parts with the result that the overall loss of strength of the beam is less than the loss of strength of the hottest part. This effect is illustrated in Figure 6.1. Typically,

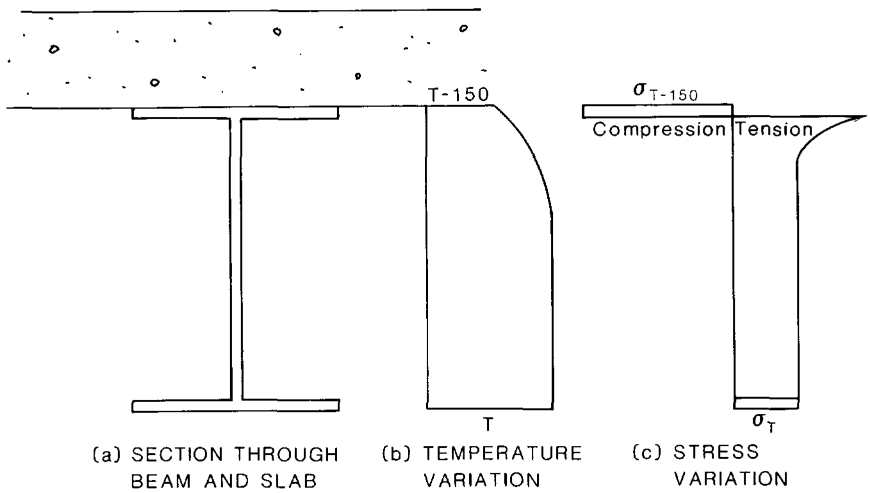


Figure 6.1 *Temperature and stress variation in I beam supporting concrete slab when limiting temperature is reached*

the temperature difference between the flanges will exceed 150°C when the lower flange reaches its limiting temperature. This is the case for both unprotected and protected sections.

In the limiting temperature method the failure of a member in a fire resistance test is characteristed by the temperature of one element or part of the member called the 'critical element'. This element is normally but not necessarily that part of the member which is hottest. For example the web of a beam may sometimes reach slightly higher temperatures than the bottom flange but it has been found that the bottom flange temperature is a better indicator of failure than the web and is therefore the critical element.

As a member heats up in a fire it supports load until the temperature of the critical element exceeds the limiting temperature. At this point, a member in compression becomes unstable, or a beam undergoes large deflections, or a member in tension extends excessively.

The ability to relate the applied load to the failure temperature in a fire allows the designer to relate the amount of fire protection required to the loading on a structure. In certain circumstances it may even be possible to show that no fire protection is necessary. However, fire resistance is expressed in terms of time. To achieve, for example, 30 minutes fire resistance one must know whether the temperature of the critical element would exceed the limiting temperature after 30 minutes exposure in a fire resistance test. For unprotected members the Code gives some guidance on how hot the critical element will get in any particular time and this is discussed in Section 4. For protected steel members it is not possible to give absolute guidance in a code as all fire protection materials are different and most are proprietary in nature. Some guidance is given in Section 8.

6.2 Critical elements

6.2.1 Members in bending

For members supporting concrete or composite floors the critical element is the bottom flange. In any other situation it is the hotter flange. For structural hollow sections a flange should be considered as a flat side or 25% of the circumference or a circular section.

6.2.2 Members in compression or tension

For members in compression or tension the critical element can be taken as the hottest flange.

6.3 Load ratios

The limiting temperature of a member in a given situation depends on the load that the member carries. Because of the range of member sizes and member lengths used in buildings it would be impractical in the Code to use actual imposed loads. It would be possible to produce a table for each available beam or column section giving the limiting temperature for a range of bending moments or axial capacities but it would obviously require a large number of different tables and would lead to a very bulky Code. However, a detailed analysis of fire test results and the use of computer models has demonstrated that in virtually every situation the limiting temperature is dependent on the fraction of the ultimate load capacity that a member supports at the time of the fire. We can therefore assume that a 254 × 146 × 43 Universal Beam loaded to 50% of its ultimate bending capacity will fail at the same temperature as a 457 × 191 × 67 Universal Beam loaded to 50% of its ultimate bending capacity.

In practice load ratios have been used for many years. There are over 100 beam and column sections available but only a relative few have ever been fire tested. We have relied on the fact that fully loaded members, designed in accordance with *BS 449*[7], fail in fire resistance tests at approximately the same temperature. In *BS 5950: Part 8* this has been extended to all levels of load so that fire protection can be more accurately and economically specified.

BS 5950: Part 8 gives limiting temperatures for different types of member for a range of load ratios. It is important to understand the limitations of the data in the Table and how to calculate the load ratios in each situation. This is reproduced in Table 6.1 of this publication. The background of this Table is discussed in Section 6.4. 4.4.2 Table 5

Table 6.1 *Limiting Temperatures for Design of Protected and Unprotected Members*

Case No.	Members in compression	Load Ratio 0.7	0.6	0.5	0.4	0.3	0.2
	Uniform heating on all faces; braced members in simple construction:						
(1)	Slenderness ratio ≤ 70	510	540	580	615	655	710
(2)	Slenderness ratio ≤ 180	460	510	545	590	635	635
	Members in bending						
	Supporting concrete or composite deck floors:						
(3)	Unprotected members, or protected members complying with clause 2.3 (a) or (b)	590	620	650	680	725	780
(4)	Other protected members	540	585	625	655	700	745
Not supporting concrete floors:							
(5)	Unprotected members, or protected members complying with Clause 2.3 (a) or (b)	520	555	585	620	660	745
(6)	Other protected members	460	510	545	590	635	690
	Members in tension						
(7)	All cases	460	510	545	590	635	690

6.3.1 Members in compression

Cases (1) and (2) of Table 6.1 present limiting temperatures for two ranges of strut slenderness. Case (1) is intended for use for columns in simple construction
where the loading is predominantly axial and moments are comparatively small. Members may be exposed to the fire on 1, 2, 3 or all faces. These are not intended for use with compressive members carrying large moments such as columns in rigid multi-storey frames or for use with columns in sway frames. This is discussed in Section 6.5.

For columns in simple multi-storey construction the load ratio, R, is given by the interaction formula of Clause 4.8.3.3.1 in *BS 5950: Part 1.* 4.4.2.3

$$R = \frac{F}{A_g p_c} + \frac{m\,M_x}{M_b} + \frac{m\,M_y}{p_y\,Z_y} \qquad (13)$$

where F = axial load during fire
M_x = maximum moment about x axis during fire
M_y = maximum moment about y axis during fire
A_g = gross cross-sectional area
p_c = compressive strength of member
m = 1.0 (Clause 4.7.7. – Part 1)
M_b = buckling resistance moment capacity about major axis
p_y = steel strength
Z_y = elastic section modulus about minor axis

This approach may also be used for other compressive members for which Part 1 allows the use of the simplified approach.

6.3.2 Members in bending

The limiting temperatures for members in bending are dependent on the maximum strain that the extreme fibres can safely sustain and whether they are supporting and laterally restrained by a composite or reinforced concrete floor.

The load ratio R for laterally restrained beams is defined as: 4.2.2.2

$$R = \frac{\text{Applied moment at the fire limit state}}{\text{Moment capacity at 20°C}}$$

For beams subject to lateral torsional buckling, designed in accordance with Clause 4.3.7 of Part 1, $R = \bar{M}/M_b$

where $\bar{M}$ = equivalent uniform moment at the limit state fire

M_b = lateral torsional buckling moment

This load ratio should be not be less than the load ratio calculated as for restrained beams.

For members in bending, Table 6.1 gives limiting temperatures for four categories of member. Cases (3) and (4) should be used for beams supporting concrete or composite floors in which case the top flange is partially insulated.

Case (3) represents probably the most common situation where a fire protected composite or non-composite beam is insulated with a reasonably deformable form of fire protection that has demonstrated its 'stickability' in a fire test. This means that the fire protection has remained intact at the large deformations corresponding to the deflection limits in a standard fire test. It should also be used for unprotected beams. Case (4) should be used if the fire protection has not demonstrated the required 'stickability' in a fire test. Cases (5) and (6) are for use with beams which do not support concrete or composite floors and which therefore do not benefit from the insulation of the floor. Again the 'stickability' of the fire protection is considered. This concept of 'stickability' and its assessment is discussed in Section 8.2.3.

6.3.3 Members in tension

Limiting temperatures for members in tension are given in case (7) of Table 6.1. The load ratio is calculated from the interaction formula of Clause 4.8.2 in Part 1. 4.4.2.4

$$R = \frac{F}{A_e p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \qquad (14)$$

where A_e = effective area in tension
M_{cx} = moment capacity about the major axis
M_{cy} = moment capacity about the minor axis

It is important to point out that close to failure a member in tension could be at least 1.2% longer than its initial cold state. This comprises 0.5% mechanical strain and approximately 0.7% thermal strain. Any protective material should be capable of accommodating such movement without distress.

6.4 Background to limiting temperatures

In recent years a large number of fire resistance tests have been carried out and many organisations throughout the world have studied the behaviour of steel members and structures in fire. There now exists a large volume of data derived largely from tests and some computer models which correlate reasonably well with the test results. It has therefore been possible to devise the limiting temperature/load ratio table in the Code. Where possible, test data has been used but this has in some places been supplemented by computer modelling.

6.4.1 Members in compression

For compressive members of low slenderness, the limiting temperatures in Case (1) are based on test results supplemented with information from the ECCS Recommendations[9]. It was found that the ECCS simplified column method provided an excellent fit to the limited test data. The load ratio, *R*, is determined from: Table 5

$$R = \text{Load multiplier} \times \text{steel strength reduction factor.}$$

The load multiplier was taken as 0.85 (as in the ECCS Recommendations) to take account of some end fixity of the columns, and also the fact that on average the steel would be stronger than the specified minimum. The strength reduction factor is based on 0.5% strain in the steel. This is used because of the relatively low strains that are experienced in columns just prior to failure. The behaviour of a typical column in a fire test is shown in Figure 5.3. Some of the test data on columns shown together with the limiting temperatures based on this approach are present in Figure 6.2. 2.3 (c)

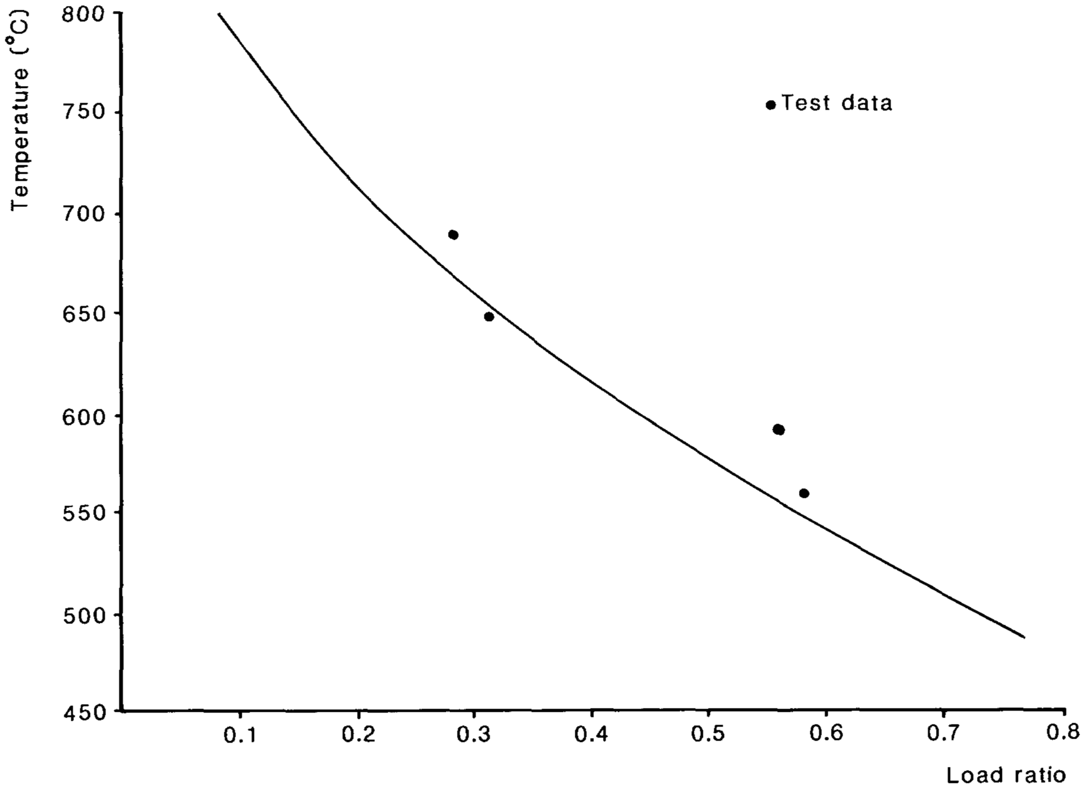


Figure 6.2 *Limiting temperatures of columns*

The test data on columns relates to slenderness less than 50. This is because of the constraints of the test furnace. Most practical columns in buildings have a slenderness less than 70, and the limiting temperatures have been extended up to this value.

For more slender compression members, commonly used as struts rather than columns, a mor conservative estimate of their strength is required. In these members, the effect of end restraint would be less pronounced. Also, high strains in the steel would affect the stability of the member by reducing the effective elastic modulus leading to increased lateral deflections under compression.

For Case (2), it was decided to adopt a strain limit of 0.5%, with the above load mulitiplier of 0.85 increased to 1.0. This is very similar to the use of a lower strain limit of 0.2% (as used in the ECCS recommendations) but retaining the factor of 0.85. The use of an upper limiting temperature corresponding to a load ratio of 0.3 avoids the potential risk of damage if normally low stressed members (e.g. bracing) are subject to additional loads in a fire. Table 5

For slender columns in buildings the same argument may be followed. Added to this is the effect that slender columns benefit more from continuity in a fire than stocky columns. However, restraint to thermal expansion may introduce secondary forces and eccentricities which cause a reduction in the column strength. There is little information on the behaviour of slender columns in fire.

6.4.2 Members in bending

The limiting temperatures for beams supporting concrete or composite floors, Case (3), are based on a large number of fire resistance tests carried out by British Steel and reported in reference (12). The test beams were loaded to produce various proportions of the maximum bending stress in accordance with *BS 449*. The loading conditions were re-analyzed and expressed in terms of the load factors in *BS 5950: Part 1*. The design strength of the steel was taken as the measured flange yield stress rather than the nominal design strength. The 'failure' condition was taken as a deflection limit of span/30. The results of this analysis and the Code design curve are shown in Figure 6.3. The design curve was taken as a lower bound of the test results. For temperatures less than 600°C the design curve was obtained by computer modelling. Table 5

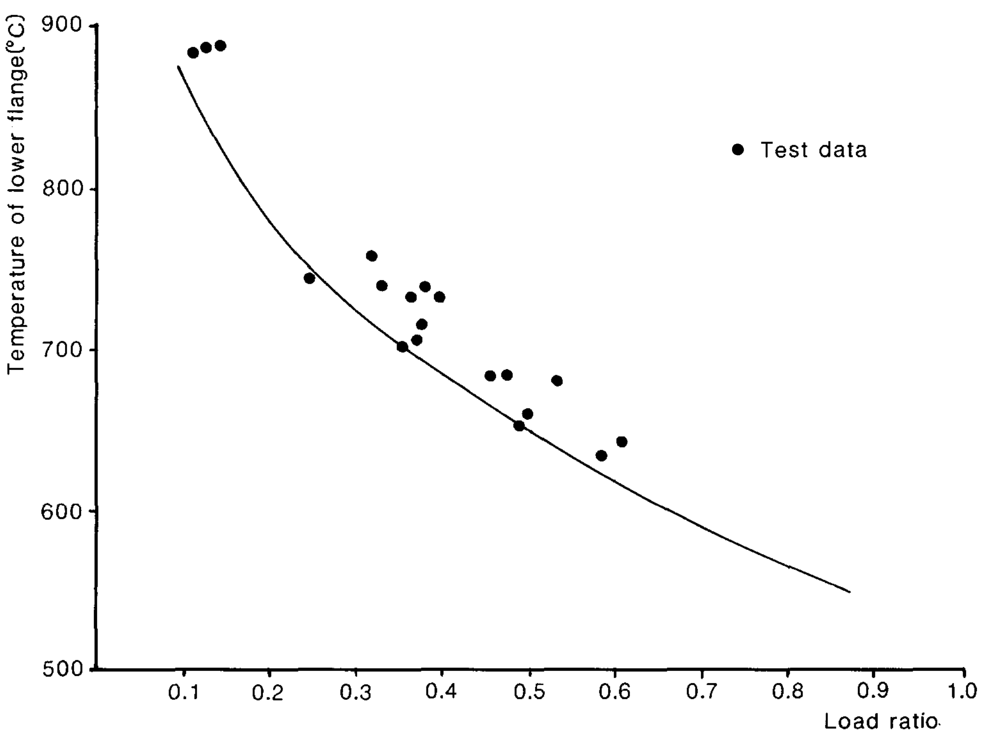


Figure 6.3 *Limiting temperatures for I section beams supporting concrete floors*

There are various cases that have been considered. Where the member supports a concrete floor (precast slab, in-situ slab, composite floor), the limiting temperature of the bottom flange of the beam increases. This is because the upper flanges retain more of the strength and the plastic netural axis of the section rises (see Figure 6.1). Consequently, more of the web resists tension which increases the effective section modulus and hence moment capacity in fire.

In fire tests, beams undergo large strains ,typically up to 6% in the lower flange as illustrated in Figure 6.4. However, this strain depends to some extent

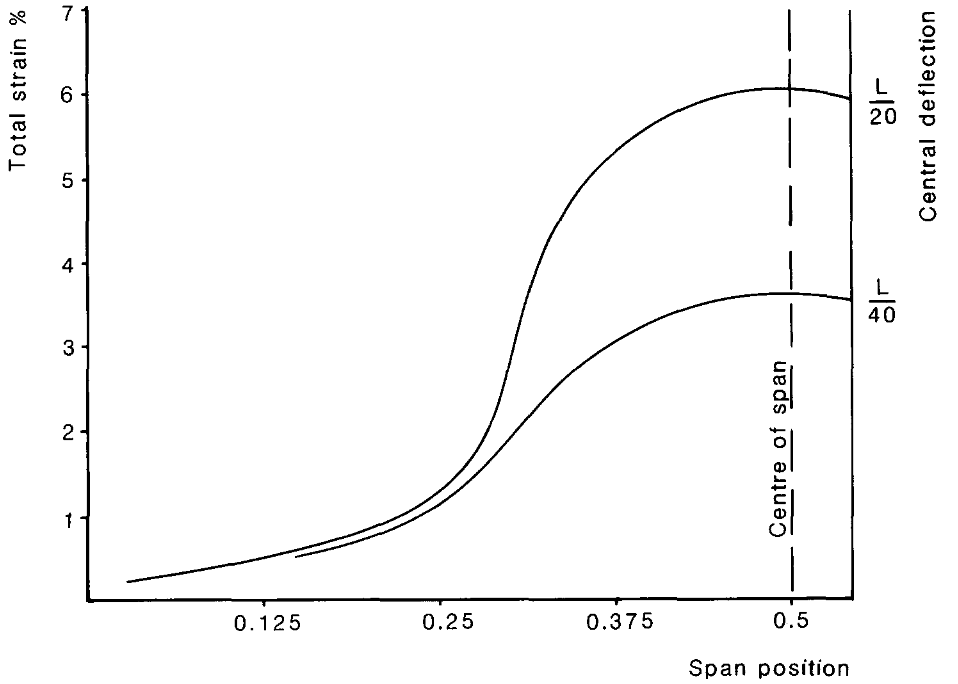


Figure 6.4 *Distribution of bottom flange strain along a simply-supported beam (uniform load, span/depth = 11)*

on the proportions of the member tested. For example, the lower the span:depth ratio of the beam the higher the strains for a given central deflection. A lower bound to the 'mechanical' proportion of this strain (excluding thermally induced strain) has been taken as 1.5%. This gives a conservative, but reasonably accurate measure of the strength of the beams. 2.3 (b)

Fire protective materials are assessed by the standard fire resistance test. It may be shown that a 'mechanical' strain of at least 1.5% is achieved in these tests provided the deflection of the beam exceeds span /40 at termination of the test (see section 8.2.3). This is important to ensure that the protective material maintains its stickability and does not become detatched or lose its protective properties in a fire.

Fire protective materials that have not been subject to the standard fire test, or have not demonstrated their stickability have been assessed using a strain limit of 0.5%. In order to 'correct' the test data for this lower strain the following approach was used:

(1) Assume the temperatures in Case (3) (Table 6.1) correspond to 1.5% strain

(2) From Table 3.1 find the strength reduction factor for each temperature in Case (3) (Table 6.1) corresponding to 1.5% strain

(3) From Table 3.1 find the temperature which gives the same strength reduction factor for 0.5% strain. This is now the limiting temperature in Table 6.1 (Case (4)).

For members in bending but not supporting concrete or composite floors the limiting temperatures in Cases (5) and (6) (Table 6.1) correspond to the strength reduction factors in Table 3.1 for 1.5% and 0.5% strain respectively. Unprotected members, or members which are fire protected with materials which have demonstrated their stickability have limiting temperatures based on 1.5% strain. Other members have limiting temperatures based on 0.5% strain. Table 5

6.4.3 Members in tension

For members in tension, *BS 476: Part 20*(5) gives no method of test so the limiting temperatures are based simply upon the measured tensile properties of the steel at elevated temperatures. It was felt that in order to limit excessive elongation a strain limit of 0.5% should be imposed. Close to 'failure' the total strain, including thermal strain could be at least 1.2%. The limiting temperatures are therefore based on the 0.5% strain strength reduction 2.3 (c)
factors from Table 3. 1.

6.5 Behaviour of columns in frames

The load ratio method of Section 6.3.1 may be used for columns in buildings where resistance to lateral loads is provided by bracing or shear walls. This also applies where the frame is designed to develop continuity so that moments are generated in the beam-column connections. In these so-called 'braced' frames the ends of the columns are restrained against lateral displacement.

Columns in unbraced frames (or sway frames) behave rather differently as they have been designed to resist bending moments induced by lateral forces. As the structure loses its stiffness in a fire, it becomes more unstable. A measure of the stability and robustness of the structure is the concept of 'notional lateral forces' in *BS 5950: Part 1*. In the absence of other guidance, it was considered reasonable to use this concept to assess the behaviour of sway frames in fire. Notional forces are specified as the greater of 1% of the factored dead load or 0.5% of the factored dead and imposed load per floor acting at each floor level. In fire conditions, it is appropriate to use reduced load factors when calculating these notional forces (see Section 2.2). Other second order effects are neglected.

The moments induced by the notional forces may be considered in
combination with the moments and axial forces due to vertical loading in fire 4.4.2.3 (b)
conditions, but not those due to wind. In principle, the load ratios for columns in sway frames may then be calculated from Equation (13) taking account of the moments and forces that are generated in unbraced frames. This leads to selection of limiting temperatures from Table 6.1. Clearly, in the single storey frames the moments arising from notional forces will be small and can usually be neglected.

In designing to resist notional forces it is possible to consider redistribution of these forces to columns not involved in the fire, assuming that the floor is capable of transferring in-plane loads. When calculating moments in the columns arising from these notional forces, points of contra-flexure may be assumed at mid-height of the columns.

An alternative, simple approach for columns in sway frames is to use a limiting temperature based on a column stressed to a load ratio of 0.67. This corresponds to a limiting temperature of 520°C for a column with a slenderness ratio less than 70. The justification for this is largely based on the good performance of buildings traditionally designed to a single limiting temperature of 550°C.

7. MOMENT CAPACITY APPROACH

7.1 General approach

The Code permits the calculation of the fire resistance of members in bending by the 'moment capacity method'. The moment capacity of the section at elevated temperature is compared to the moments on the member at the fire limit state. In using this method the temperature distribution through the section at the appropriate fire resistance periods must be known. Temperatures may be computed or determined from fire tests. 4.4.4

7.1.1 Method of calculation

Knowing the temperature profile through the critical section, the member is divided into a set of elements of approximately equal temperature. The number of elements will depend on the complexity of the member and its temperature gradient. The reduced strengths of the various elements of the cross-section can be calculated at elevated temperatures using the data in Tables 3.1 and 3.2 (Section 3.2). The 'plastic' neutral axis of the section is determined, such that in the absence of axial forces, the net tensile and compressive forces acting on the section are in equilibrium. The moment capacity of the section then follows by multiplying the reduced strength of each element by the distance from the neutral axis and summing all the elements in the section.

The Code restricts the use of the moment capacity method to sections which are 'compact' as defined in *BS 5950: Part 1*. This is because almost every fire test has been carried out on laterally restrained beams which were either compact or plastic. There is little or no data on semi-compact or slender sections. It is assumed that the member fails in a flexural manner, without occurrence of premature shear or instability effects in fire.

In fire, the position of the 'plastic' neutral axis changes. For beam elements the lower parts of the cross-section are exposed to the fire, whereas the upper parts are usually better insulated. As the lower parts weaken, so the plastic neutral axis shifts upwards. For simple beams supporting concrete floors the plastic neutral axis can rise close to the upper flange. This behaviour is illustrated in Figure 6.1.

7.1.2 Application of the method

The moment capacity determined by the above method will usually be significantly lower than that obtained by the limiting temperature method or from a fire test. This is illustrated for a simple beam in Section 7.2. The reasons for this conservatism are:

- steel strengths are higher than the minimum specified strength
- higher strains are developed than assumed in determining the material strengths
- temperatures are often lower than assumed
- the interaction between the elements may offer more restraint or composite action than assumed.

In principle, the moment capacity method may be used for non-composite or composite beams, or special structural forms, such as shelf-angle or slim floors for which temperature data is available. The method is useful for assessing the sensitivity of the fire resistance of the construction to changes in temperature distribution and the sizing of the elements.

7.2 Use of the moment capacity method for simple beams

The moment capacity method may be used to provide a direct comparison with the limiting temperature approach for simple beams supporting concrete floors as follows:

The temperature distribution in an I section beam may be considered to be uniform over the lower portion of the web and the fully exposed lower flange. The upper flange temperature of an unprotected beam is 150 to 200°C lower than that of the lower flange at its limiting temperature. This reduction in temperature extends partly into the web (see Figure 6.1).

The reduced moment capacity of the section relative to normal conditions can be alternatively expressed as:

Load ratio <

$$\frac{\text{section modulus in fire conditions}}{\text{section modulus in normal conditions}} \times \text{strength reduction factor} \qquad (15)$$

The strength reduction factor is that applied to the basic strength of the steel as represented by the temperature of the lower flange (see Tables 3.1 and 3.2) The section modulus in fire may be determined on the basis of an assumed temperature distribution. This is normally such that the residual strength of the insulated upper flange closely equates to the combined residual strengths of the lower flange and the web. The plastic neutral axis of the section therefore lies close to the top of the web.

The section modulus in fire may be then determined by considering the moment contribution of the stress blocks of the web and the lower flange. An allowance may be made for the reduced temperature and hence increased strength of the upper portion of the web. It is normally found that the ratio of the section modulus of an I section in fire to that in normal design increases to a value between 1.2 and 1.5 depending on the proportions of the cross-section. This increase arises because the contribution of the web to the plastic moment resistance of the section has at least doubled due to the change in the position of the plastic neutral axis.

Consider an 838 × 292 × 194 kg/m UB (with a section factor of 101 m^{-1}) supporting a concrete floor. The temperature distribution through the unprotected section after 30 minutes fire exposure is presented in Figure 7.1. This data was obtained from a finite element analysis using FIRES–T (Section 4.7). The 'design' temperature of the lower flange is 707°C at this period of fire exposure (slightly lower than given in Table 4.3).

The cross-section may be divided into representative blocks and the position of the plastic neutral axis calculated by equating tensile and compressive forces. The moment capacity of the section is then calculated by taking moments of each of the stress blocks around the plastic neutral axis.

It may be demonstrated in this example that the plastic neutral axis in fire conditions is only 20 mm below the soffit of the concrete floor. At a lower flange temperature of 707°C, the strength reduction factor for steel is 0.21 (for 1.5% strain). However, the moment capacity of the section in fire, taking account of the stress blocks in Figure 7.1, is 0.3 times its 'cold' moment capacity (i.e. load ratio of 0.3). This represents a 43% increase over the moment capacity of the section based on a uniform design temperature throughout the section.

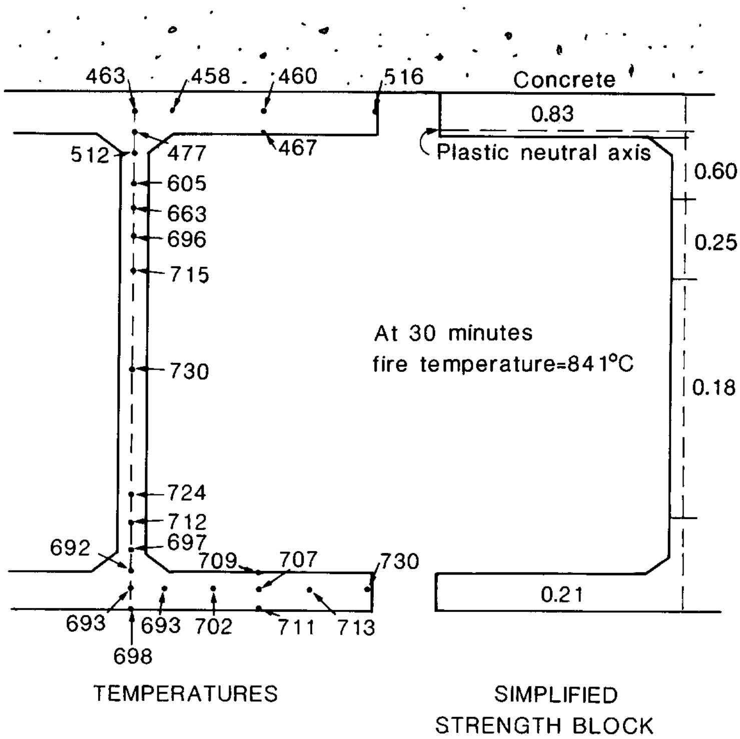


Figure 7.1 *Example of use of moment capacity method for 838 × 292 × 194 kg/m UB*

From Table 6.1, the load ratio corresponding to a limiting temperature of 707°C is 0.34. Allowance may be made for the fact that, on average, steel will be at least 10% stronger than the characteristic value used in normal design and that steel strains considerably higher than 1.5% will be experienced in tests. It is also known that methods such as FIRES–T give a conservative measure of temperatures in unprotected sections. Therefore, the moment capacity method results in an equivalent load ratio of 0.33 which agrees reasonably well with the tabulated value.

7.3 Application to particular structural forms

In principle, the moment capacity method may be applied to any structural form, but it has proved to be most appropriate for determining the capacity of partially protected beams and floors in fire. The two best examples of this are shelf-angle beams and composite deck slabs.

The design of composite deck slabs can be carried out by what is known as the 'fire engineering' method which is in essence the same as the moment capacity method. Thermal profile data exists for different concrete types and slab forms, from which the reduced strength of the concrete and steel can be calculated. The moment capacities of the section in hogging and sagging bending can then be evaluated. For continuous floors, their capacities can be combined by considering the plastic failure mechanism of the floor. This approach is covered in Section 9.

The design of shelf-angle beams is covered in Appendix C of the Code. In this form of construction, improved fire resistance can be achieved because the upper portion of the web and the top flange and the upper leg of the support angle are well insulated from the fire. Temperature data at specific time intervals is available from fire tests. The moment capacity method can therefore be used to determine the residual strength of the member taking account of the temperatures and the strengths of the various elements. This is covered in Section 10.

8. EFFECT OF FIRE PROTECTIVE MATERIALS

8.1 Performance of traditional materials

Fire protective materials for structural steelwork have a number of important characteristics. They should:

- be relatively cheap (in comparison to cost of the steelwork) and easy to apply to a range of steel sections
- be safe (e.g. not be toxic or hazardous during application, in-service, or during a fire)
- insulate the steelwork against the effects of a fire
- remain intact (without becoming detached or forming wide fractures) so that the steel is protected despite being badly deformed.

Traditional fire protective materials are concrete, brickwork, and gypsum or plaster board. The 1975 Building Regulations(22) contained 'deemed to satisfy' requirements for the use of these 'traditional' materials but these were deleted from the 1986 update of the England and Wales Regulations(3). The BRE publication by Morris, Read and Cooke(21), reviews the use of these materials and is referred to in the Code. Salient data is presented in Tables 5.1 and 5.2 of Section 5.4 for guidance.

It should not be assumed that fire tests have been carried out to justify all the information in these Tables. However, performance in service has suggested that these methods of fire protection are adequate. Indeed, some of these requirements may be felt to be onerous, particularly where there are practical difficulties in the installation of these traditional methods of protection.

8.1.1 Concrete encasement

Consider firstly concrete encasement as the most common traditional method of fire protecting steelwork: The thickness of the concrete cover to the flanges of I section beams or columns determines the protection offered. Normal weight concrete is a poor insulant but the 'heat-sink' effect of the solid mass of concrete means that the temperature rise in the core of the section is relatively slow. The problem with concrete as a fire protective material is that it tends to break-off or spall in fire, mainly as a result of trapped moisture in the concrete. Supplementary mesh reinforcement is generally provided around the section to help retain the concrete cover in a fire.

In concrete encased beams, it is the temperature of the lower flange of the beam that largely determines the strength of the section. In concrete encased columns, the concrete both reduces the slenderness of the column, and keeps the embedded web of the section at a relatively low temperature. The benefits of encasement of columns are therefore greater than for beams. No account is usually taken of the compressive strength of the concrete unless measures are taken to ensure effective continuity of the concrete at column-column junctions (at beam positions).

There are also practical difficulties in connecting columns and beams once the floors are in place. Precasting of the members is one solution, as illustrated in Figure 8.1. The concrete around the connections is placed in situ.

The design of hollow concrete filled sections is covered in Section 11.

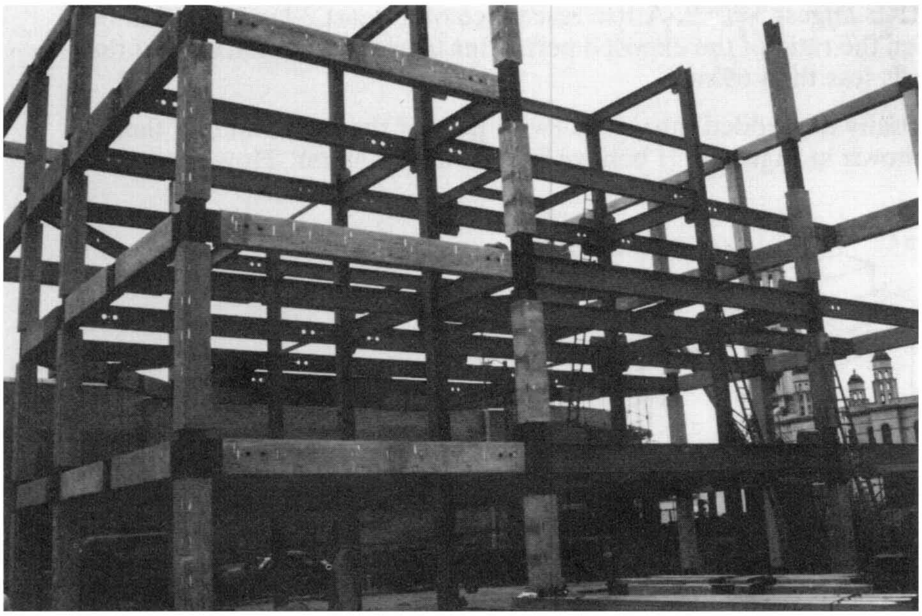


Figure 8.1 *Example of concrete encasement to beams and columns*

8.1.2 Brick-or blockwork

Brick-or blockwork can be used to fully or partially encase steel columns and is a practical method of providing the required degree of fire protection. Aerated concrete blockwork is a better insulant than brickwork and concrete. A number of fire tests have been carried out to establish the fire protection offered when concrete blocks are placed between the flanges of column sections (Figure 8.2). The behaviour of columns with 'blocked in webs' is

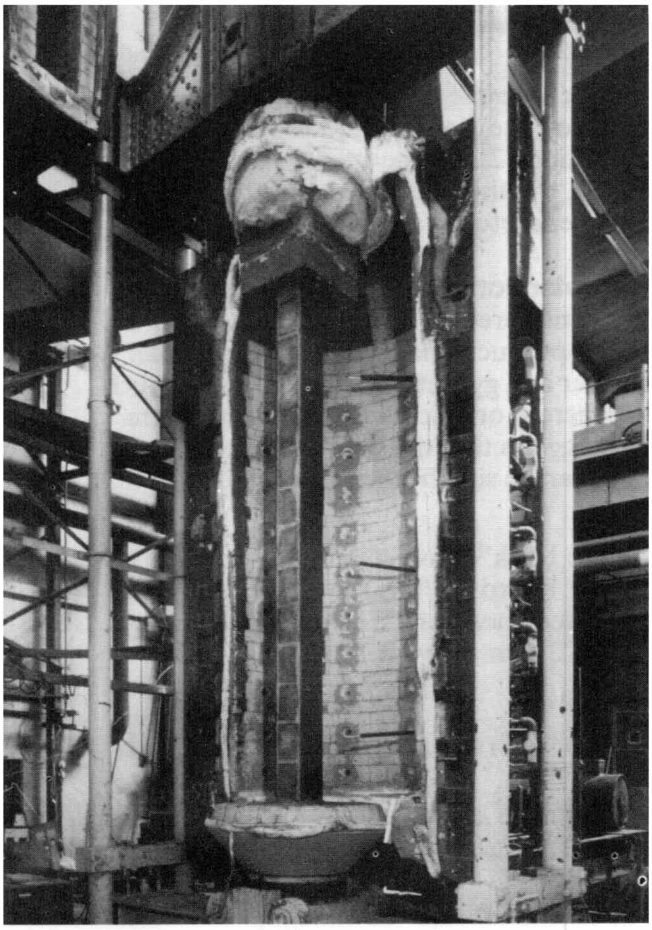


Figure 8.2 *Fire test on blocked-in column*

covered in BRE Digest 317[18]. A fire resistance of at least 30 minutes can be achieved when the ratio of the exposed perimeter to cross-sectional area of the steel column is less than 69 m^{-1}. 4.3.2

Columns partially embedded into walls (with part of the web and one flange exposed as shown in Figure 8.3) behave in a similar manner. However, the

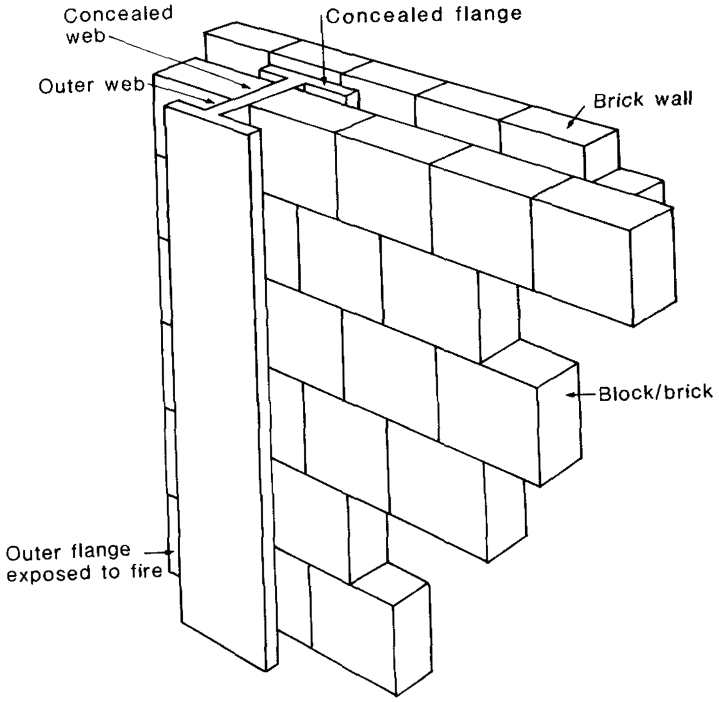


Figure 8.3 *Configuration of steel column built into a wall*

temperature gradient across the section can induce significant thermal bowing. The effect of these movements on the stability and cracking of the wall panels may need to be assessed (Section 12.4) if the steelwork is to be left unprotected. If fire protection is applied, the thickness of the protection may be assessed from the reduced exposed perimeter of the member. 4.10.4

8.1.3 Gypsum and plaster board protection

These generic materials are generally considered to be traditional means of protection despite the fact that they are products manufactured by a small number of suppliers. As materials, plaster and gypsum plaster board are much better insulants than normal weight concrete or brickwork. Their moisture contents can be as high as 20% which suggests that the heat (and hence time) required to drive-off excessive moisture can be significant and can enhance the fire resistance offered.

Reference to the British Gypsum 'White book'[23] is recommended for selection of appropriate fixing details for boards of this type. Two layers of board are often required when encasing steel members. The boards are held in place with steel wire binders (typically 1.2 mm wires at 100 mm spacing).

8.2 Performance of proprietary fire protective materials

8.2.1 Different systems for fire protection of steelwork

A number of different forms of proprietary fire protective materials are marketed. In simple terms these are:

- cementitious-type sprays, such as perlite-cement, vermiculite, vermiculite-cement, glass or mineral fibre-cement sprays

- fire boards, such as fibro-silicate, gypsum and vermiculite
- mineral fibre and other similar mat materials
- intumescent coatings.

There are a number of different manufacturers of each of these systems. Sprayed fire protection appears to be currently popular in commercial steel buildings where the floor soffit is hidden and where additional cladding is provided around the steel columns (see Figure 4.2). Box or board systems are more popular where the protection to the beams and columns is left exposed (see Figure 4.3).

Sprayed systems are usually applied in a number of layers. A priming coat applied to the steel section may be recommended by the manufacturer. The main advantage of sprayed systems is that they can easily protect complicated beam-column junctions, trusses and secondary elements. Their main disadvantage is the mess and dust created during spraying.

Board systems often use additional noggings and filler pieces between the flanges of the beam to which the boards are attached. Their method of jointing is important in order to prevent gaps opening up. Pre-formed box systems are also used.

Intumescent coatings are those which expand or 'intumesce' on heating, thereby offering protection to the steelwork. They are generally used for architectural reasons where the steelwork is left fully exposed. Thin intumescent coatings (1 to 2 mm thick) can provide up to 1½ hour fire resistance.
Good examples are in the many cast and wrought iron structures which have been renovated in recent years (Figure 8.4).

Figure 8.4 *Intumescent paint protection to cast iron columns and beams*

8.2.2 Traditional means of appraising the performance of fire protection materials

Traditionally, the performance of fire protective materials has been evaluated using the method in the 'Yellow Book'[(2)] coupled with an independent testing procedure. Two forms of test are required: insulation tests and 'stickability' tests. The former is concerned with determining the thickness of fire protection needed to keep the mean steel temperature at or below 550°C at a given fire resistance period. The latter is needed to ensure that the fire protection remains intact.

As noted in Section 4, the important parameter in determining the thermal performance of the steel section is the section factor, H_p/A. The mean temperature of a beam section in a fire test is taken as the average of the lower flange and the mid-point of the web.

A number of small scale unloaded fire tests can be carried out on different sections and with a range of thicknesses of protection. From these tests a regression formula can be developed of the form:

$$F.R = A_1 + A_2\, d_i\, A/H_p + A_3 d_i \qquad (16)$$

where d_i = thickness of protection (mm)
$F.R$ = time for the mean temperature to reach 550°C
A_1, A_2 and A_3 are constants

A_1, A_2 and A_3 are determined for the material under test by multiple linear regression over the range of the test data. This equation can then be used to determine the thickness of protection required for other sections and fire resistance periods. Physically, this formula has little meaning other than to broadly represent the influence of the two main parameters.

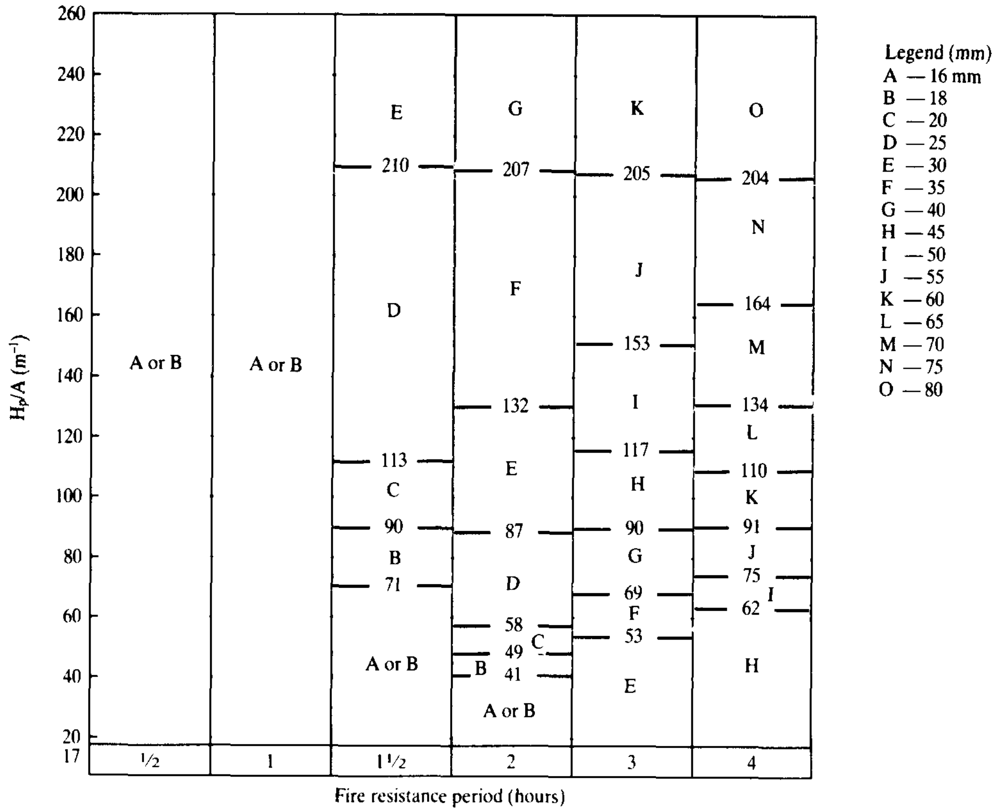
From Equation (16), the required thickness of fire protection can be determined as a function of H_p/A and the fire resistance period required. Examples of data in the 'Yellow Book'[(2)] for typical sprayed and box protection are presented in Tables 8.1 and 8.2. The thickness of sprayed

Table 8.1 *Minimum thickness of a typical spray protection to an I section*

H_p/A	Dry thickness in mm to provide fire resistance of					
up to	½ hour	1 hour	1½ hour	2 hours	3 hours	4 hours
30	10	10	10	10	15	25
50	10	10	10	14	21	29
70	10	10	12	17	27	36
90	10	10	14	20	31	42
110	10	10	16	22	34	47
130	10	10	17	24	37	51
150	10	11	18	25	40	54
170	10	12	19	27	42	57
190	10	12	20	28	44	59
210	10	13	21	29	45	
230	10	13	21	30	47	
250	10	14	22	31	48	
270	10	14	23	31	49	
290	10	14	23	32	50	
310	10	14	23	33	51	

Linear interpolation is permissible between values of H_p/A

Table 8.2 *Minimum thickness of a typical box protection to an I section*



protection can increase gradually, but the thickness of box protection increases in stages (typically 5 mm).

8.2.3 Stickability of fire protective materials

The 'stickability' of the fire protection is assessed using loaded beams and columns in a large test furnace. The recommended size of beam is 305 × 127 × 42 kg/m UB spanning 4.5 m (4.0 m heated length) as shown in Figure 3.5. Two beams are tested with the minimum and maximum thicknesses of protection that are likely to be used. The tests are carried out to a deflection approaching the test limit (span/30 in the former *BS 476: Part 8*). Typically tests would be terminated at a deflection exceeding span/40. At this deflection relatively high strains are developed in the steel flanges.

A column test is also carried out using a recommended section of 203 × 203 × 52 kg/m UC of 3 m height as shown in Figure 5.3. For assessment purposes the section is provided with the maximum thickness of fire protection. The purpose of both the beam and column tests is to identify if the protection cracks, or gaps open up, so permitting heat to reach the steel section prematurely. The 'stickability' limit therefore determines the permitted maximum fire resistance period, irrespective of the insulation criteria.

The 'stickability' test is important as it gives a measure of the strains that can be used in determining the strength reduction factor for the steel section (see Section 6.4). In carrying out beam fire tests there is a strong correlation between deflection and span:depth ratio of the test beam. The minimum flange strain which the beam should undergo consists of 1.5% mechanical strain and approximately 1.0% thermal strain, giving a total strain of about 2.5%. In order for this strain to be experienced at a deflection not less than span/40, the span:depth ratio of the test beam should not exceed 15, as illustrated in Figure 8.5.

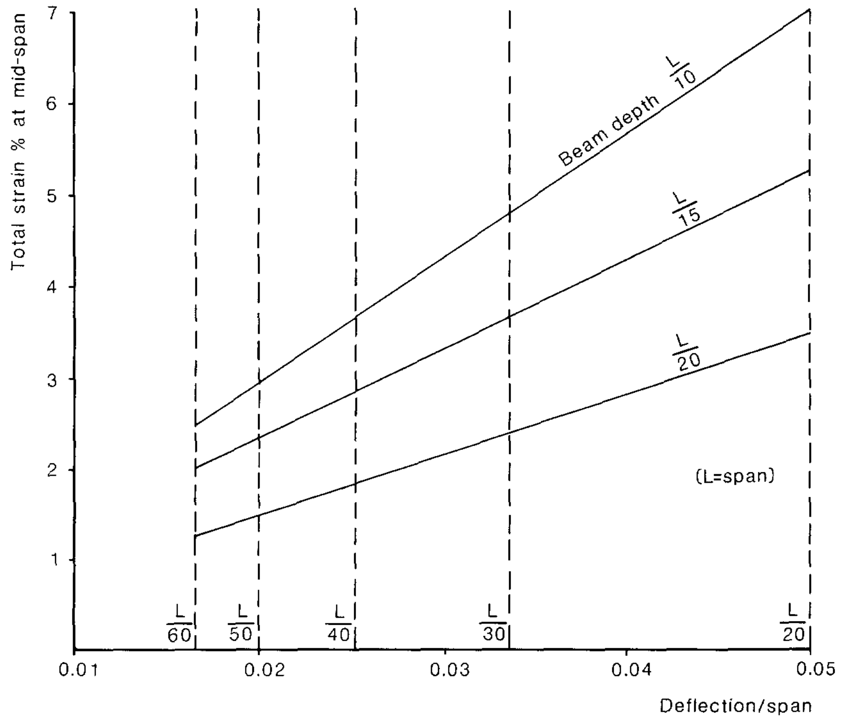


Figure 8.5 *Effect of testing different beam proportions on maximum strain at different deflection limits*

8.3 Design formula for determining thickness of fire protection

8.3.1 Generalized approach

The passage of heat through a fire protective material into a steel section is controlled by the differential Equation (12) in Section 4.4. Considering this general equation for 'heavy' protection materials and ignoring the minor terms, the temperature rise in the steel section is:

$$\delta\theta_s = \frac{H_p k_i}{A C_s \rho_s d_i} \cdot \frac{1}{(1+\xi)} \cdot (\theta_f - \theta_s)\,\delta t \qquad (17)$$

where H_p/A = section factor of the member
k_i = thermal conductivity of the fire protective material of thickness d_i
θ_s and θ_f are the steel and fire temperatures respectively at time t.

Most 'heavy' insulants have a specific heat of 1100 to 1200 J/kg°C and the specific heat of steel is taken as 520 J/kg°C. Therefore the value of ξ defined in Equation (12) may be approximated to:

$$\xi = \frac{d_i \rho_i}{\rho_s} \cdot \frac{H_p}{A}$$

where ρ_i and ρ_s are the densities of the insulation and steel repectively.

The standard temperature-time curve may be input into Equation (17). An example of the rise of steel temperature in a protected member is shown in Figure 4.4. The temperatures in natural fires (Section 13) may also be input, and the steel temperatures determined from this equation. An important observation of the temperatures in a natural fire is that heat enters the steel section as long as θ_f exceeds θ_s. Therefore, the maximum temperature in the steel section occurs when θ_s equals θ_f (i.e. on the declining part of the natural fire curve).

The ECCS Recommendations (9) express the steel temperature in a standard fire (derived from Equation (11)) in terms of an empirical formula. This is valid in the range of θ_s between 400 and 600°C. The time at which a steel temperature of θ_s is achieved is given by:

$$t_e = 40(\theta_s - 140)\left(\frac{d_i A}{k_i H_p}\right)^{0.77} \tag{18}$$

where t_e is in minutes
k_i is in W/m °C
H_p/A is in m^{-1}
θ_s is in °C

A study[24] undertaken during the development of *BS 5950: Part 8* has shown that better fit over the range of application is obtained with a coefficient of 38 (rather than 40) and a power of 0.7 (rather than 0.77). However for consistency it was decided to use the existing coefficients in the ECCS equation. Rearranging the above equation in terms of the required thickness of protection d_i gives:

$$d_i = \frac{H_p k_i}{A}\left(\frac{t_e}{40(\theta_s - 140)}\right)^{1.3}$$

$$= H_p \frac{k_i}{A} \cdot \frac{I_f}{10^6} \tag{19}$$

Appendix D

where I_f is an insulation factor in terms of the fire resistance period and the limiting temperature of the steel (m^3/kW).

Equation (19) may be extended to cover 'heavier' forms of fire protection by considering the generalised heat flow Equation (12) in Section 4.4. Rearranging this Equation in terms of d_i, it may be shown that Equation (12) reduces to:

$$d_i = \frac{-1 + \sqrt{1 + 4\dfrac{\rho_i}{\rho_s} \cdot \left(\dfrac{H_p}{A}\right)^2 k_i (I_f 10^{-6})}}{2\dfrac{\rho_i}{\rho_s} \cdot \dfrac{H_p}{A}} \tag{20}$$

Introducing a factor

$$\mu = k_i \frac{\rho_i}{\rho_s} \frac{I_f}{10^6} \left(\frac{H_p}{A}\right)^2 \tag{21}$$

gives

$$d_i = \frac{H_p k_i}{A} \cdot \frac{I_f}{10^6} \cdot F_\omega \tag{22}$$

F_ω is now a reduction factor representing the effect of the density of the fire protection, where

$$F_\omega = \frac{(1+4\mu)^{0.5} - 1}{2\mu} \tag{23}$$

Practically, F_ω has a value of between 0.6 and 1.0. Numerical data for I_f and F_ω are presented in Tables 8.3 and 8.4. It should be noted that this method is relatively conservative for low values of limiting temperature θ_s.

This method is consistent with ECCS Recommendations[9] and is a simplification of the ECCS approach. It is covered in Appendix D of the Code as a means of determining fire protection thickness. The only limitation of this

BS 5950 Part 8

Table 16

Table 8.3 *Insulation factor (I_f)*

Steel temp.°C	Insulation factor for fire resistance period of: ½ hr	1 hr	1½ hr	2 hr	3 hr	4 hr
400	500	1230	2100	3000	5100	7400
450	400	980	1650	2400	4100	5900
500	325	800	1360	1980	3350	4900
550	275	680	1150	1670	2850	4100
600	240	590	990	1440	2450	3550
650	210	510	870	1260	2150	3100
700	185	450	770	1120	1890	2750
750	165	405	690	1000	1690	2450
800	150	365	620	900	1530	2200

Table 17

Table 8.4 *Values of F_w as a function of effective density μ*

Fire protection material weight factor							
μ	0.0	0.05	0.1	0.5	1.0	1.5	2.0
F_w	1.0	0.95	0.92	0.73	0.6	0.55	0.50

method is the lack, at present, of representative thermal properties of specific rather than generic protection materials.

8.3.2 Effect of moisture content

Most fire protective materials contain moisture. When temperatures within the materials approach 100°C, further heat input does not increase the temperature of the material but vaporizes any free moisture (latent heat of vaporization). This causes a delay or 'dwell' in the temperature-time response of the protected steel section (Figure 4.4). This is a well-known phenomenon, and can be significant in materials where the stored moisture exceeds 3% by weight.

The 'dwell time' t_v can be determined by the formula:

$$t_v = \frac{p\ \rho_i d_i^2}{5\ k_i} \text{ minutes} \qquad (24)$$

where p is the percentage of moisture in the material.

The other units are as defined in Section 8.3.1.

Hence the total time taken to reach a limiting steel temperature θ_s is $t_e + t_v$ (minutes). However, if it is required to determine d_i in terms of t_e, then it is necessary to modify Equation (24). This is done by increasing the effective density of the insulation, according to the approximate relationship:

Appendix D

$$\rho_i' = \rho_i\,(1 + 0.03\,p) \qquad (25)$$

This formula has been shown to be accurate for protective materials achieving a fire resistance of up to 120 minutes. ρ_i' now replaces ρ_i in Equations (20) and (21).

8.4 Determination of material properties

The important parameters in determining the required thickness of fire protection are the thermal conductivity k_i, the density and moisture content of the fire protection material. Data exists on the performance of generic materials such as vermiculite or gypsum board and these are given in Table 8.5. For many materials there is a change in thermal conductivity with temperature as illustrated in Figure 8.6. The thermal response is less sensitive to changes in the density of the fire protection material.

Table 8.5 *Examples of the properties of some common fire protective materials*

Protective material	Density (kg/m^3)	% moisture content (by weight)	Specific heat C_i (J/kg°C)	Thermal Conductivity k_i (W/m°C)
Normal weight Concrete	2350	3	1300	1.3~1.7
Lightweight Concrete	1800	4~5	1200	0.6~0.8
Cellular blocks	1000	2.5	1200	0.4
Brickwork	2000	–	1200	1.2
Gyspum boards	800	15~20	1700	0.2
Fibre Silicate sheets	600~800	3	1200	0.15
Perlite or Vermiculite sheets	500~800	10~15	1100	0.15
Mineral wool slabs	800	2	1100	0.2
Vermiculite or Perlite cement	550	20	1100	0.12
Sprayed Mineral Fibre	300	1	1200	0.10

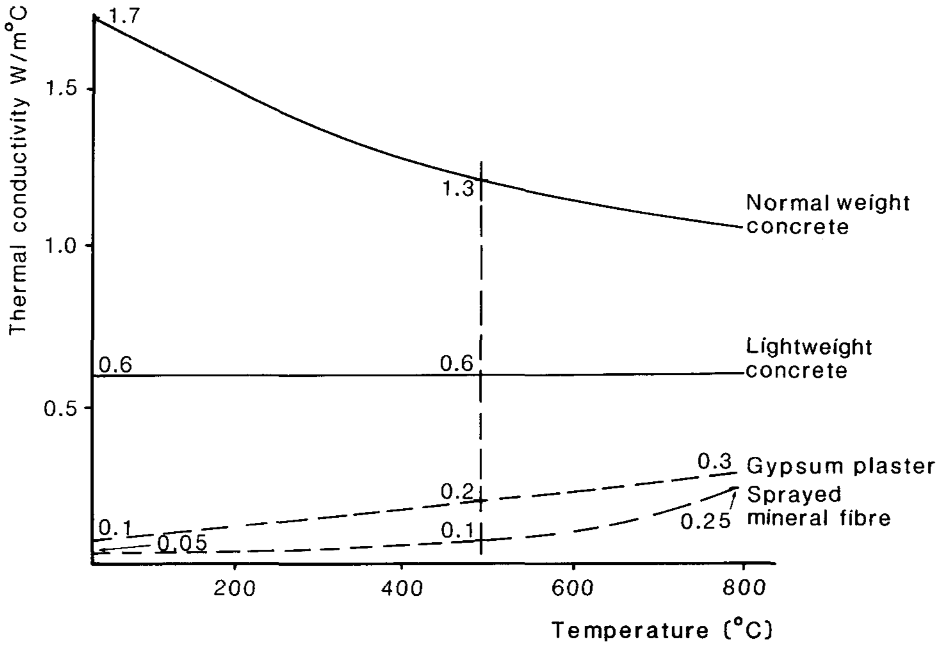


Figure 8.6 *Variation of thermal conductivity of typical materials with temperature*

It remains to set up an appropriate test or to modify an existing test to determine the thermal conductivity of the fire protection material. A steady state test may be developed whereby the quantity of heat passing through, and the temperature drop across the fire protective material can be measured directly. For calculation purposes the average thermal conductivity over the temperature range of 400 to 600°C is to be used.

Alternatively, the transient temperature exposure in a standard fire resistance test may be used. If it is assumed that Equation (19) is valid, then the value of k_i can be determined by analysing the thermal performance of a particular thickness of fire protective material. By implication, this value of k_i is an 'average' value up to the limiting temperature under consideration. The effect of the moisture content should be included, as in Equation (25). The values of k_i determined by the two methods may be slightly different.

In the future it is likely that manufacturers will be encouraged to carry out tests or to analyse existing fire test data to determine the thermal properties of their materials. This is also expected to be the approach adopted in the forthcoming Eurocode Annex relating to the fire resistance of steel structures.

9 BEHAVIOUR OF COMPOSITE DECK SLABS

'Composite construction' is the general term used to denote the composite action of steel beams and concrete floors. 'Composite deck slabs' are those where profiled steel decking acts as permanent formwork and as reinforcement to the concrete placed on top (Figure 9.1). Design of composite deck slabs is covered by *BS 5950: Part 4*[25]. 4.9.2

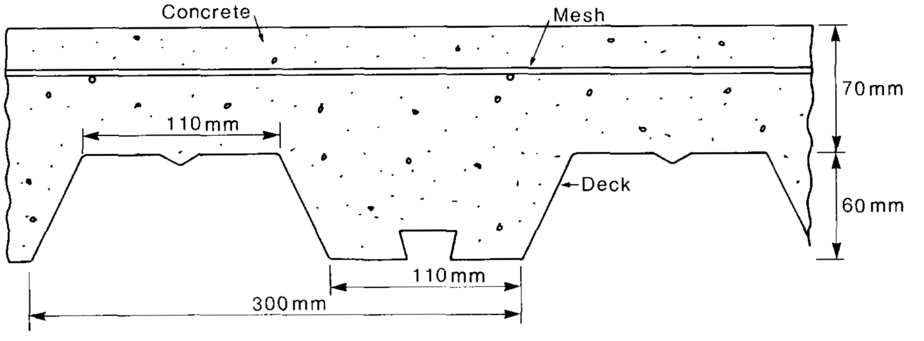


Figure 9.1 *Section through a typical composite slab*

Composite deck slabs are usually of 100 mm to 150 mm overall depth with spans ranging from 2.5 to 4.0 m. Composite slabs are not usually propped during the construction stage. Several decks are marketed with steel thicknesses of 0.8 to 1.5 mm. A common feature of these decks is the use of embossments or indentations to improve the mechanical bond between the concrete and the steel.

9.1 Minimum slab depths

The minimum depth of a composite deck slab is determined on the basis of the 'insulation' criterion in *BS 476: Part 20*[5]. In *BS 8110*[26] terms, a reinforced concrete ribbed slab is a slab with ribs at relatively wide spacing and therefore the minimum slab depth is considered to be the depth of the concrete topping. Composite deck slabs comprising trapezoidal profiles are treated in this way, but the 'heat-sink' effect of the closely spaced ribs means that minimum slab depths can be relaxed relative to those in *BS 8110.* Composite slabs comprising dovetail profiles behave effectively as solid slabs. 4.9.2.2

An extensive series of fire tests has recently been completed by the Fire Research Station (in 1986) to determine the thermal characteristics of 40 different configurations of composite deck slabs. The slab depths ranged from 90 to 196 mm and the deck types were chosen to be representative of current practice. A full range of tests was carried out on both normal (gravel aggregate) and lightweight (Lytag aggregate) concrete. The individual modules tested were each nominally 1 m × 0.65 m.

The temperatures rise on the upper unexposed surface of floors is limited to 140°C as a mean or 180°C as a peak temperature. In determining temperatures on, and close to the upper surface of the slab, account should be taken of the 'dwell' or delay in temperature at around 100°C resulting from the vaporization of free moisture. It is conservative to reduce the temperature-time curves by the total apparent 'dwell-time', although a certain percentage of moisture (2% to 3% by weight) is present in all concrete.

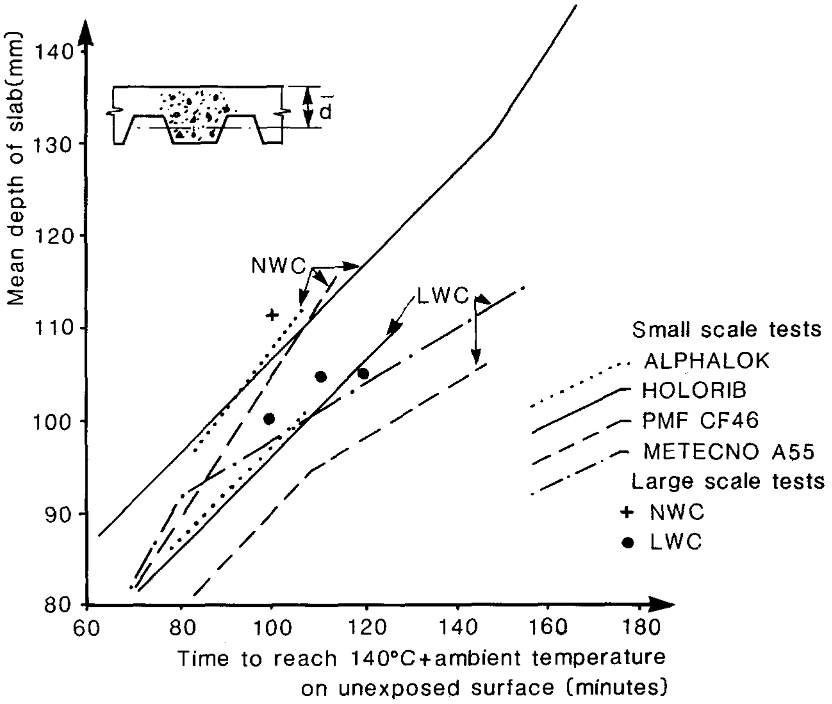


Figure 9.2 *Fire insulation tests on composite slabs expressed in terms of mean slab depth*

One form of presentation of the results for a range of deck profiles is in terms of the mean depth of concrete, which is the approach used by the ECCS[27]. Figure 9.2 shows the test data presented in this way. This largely eliminates the effect of profile shape. The graphs for normal and lightweight concrete appear to be displaced by around 20 minutes, or conversely, by a slab depth of 10 mm. It should be noted that the ECCS data for normal weight concrete predicts mean slab depths some 20 mm lower than suggested by the above test data, and are not considered appropriate for design use.

For consistency with current UK practice, minimum slab depths have been expressed in terms of the overall slab depth of 'dovetail' profiles and minimum topping depth for trapezoidal profiles. These minimum slab depths have been relaxed significantly for slabs with trapezoidal profiles relative to the early guidance based on ribbed concrete slabs[14]. 4.9.2 Table 14 Table 13

9.2 Review of test data

Since 1982, a considerable number of large-scale tests on composite deck slabs has been carried out by various research organizations in the UK. The objective of the research was to show that for the normal range of slab depths, imposed loads and decking types, fire resistances of up to 90 minutes could be achieved with only mesh reinforcement in the concrete. This period of fire resistance would be required for most large commercial buildings (except in basement or storage areas). For greater periods of fire resistance, additional reinforcement would be required normally in the form of bars placed in the troughs of the deck.

Before 1982, few fire tests had been performed on composite deck slabs continuous over internal supports. Simply supported slabs with mesh reinforcement demonstrated fire resistances of 30 to 60 minutes and, therefore, most of the existing tests included additional reinforcing bars. The beneficial effect of structural continuity was known from reinforced concrete design, and a series of large-scale fire tests was devised on two-span slabs representing the end bays of continuous floors.

A resumé of all the composite slabs that have been fire-tested in the UK since 1982 is presented in Table 9.1. Also indicated are the reinforcement patterns

Table 9.1 *Summary of UK fire tests on composite slabs*

Profile	Concrete type	Slab depth (mm)	Span (m)	Imposed load (kN/m^2)	Reinforcement	Surface temp. (C) After 1 h	Surface temp. (C) After 1½ h	Test period (min)	Test ref.
Robertson QL59	LWC	130	3.0s	6.7	A142 mesh	73	–	60	CIRIA 1
Roberton QL59	LWC	130	3.0c*	6.7	A142 mesh	70	100	105	CIRIA 2
Robertson QL59	LWC	130	3.0c	6.7	A142 mesh	95	110	90	CIRIA 3
Holorib (UK)	LWC	120	3.0c*	6.7	A142 mesh	60	100	90	CIRIA 4
PMF CF46	LWC	110	3.0c	5.25	Y5@225 as mesh	110	135	101	FRS–BS1
Holorib (UK)	LWC	100	3.0c	5.75	Y5@150 as mesh	90	120	87†	FRS-BS2
PMF CF46	NWC	135	3.0c	6.75	Y5@225 as mesh	85	95	120 (136)	FRS-BS3
Robertson QL59	NWC	140	3.6c*	6.7	A193 mesh	66	98	90	CIRIA 5
Metecno A55	NWC	140	3.6c*	6.7	A193 mesh	65	95	90	CIRIA 6
Holorib (UK)	LWC	150	3.0c*	10.0	A193 mesh	45	61	120	R.LEES 1
Ribdeck 60	LWC	140	3.0c*	5.6	A193 mesh	64	93	136	R.LEES 2
Ribdeck 60	LWC	140	3.0c*	8.5	A252	56	77	149	R.LEES 3

The tests are in chronological sequence from July 1983 until July 1989
† failed prematurely because of loss of protection to beams
** test on long span/short span configuration*

and the test loads used. In the series of fire tests carried out by FRS–BS, special cranked mesh was made, whereas in the other tests, standard mesh was used. Testing of all the slabs was carried out after storing for 5 to 6 months in dry conditions, so that the concrete could attain close to its equilibrium moisture content.

For much of the early part of fire tests on composite floors the increase in deflection is a result of temperature-induced curvature and only later by structural weakening. The displacement time curve for the floor is approximately linear until close to failure when the rate of deflection increases significantly. Failure is determined when a limiting deflection of span/20 is reached as the rate of deflection is usually below the limit recommended by *BS 476: Part 20*[(5)] at this stage.

An important observation in all the tests was the absence of severe debonding of the deck from the slab during the fire tests. A typical view inside the test furnace is shown in Figure 9.3. On cooling, some debonding often occurred because of irreversible extension of the deck relative to the concrete.

Figure 9.3 *Internal view of fire test on composite slab*

Temperatures at various points within a typical slab section are shown in Figure 9.4. Reinforcement temperatures were generally low indicating that the full strength of the mesh could be mobilized in these tests. All the tests indicated that 90 minute fire resistance could be achieved, provided there was structural continuity of the slab and its mesh reinforcement. There was no adverse effect on the behaviour of the composite beams resulting from the deformation of the composite slabs in fire.

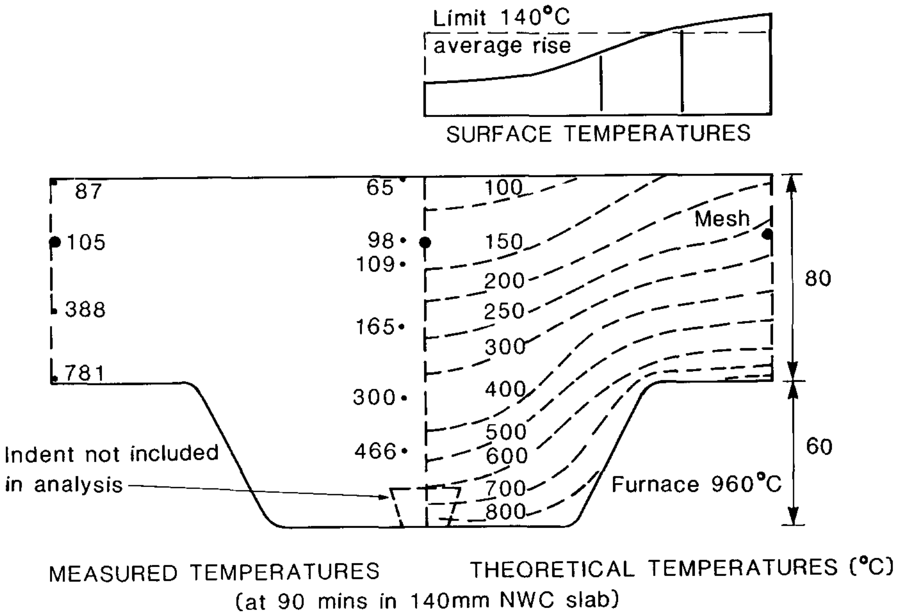


Figure 9.4 *Comparison of measured and theoretical temperatures in composite slab*

9.3 Fire engineering method

A method of calculating the fire resistance of reinforced concrete structures was first proposed in the Institution of Structural Engineers report on *Design and detailing of concrete structures for fire resistance*[14]. In principle, this fire engineering method uses measurements of temperature rise in concrete sections, together with the known behaviour of steel and concrete at elevated temperatures to calculate the reduced moment capacity of the section. The appropriate fire resistance period of simple beams and floors can be ascertained by comparing the reduced moment capacity of the section with the applied moment in fire conditions.

A publication by the Steel Construction Institute[28] has modified the method in reference (14) to reflect the fire behaviour of composite deck slabs and is also referred to in the Code. The temperature distribution within the section is given in the Code and assumes that the depth of a given point within the section is measured normal to the surface of the steel deck. This approach is accurate for points along the centre-line of the rib of the section for normal trough widths. The temperatures in lightweight concrete are approximately 20% lower than in normal weight concrete. 4.9.2 Table 12

The fire engineering method assumes that the plastic moment capacity of the section can be developed at elevated temperatures, and that redistribution of moments can take place in continuous members. Shear failure should not occur before flexural failure.

The elevated temperature properties of concrete and reinforcing steel are taken from references (14) and (28). As a simplification of the actual behaviour, the strength reduction factors may be taken as in Section 3.4. The beneficial properties of lightweight concrete are again apparent in terms of its less rapid decrease in strength with temperature than normal weight concrete.

The tensile capacity of the deck is ignored in fire engineering calculations although the paper by Cooke, Lawson and Newman[29] suggests that it would be possible to justify a residual tensile capacity of 5% of the 'cold' capacity of the deck for composite decks designed to *BS 5950: Part 4*[25]. Indeed, this partly explains why the fire engineering method under-predicts the fire resistance of the composite slabs that have been fire-tested.

In continuous composite slabs there is a redistribution of moment during a fire from the mid-span area to the supports. At failure, the moment capacities of the slab in hogging and sagging (negative and positive moment) may be combined, as in an equivalent plastic failure mechanism. The hogging capacity would remain close to its 'cold-state' value whereas the sagging capacity would diminish considerably.

The condition for plastic failure of an end span of a continuous composite deck slab is given by the formula:

$$M_p + 0.5\,M_n\left(1 - \frac{M_n}{8M_o}\right) \geq M_o \qquad (26)$$

where M_p = sagging and
M_n = hogging moment capacity of the composite section
M_o = free bending moment applied to the simply-supported slab in fire conditions.

The second term approximates to a value of $0.45\,M_n$. For an internal span of a continuous slab the plastic moment capacity is simply:

$$M_n + M_p \geq M_o \qquad (27)$$

In general, it is the end span condition that determine the amount of reinforcement that is needed. It is important that the reinforcement in the hogging moment region has adequate ductility. Reinforcement to *BS 4449* achieves this by specifying a minimum elongation of 12%. Mesh to *BS 4483* should be required to achieve this ductility if used in fire engineering calculations or in the simplified method below.

9.4 Simplified method

Guidance on the fire resistance of composite deck slabs is presented in *CIRIA Special Publication 42*(30) and is now embodied in the SCI Publication *The Fire Resistance of Composite Floors with Steel Decking*(28). Simply-supported slabs are considered to have a fire resistance of 30 minutes but continuous slabs can achieve a fire resistance of 90 minutes. Minimum slab depths and mesh reinforcement are given in Tables 9.2 (a) and (b). This applies for imposed loads up to 6.7 kN/m^2.

For longer periods of fire resistance, or heavier imposed loads or longer spans, the fire engineering method is to be used consistent with the minimum slab depths for fire insulation purposes. The simplified method does, however, allow the designer to interpolate from the tabular data by comparing the equivalent moment in the design case to the nearest tabulated case.

Table 9.2 *Simplified rules for fire resistance of composite slabs*

(a) Trapezoidal profiles (depth not exceeding 60 mm)

Max. span (m)	Fire rating (h)	Minimum dimensions: Sheet thickness (mm)	Slab depth (mm) NWC	Slab depth (mm) LWC	Mesh size
2.7	1	0.8	130	120	A142
3.0	1	0.9	130	120	A142
	1½	0.9	140	130	A142
3.6	1	1.0	130	120	A193
	1½	1.0	140	130	A193

(b) Dovetail (re-entrant) profiles (depth not exceeding 50 mm)

2.5	1	0.8	100	100	A142
	1½	0.8	110	105	A142
3.0	1	0.9	120	110	A142
	1½	0.9	130	120	A142
3.6	1	1.0	125	120	A193
	1½	1.0	135	125	A193

9.5 Composite beams

Composite beams achieve composite action by use of shear-connectors between the steel beam and concrete or composite slab[31]. Relatively few fire tests have been carried out on composite beams to failure,but it is possible to evaluate the likely fire performance by the moment-capacity method. 4.9.4

The behaviour of a composite beam in fire conditions is illustrated in Figure 9.5. Initially, elastic conditions apply when the beam is loaded at room temperature. At the limiting temperature of the lower flange, the temperature distribution through the steel section is of the form of Figure 6.1.

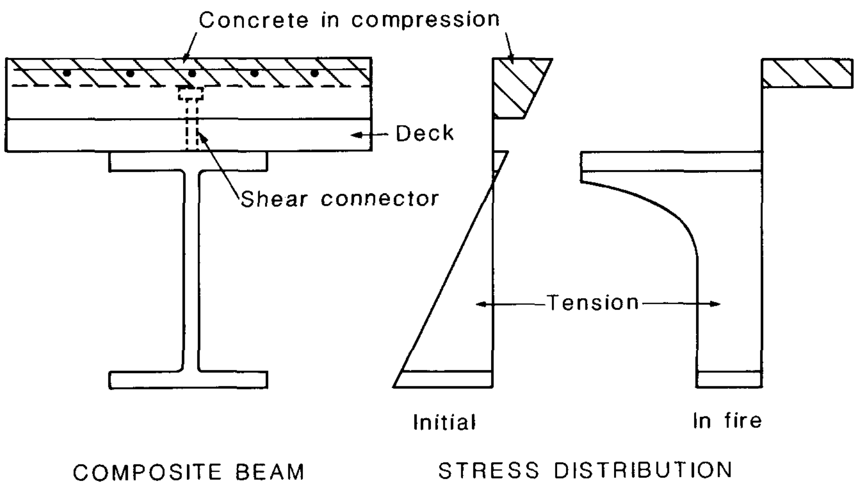


Figure 9.5 *Stress distribution in composite beam in fire*

Consequently, the stress distribution has changed in that now all of the steel section is in tension, and this is balanced by a compressive force in the upper part of the concrete.

Hence, in fire, the plastic neutral axis of the composite section usually lies in the concrete slab. For a given curvature and hence deflection, higher bottom flange strains are generated than in a simple beam. A strain limit of 2% may be used in assessing the moment capacity of the section, subject to the 'stickability' of the fire protection (see Section 6.4) 2.3 (c)

The concrete or composite slab is almost fully effective in compression, as only the bottom 20 mm or so of concrete will be adversely affected by heat. Indeed, the area of concrete over the beam is well insulated from the fire. There is potentially some reduction in the strength of composite beams arising from the deflection of the slab between the beams, thus lowering the centroid of the effective breadth of the concrete flange. This is a relatively small effect and it may be assumed that the compressive capacity of the slab is reduced in total by about 20% at a fire resistance period 90 minutes. However, this is offset by the use of a reduced material factor for concrete (see Section 2.3).

Recent tests on composite beams in fire suggest that it is not necessary to fill the voids created above the steel beam by the dovetail deck profile. This observation may be extended to trapizoidal deck profiles for fire resistance periods of up to 60 minutes. Guidance is currently being prepared on this subject.

Moment capacity calculations suggest that the same limiting temperatures as for non-composite sections may be used. This is a relatively simple approach and is a relaxation on the previous guidance[32], where it was suggested that limiting temperatures should be reduced by 50°C. This means that the same thickness of fire protection is used in non-composite and composite beams for the same load ratio. There is no adverse effect on the shear connectors because the longitudinal force transferred reduces as the temperature of the steel section increases. 4.9.4

10. SHELF ANGLE FLOORS

The shelf angle floor consists of an I section with angles welded or bolted to the webs. Precast concrete planks are positioned on the angles, and the sizing of the angles is normally controlled by practical aspects of the construction, such as the the bearing distance of the plank from the web of the beam. In-situ concrete is placed in the void around the beam.

A simplified method for verifying the fire resistance of shelf angle floors is included in an Appendix to the Code. It is based on the moment capacity method (see Section 7) and includes a tabular method of obtaining the temperature distribution of the assembly. Appendix E

In a shelf angle floor beam the concrete floor shields the upper part of the steel beam from a fire below the floor. It was realised that if the shelf angles were placed with their legs pointing upwards, i.e. the reverse of the conventional arrangement, the concrete would also protect the legs of the angles, giving improved fire resistance under load. A number of fire resistance tests have been carried out on shelf angle floor beams and they showed that substantial load could be carried for up to 90 minutes in a fire resistance test without the need for fire protection to be applied to the lower part of the beam.

10.1 Review of test data

Seven loaded fire resistance tests have been carried out on shelf angle floors of various geometries and with varying levels of load[12]. The tests were all carried out on 4.5 metre span beams with a typical cross-section as shown in Figure 10.1. The angles were bolted to the beams. Loads were applied via the

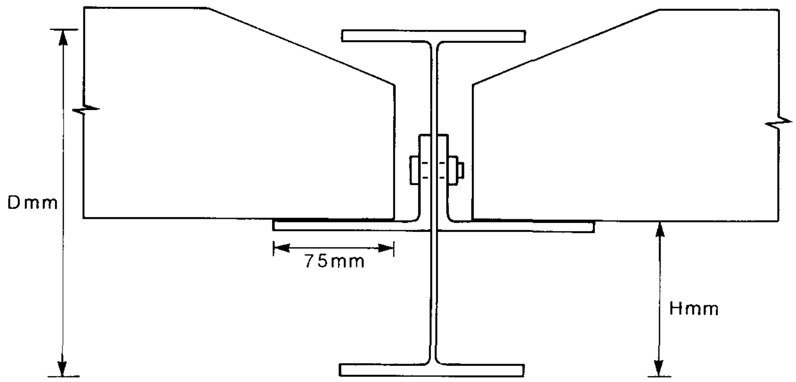


Figure 10.1 *Test arrangement for shelf-angle floor*

concrete units. These units were specially cast and spanned from the furnace edge to the shelf angles. In the tests dry sand was used to fill the void between the beam and the concrete units to facilitate construction and to avoid any composite action. In practice a grout would be used. The main results of these tests are summarised in Table 10.1.

It can be seen from these results that the fire resistance is dependant on both the load ratio and the ratio of exposed web to total steel depth. However, the relationship is not simple and it has not been possible to present this information in a simple graphical or tabular form. The load ratio method cannot therefore be used when considering shelf angle floor beams. Nevertheless, it appears that 60 minutes fire resistance can be achieved for a load ratio of 0.5 when the ratio of the exposed web to the total depth of the section does not exceed 0.5.

Table 10.1 *Summary of fire test data on shelf angle floors*

Beam	Grade	Angle	Grade	Span (metres)	Load Ratio	*F.R.* (Mins)	*H/D**	θ_1*	θ_2*	θ_3*	θ_4*
406 × 178 × 54	43	125 × 75 × 12	50	4.5	0.56	67	0.43	915	824	611	94
305 × 165 × 40	43	125 × 75 × 12	50	4.5	0.58	83	0.28	924	878	617	92
406 × 178 × 54	50	125 × 75 × 12	50	4.5	0.23	94	0.45	992	945	716	103
406 × 178 × 54	43	125 × 75 × 22	50	4.5	0.52	29	0.70	733	571	368	97
406 × 178 × 54	43	125 × 75 × 12	50	4.5	0.41	70	0.45	914	839	613	94
406 × 178 × 54	43	125 × 75 × 12	50	4.5	0.25	74	0.70	915	803	616	222
254 × 146 × 43	43	125 × 75 × 12	50	4.5	0.54	91	0.31	970	925	675	133
254 × 146 × 43	43	125 × 75 × 12	50	4.5	0.55	69	0.43	912	843	615	158

**For dimensions H and D and for temperature details see Figures 10.1 and 10.2*

10.2 The temperature distribution in shelf angle floor beams

An investigation of the temperature distributions recorded in the tests showed that temperatures could be modelled with a good degree of accuracy using a simple tabular method. The distribution was found to be of the form shown in Figure 10.2. Appendix E3

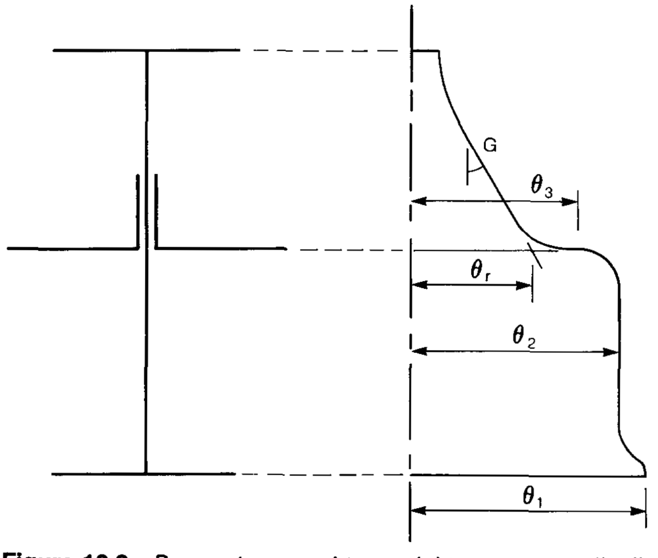


Figure 10.2 *Parameters used to model temperature distribution*

The procedure for obtaining the temperature distribution in shelf angle floors is given in Table 10.2. The temperature correction taking into account the shielding offered by the slab is presented in Table 10.3. The temperature of

Table 10.2 *Method of obtaining shelf angle floor temperatures*

Location on Section	Reference Temperature	D_e/B_e Correction
Bottom flange, θ_1	Table 7	Table 8
Exposed web, θ_2	Table 18	Table 18
Exposed angle, θ_3	Table 18	
Angle root θ_R	Table 18	
Gradient G	Table 19	

NOTE: Table numbers are as given in the Code

Table 10.3 *Temperature reduction for lower flange of beams*

Aspect ratio D_e/B_e	Design temperature reduction °C for fire resistance period of :			
	½ hr	1 hr	1½ hr	>1½ hr
up to 0.6	80	40	20	0
over 0.6 to 0.8	40	20	0	0
over 0.8 to 1.1	20	0	0	0
over 1.1 to 1.5	10	0	0	0
over 1.5	0	0	0	0

D_e = exposed depth of web
B_e = flange width

the upper part of the beam is not needed accurately as it is below 300°C and can be considered to be acting at its full strength. The 300°C boundary can lie either in the web or in the upper part of the angles.

A comparison between the temperature distribution given by the Code and the measured distribution is shown in Figure 10.3. This comparison was made for a 406 × 178 × 54 grade 43UB with exposed web ratio (H/D) of 0.43 after 60 minutes of a fire resistance test.

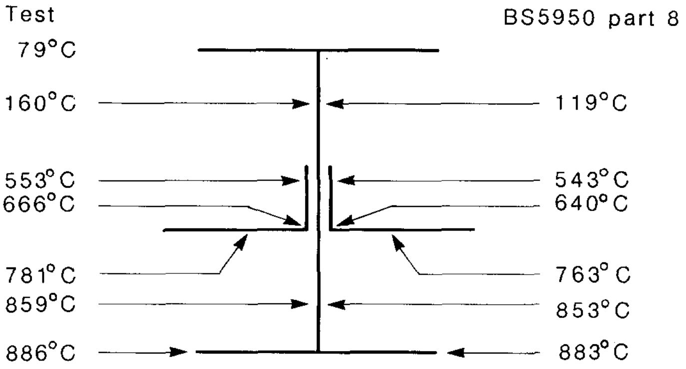


Figure 10.3 *Temperatures in a shelf angle floor beam after 60 minutes (406 × 178 × 54 kg/m UB with H/D = 0.43)*

10.3 Strength of shelf angle floor beams

The Code presents a method of calculating the strength of shelf angle floor beams based on the moment capacity method. The shelf angles are included in the calculation of the reduced strength of the section in fire, although they would not normally be included for ordinary design. The angles should not be less than 125 × 75 × 12 (grade 50) with the longer leg horizontal. Appendix E1 (d)

10.3.1 Bending strength of shelf angle floors

The bending strength may be checked using the moment capacity method. Appendix E2 This method uses the temperature distribution in Section 10.2 to determine the reduced axial strength of the various segments of the section, and hence to calculate the moment capacity of the member in fire. The stress distribution through a typical shelf angle floor is shown in Figure 10.4. This calculation assumes that strains exceeding 1.5% are reached in the angle (see Section 3). 2.3 (b)

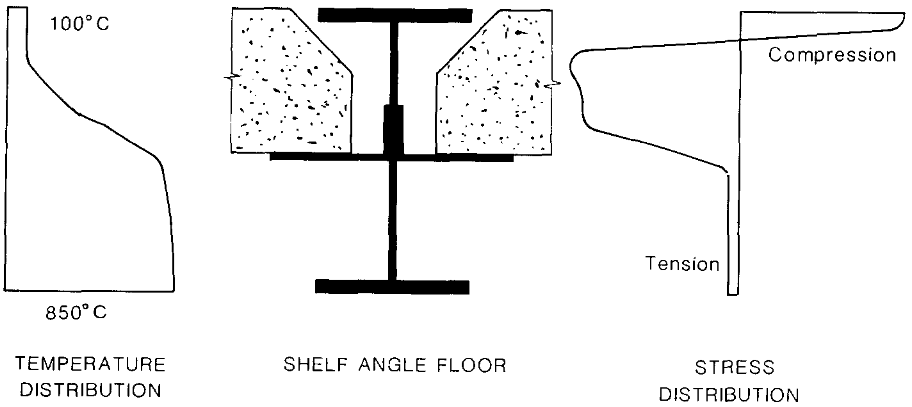


Figure 10.4 *Stress distribution in shelf angle floor*

Calculations by the above method are conservative because some composite action occurs at the large deflections experienced in a fire test. It is thought that the precast units become wedged between the shelf angle and the underside of the top flange of the beam. In practice grout filling the gap between the beam and the precast units would provide a greater heat sink and more composite action than was present in the fire tests. Further research is being carried out which may lead to the utilization of the additional capacity due to composite action.

10.3.2 Transverse bending of angle

In a shelf angle floor the main load path to the beam is via the angles that support the precast slab. In this case the angles are exposed to the fire so the bending capacity of the angles must be checked at the appropriate steel temperature. Although, the minimum bearing is 75 mm, in a fire the precast units can be assumed to bear on the angles close to their ends. In the design example the load is assumed to act 10mm in from the end of the unit. The bending capacity of the leg of the angle is based on 1.2 $p_y Z$ rather than $p_y S$ because the shape factor (S/Z) of a rectangular section is greater than 1.2[1]. Appendix

10.3.3 Connection of angle to beam

In all the fire tests to date the angles have been bolted to the beam and have performed satisfactorily. Tests are shortly to be carried out on beams with welded angles. The Code allows either bolts or welds to be used and gives some guidance on the strength of these connections at elevated temperature. The strength of welds is taken as equivalent to the strength of steel at the same temperature but an additional safety factor of 0.8 is included. The Code specifies that the 0.5% strain reduction factor should be used in the design of the bolts and welds. A low strain limit is used because of concern over the ductility of connections relative to the steel member. Further research is being carried out into the behaviour of connections in fire. Appendix E1 (g)

The longitudinal shear force which the bolts or weld must transmit depends on the size and strength of the angles. The axial force in the angle is calculated from its reduced strength at the fire limit state. This force is assumed to have developed in the angle over a distance between the end of the beam and the point of maximum moment. The shear per unit length that the bolts or welds must transmit can then be calculated based on uniform shear flow at the large deformations at the fire limit state. The process is very similar to the calculation of shear connection in composite beams. If welding is used, any weld below the angle i.e. exposed directly to the fire is considered to be ineffective. These welds will be at a much higher temperature than welds above the angle and will also be subject to higher strains.

10.3.4 Deflection of shelf angle floors in fire tests

The deflection of shelf angle floors can be computed by dividing the beam into segments along its span. Each segment is subject to a strain distribution so that the moment capacity of the section at a particular temperature distribution equals the applied moment. By integration of strains along the span of the beam, the central deflection can be determined. This is illustrated in comparison to test results in Figure 10.5.

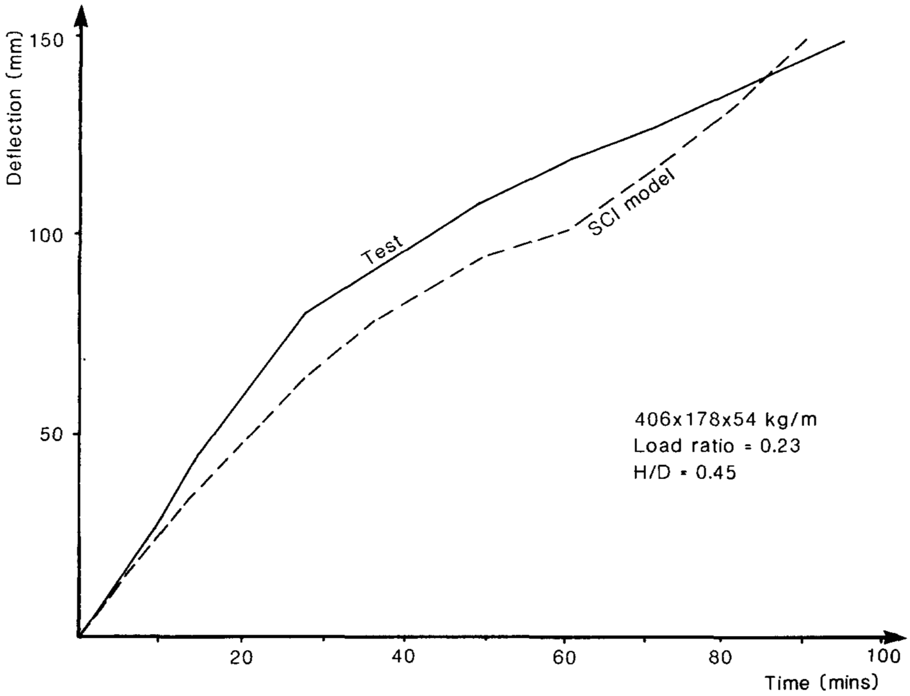


Figure 10.5 *Comparison between observed and computed deflection for shelf angle floor beam*

11. STRUCTURAL HOLLOW SECTIONS

11.1 General

The performance of structural hollow sections (SHS) in fire is generally very good. This is because the heated perimeter of the section is much smaller than that of an I section for the same cross-sectional area. The torsional resistance, beneficial at normal temperatures, is maintained at elevated temperatures so lateral instability problems do not occur.

The fire resistance of structural hollow sections can be improved by applying external insulating materials and/or filling the section with concrete or water. When a board system is used, which encases the section, a similar amount of radiant heat reaches the SHS as that which reaches an encased I or H section and hence the required thickness of fire protection is the same.

However, there is some evidence that SHS heat up faster than I or H sections of the same section factor (H_p/A) with *profile* protection of the same thickness. This is because there are no outstanding flanges to shield parts of the perimeter so the general level of radiant heat received is higher. Therefore the Code specifies a factor that should be applied to the thickness of the fire protection if based on tests for profiled protection to an I or H section.

Protection thickness for SHS (profile protection) =

Protection thickness for I or H section × Multiplication Factor,

where Multiplication Factor $= \left[1 + \frac{(H_p/A)}{1000}\right] > 1.25$ (28) 4.3.3.4

Thus for a 200 × 200 × 6.3 RHS used as a column with a section factor of 165 m^{-1} it follows that Equation (28) results in a fire protection thickness 16.5% greater than for an I or H section of similar section factor (say 203 × 203 × 60UC with a section factor of 160 m^{-1}). However because the perimeter of the RHS is smaller the total amount of material required for the RHS is still approximately 25% less than for a Universal Column. For unprotected sections no modifications are required to the limiting temperature method presented in Section 6.

The fire protection of circular hollow sections with box systems warrants a special assessment of the section factor. If the enclosing square of the section is used, the heated perimeter will be higher than if the circumference is used and this could lead to the use of an increased amount of fire protection. This would imply that box encasing a circular section, which introduces air gaps in the corners, actually makes the rate of heating worse! In view of this the circumference is used as the heated perimeter of a circular hollow section.

11.2 Concrete filled structural hollow sections

Various forms of composite columns are illustrated in Figure 11.1. The structural design of concrete filled structural hollow sections will eventually be covered by *BS 5950: Part 3.2*. Design recommendations for concrete filled sections are covered in the UK by British Steel publication *Design manual for SHS concrete filled columns*[33]. This is itself based on the requirements of *BS 5400: Part 5*[34]. 4.6

The presence of load bearing concrete within a hollow steel column has a beneficial effect on the fire resistance of the steel section; in many cases adequate periods of fire resistance can be obtained by concrete filling without

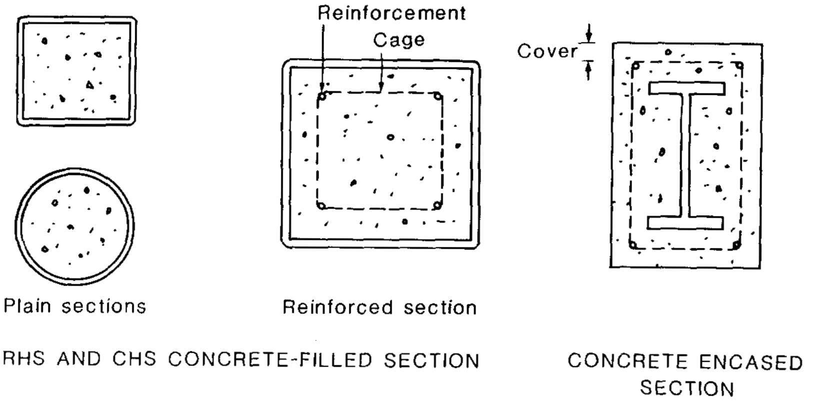


Figure 11.1 *Examples of composite columns*

the need for external protection (see Table 11.1). Further improvements in fire resistance can be obtained by reducing the applied loads or by neglecting the contribution of the concrete in the 'normal' design of the column.

Table 11.1 *Fire resistance of concrete filled hollow section columns as a function of load ratio* Table 9

Fire Resistance (Min)	Load ratio η	
	Plain concrete	5% Fibre concrete
30	–	–
60	0.51	0.67
90	0.40	0.53
120	0.36	0.47

fire resistance can be obtained by reducing the applied loads or by neglecting the contribution of the concrete in the 'normal' design of the column.

Steel fibres can be added to the concrete, or longitudinal bar reinforcement placed within the concrete to further improve fire behaviour to extend the fire resistance time or alternatively, enable the column core to withstand higher axial loads or significant moments without the need to apply external protection.

The method of verifying the fire resistance of concrete filled structural hollow sections was developed as the result of an extensive fire test programme supported by ECSC and CIDECT. Over 100 fire resistance tests were carried out under this programme. These tests were largely on rectangular rather than circular sections so the guidance in the Code is restricted to rectangular sections unless externally applied fire protection materials are used.

The test programme was also restricted to sections filled with normal weight concrete so although some benefit would be gained from using lightweight concrete its use is excluded from the Code.

During heating, concrete gives off large volumes of steam and so *the provision of vent holes in the section is very important.* Failure to adequately vent the steam could lead to splitting of the steel shell. Two holes should be provided at 4 metre centres or at storey height positions, whichever is more frequent. For structural hollow sections up to 400 mm square, 12 mm diameter vent holes 4.6.1
have been demonstrated to be sufficient.

Examples of concrete-filled columns used in external and internal applications are shown in Figure 11.2 and 11.3.

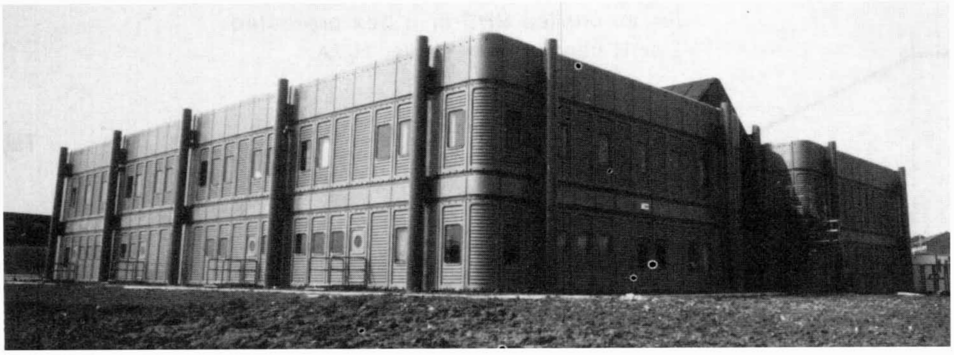


Figure 11.2 *External concrete-filled columns*

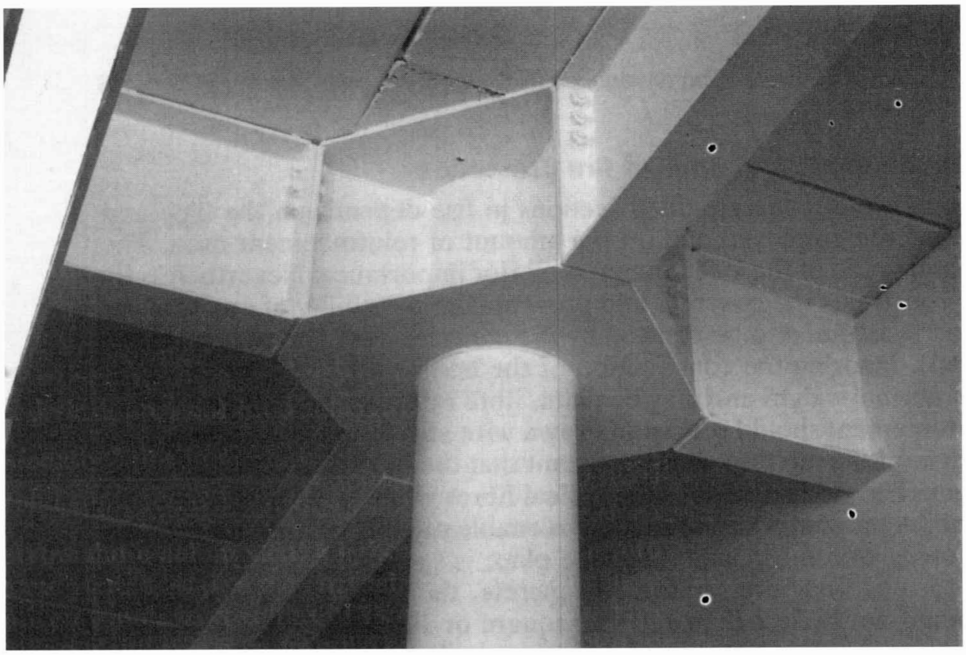


Figure 11.3 *Detail of junction between concrete-filled column and beams*

11.2.1 Externally applied fire protection

The filling of structural hollow sections with concrete increases their thermal capacity and therefore reduces the rate at which they heat up in fire. This increase in their inherent fire resistance in comparison to unfilled sections means that in some circumstances reduced amounts of fire protection can be used. In Table 11 of the Code a protection thickness reduction factor is given. This factor is to be applied to the protection thickness required for the unfilled section and can be applied to both profile and box systems. For profile systems, if an I or H section is used for the basis of calculation of protection thickness, the modification factor discussed in Section 11.1 must also be included. The factor given in the Code is shown graphically in Figure 11.4. 4.6.3 Table 11

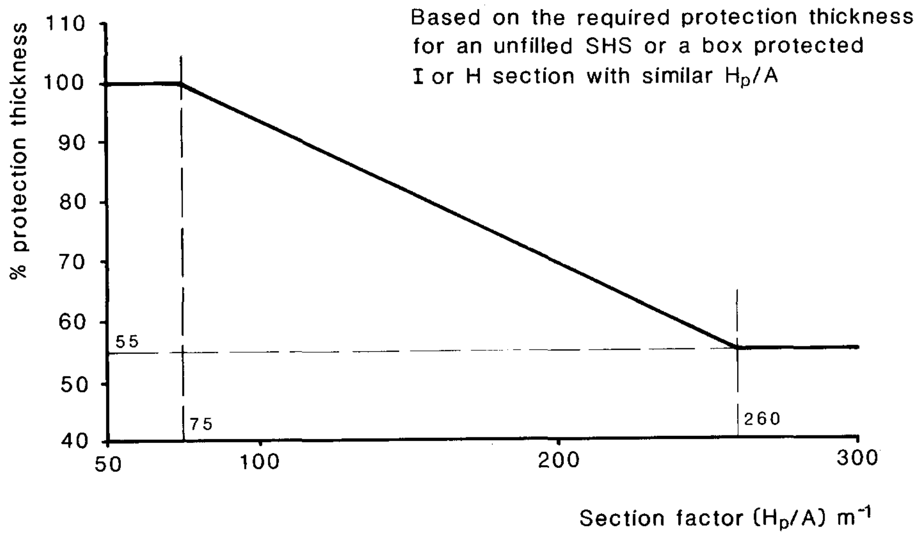


Table 11

Figure 11.4 *Effect of concrete-filling on required thickness of fire protection in RHS or CHS columns*

11.2.2 Sections without applied fire protection

The performance of concrete-filled sections in fire depends on the type and grade of concrete employed and on the amount of reinforcement used. The thickness and grade of the steel are not of major importance. Nevertheless the steel tube constrains the concrete and prevents the possibility of any spalling. All the load is assumed to be resisted by the concrete core and reinforcement (if provided), ignoring the contribution of the hollow section. The concrete must be of normal weight and may be plain, fibre reinforced or bar reinforced. Fibre reinforcement should consist of drawn wire and form approximately 5% by dry weight of the mixture. It is important that the fibres have adequate pull out strength. The Code describes the typical fibres which were used in the test programme. A minimum size of section to enable reliable filling with concrete and positioning of reinforcement to take place is specified. To use this approach for plain or fibre reinforced concrete, the Code specifies that the section should not be less than 140 mm square or 100 mm × 200 mm rectangular section. For bar reinforced columns these limits are increased to 200 mm square or 150 mm × 250 mm rectangular.

4.6.2
4.6.1

11.2.3 Strength of plain or fibre reinforced columns

The strength of concrete-filled hollow sections is verified in the Code by use of a time dependent load ratio. The capacity of the columns ignores the contribution of the steel section. A column is considered to have a particular fire resistance if the load ratio is less than the limiting value given in Table 11.1. The load ratio is based on the expression given in Reference (33) but has been extended to cover biaxial bending and written in terms of loads and moments rather than stresses.

4.6.2.1
Table 9

From Reference (33), the load ratio is:

$$\eta = \frac{p_a + p_b}{0.83 K f_{cu}} \qquad (29)$$

If biaxial bending is included the load ratio becomes:

$$\eta = \frac{p_a + p_{bx} + p_{by}}{0.83 K f_{cu}} \qquad (30)$$

where $p_a = \dfrac{N_f}{A_c}$

$p_{bx} = \dfrac{6M_x}{d_y d_x^2}$ (elastic)

$p_{by} = \dfrac{6M_y}{d_y^2 d_x}$ (elastic)

Consequently,

$$\eta = \frac{\dfrac{N_f}{A_c} + \dfrac{6}{d_y d_x}\left[\dfrac{M_x}{d_x} + \dfrac{M_y}{d_y}\right]}{0.83 K f_{cu} A_c}$$

but $d_y d_x = A_c$

$$\eta = \frac{N_f + 6\left[\dfrac{M_x}{d_x} + \dfrac{M_y}{d_y}\right]}{0.83 K f_{cu}} \qquad (31)$$

The condition that no part of the section is in tension is satisfied when:

$p_a \geqslant p_{bx} + p_{cx}$

i.e. $N_f \geqslant 6\left[\dfrac{M_x}{d_x} + \dfrac{M_y}{d_y}\right]$ (32)

where

p_a = compressive stress on core
p_{bx} = bending compressive stress due to M_x
p_{by} = bending compressive stress due to M_y
d_x = depth of section normal to xx axis
d_y = depth of section normal to yy axis
A_c = area of core
N_f = compressive load
M_x = moment about xx axis -positive)
M_y = moment about yy axis (positive)
K = buckling factor from Table 10 in the Code
f_{cu} = characteristic cube strength

Any concrete-filled column designed following the recommendations of reference (33) has at least half an hour fire resistance.

11.2.4 Strength of bar reinforced columns

The method given in the Code is broadly similar to that for plain or fibre reinforced columns but the general form of the load ratio is different. Again, any column has a minimum of half an hour fire resistance.

The load ratio is based on the expression given in reference (33) but has again been extended to cover biaxial bending.

BS 5950
Part 8

The load ratio is: 4.6.2.2

$$\eta = \frac{N_f}{K(0.83 f_{cu} A_c + f_y A_r)} \left[\frac{M_{px}}{M_{px} - M_x} \right] \tag{33}$$

This is extended to cover biaxial bending in the Code by modifying the term in brackets to:

$$\frac{1}{1 - \frac{M_x}{M_{px}} - \frac{M_y}{M_{py}}} \tag{34}$$

where

f_y = yield stength of reinforcement
A_r = area of reinforcement
M_{px} = plastic moment capacity of the reinforcement about the xx axis
M_{py} = plastic moment capacity of the reinforcement about the yy axis

The Code limits the amount of reinforcement to 4% of the core area. If for practical reasons, for example, 5% reinforcement is used, only 80% of it may be used in calculating M_{px} M_{py} and A_r. It is important, however, to ensure that the section is properly filled and compacted when large amounts of reinforcement are used. The cover to the reinforcement should be in accordance with *BS 8110*[26] and should be measured from the surface of the bars to the edge of the concrete core.

11.2.5 Buckling factor K

All the expressions for the time dependent load ratio are of the form:

$$\eta \leqslant \frac{\text{Load at fire limit state}}{K \times \text{squash load capacity at normal temperature}} \tag{35}$$

The buckling factor, *K*, relates the axial capacity of the column, taking into account its effective length and slenderness, to the squash load of the concrete section. In the Code, *K* is given for various values of slenderness, (*L*/*r*). This table is based on *BS 5400: Part 5*[34], Table 13.1 and should be considered to be slightly conservative. 4.6.2.2 Table 10

The radius of gyration is calculated for plain or fibre reinforced concrete in the usual way but for bar reinforced columns both the second moment of area and the area are weighted to reflect the different properties of steel and concrete as follows:

$$r = 0.43 \sqrt{\frac{450 f_{cu} I_c + E_r I_r}{0.83 f_{cu} A_c + f_y A_r}} \tag{36}$$

where

I_c = second moment of area of concrete core in the plane of buckling
I_r = second moment of area of the reinforcement in the plane of buckling
E_r = modulus of elasticity of reinforcement.

11.3 Water-filled sections

The filling of a hollow section with water improves its fire resistance and this principle is permitted by the Code. However, the design of a viable water cooled structure is very complex and beyond the scope of both the Code and this publication. Reference (35) deals with the subject in more detail.

It is normal to limit the water filling to columns or sloping members only. Horizontal members may be affected by formation of steam pockets.

It is also normal to interconnect the columns to permit gravitational circulation and to have some form of replenishment system to compensate for steam losses. In multi-storey applications it might be necessary to partition the system horizontally to limit the hydrostatic pressure.

Although the term 'water-filled' is generally used, practical systems must contain various additives such as anti-freeze, corrosion inhibitors and fungicides.

A possible water filling arrangement is illustrated in Figure 11.5.

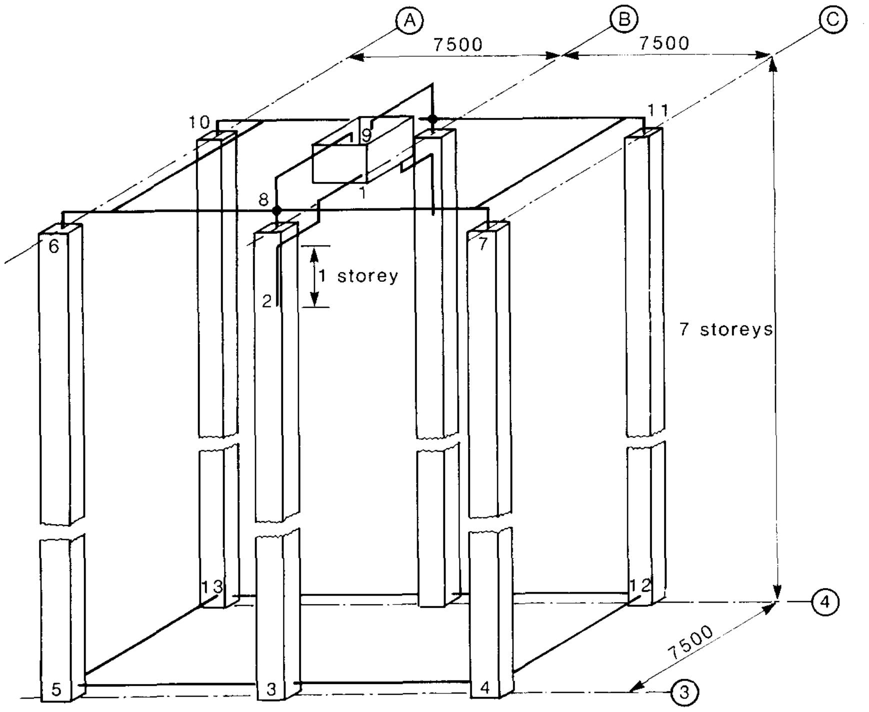


Figure 11.5 *Diagrammatic arrangement of water cooling system (for one zone)*

12. BEHAVIOUR OF OTHER ELEMENTS AND STRUCTURES

12.1 Portal frames

Building regulations concerned with the spread of fire from one building to another sometimes require the designer to demonstrate that the collapse of an unprotected portal frame rafter in fire will not cause collapse of a boundary wall. This check is only necessary where the boundary wall is required to have fire resistance, for example where it is close to another building. The Code states that the design is satisfactory if the bases and foundations of the boundary columns are designed to resist the forces and moments generated by rafter collapse and the columns are adequately fire protected. Specific guidance on calculation methods is given in the Appendix to the Code and reference is made to the Steel Construction Institute's publication *Fire and Steel Construction: The behaviour of steel portal frames in boundary conditions*(36). This publication is recognised by all the UK regulatory authorities. Appendix F

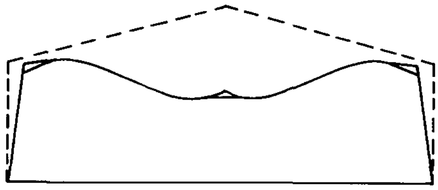
The design method contained in Appendix F of the code is taken directly from the SCI publication(36) with some clarifications and simplifications.

It should be noted that the Code and the regulatory authorities are only concerned with problems associated with unprotected rafters. The method given is not appropriate for frames with protected rafters as these will not undergo large deformations in a fire. The authorities in England and Wales have recently indicated that this check may not be necessary for sprinklered buildings.

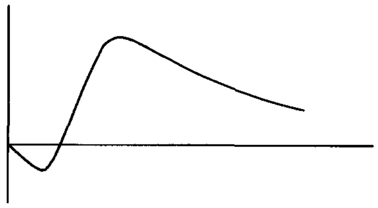
12.1.1 Behaviour of portal frames in fire

In a severe fire a portal frame will initially spread at the eaves as the rafter expands and the columns bow slightly. As the rafter increases in temperature and begins to lose strength, plastic hinges will form at the eaves and apex and it will start to collapse.

The rafter may buckle sideways in order to pass through the horizontal position before eventually hanging as some form of catenary. At this point the eaves will be pulled inwards. If prolonged heating occurs the rafter will slowly subside until it touches the ground. During a fire the base overturning moment will initially act outwards and then reverse to act inwards. This is shown diagramatically in Figure 12.1. To avoid collapse of the column, the peak moment acting 'inwards' must be considered.



Rafter sags until an equilibrium position is reached



The variation of overturning moment with time

Figure 12.1 *Extreme deformation of portal frame in fire conditions*

The form of the catenary action is interesting as calculations on the strength of actual portals which have been in fires has shown that in order to remain standing the rafter must still have a degree of bending resistance, and is often described as a 'stiff' catenary.

12.1.2 Forces and moment at column base

The expressions given in Appendix F of the Code for horizontal reaction (HR) and the overturning moment (OTM) are both in two parts.

$$HR = K\left[W_f SGA \frac{CM_{pr}}{G}\right] \nless 0.1\,\frac{M_{pc}}{Y} \qquad (37)$$

Appendix F1 (2)

$$OTM = K\left[W_f SGY\left(A + \frac{B}{Y}\right) - M_{pr}\left(\frac{CY}{G} - 0.065\right)\right] \nless 0.1\,M_{pc} \qquad (38)$$

Appendix

where W_f = Load at time of collapse (kN/m^2) (inclusive of remaining roof cladding)
S = distance between frame centres (m)
G = distance between ends of haunches (m)
Y = vertical height of end of haunch (m)
M_{pr} = plastic moment capacity of rafter (kNm)
M_{pc} = plastic moment capacity of column (kNm).
K = 1 for single bay frames or as taken from a Table for multi-bay frames — Table 21
A and C are taken from Tables — Table 20

$$B = \frac{L^2 - G^2}{8G}$$

where L = span (m) of frame

The part of these expressions involving the applied load W_f is caused by catenary action. The remaining part of these expressions is the relief term due to the residual rafter strength.

The above expressions have been derived for the case where the ratio of the span of the rafter to the height of eaves is greater than 1.6. This is done so that the simplified method above is reasonably accurate with respect to the rigorous method in reference(36). The simplified method becomes over-conservative when this ratio is less than 1.6.

Under certain circumstances both expressions can give negative results. This does not mean that the columns are about to collapse outwards but that the assumptions built into the model are no longer valid and the answers are meaningless. In reality if either expression is negative it is an indication that rafter collapse may not occur. The situation is more likely to exist if for some reason the frame is overdesigned. To ensure that some form of base fixity is provided a lower positive limit has been specified for both the horizontal reaction and overturning moment. This is based on 10% of the plastic moment capacity of the column.

The calculation of vertical reaction is straightforward. However, for ease of foundation design it should not be underestimated as difficulty can arise in designing a foundation to carry a large moment and small vertical load.

12.1.4 Other considerations

(a) Restraint

Appendix F4

The requirements for restraining the column in the plane of the wall are to ensure that the columns do not collapse if longitudinal bracing becomes ineffective in a fire. This is normally achieved by providing four holding down bolts in the base plate at a minimum spacing equal to 70% of the flange width. An additional requirement is that the column should be properly restrained under normal conditions by steel side rails, or masonry walls. It is not necessary to fire protect the side rails or bracing members.

(b) Loading
The load at the time of rafter collapse will be considerably less than the full design load. Snow loading may be ignored and it is likely that the fire may have damaged the roof cladding causing reduction in dead load. The percentage of dead weight of roof cladding systems remaining may be obtained from the Code and Reference (36). It is based on an assessment of various roofing systems carried out by the Fire Research Station. Table 22

For frames with heights to eaves greater than 8 metres wind loading should be considered. Changes in frame geometry and loss of roof cladding may lead to modified wind pressure coefficients (see Reference (36)).

(c) Non boundary columns
Only the bases of boundary columns require special attention even though the collapse model is based on symmetric behaviour. This is because the collapse of a 'pinned' non-boundary column will almost invariably cause the overturning moment on the boundary column to be reduced. The rafter will hang from the boundary column at a steeper angle and it may also touch the ground. The catenary force will therefore be reduced resulting in a lower overturning moment.

(d) Internal columns
Normally it is not necessary to fire protect internal columns unless they are built into a separating wall or unless the frame is high compared with the span. For multi-span frames the factor K in the expressions for HR and OTM, is either 1.0 or 1.3 depending on the frame geometry. For most frames, K will be 1.0. Appendix F1 Table 21

The factor takes into account the effect of the partial collapse of an internal column adjacent to a boundary column. Observations of internal unprotected columns in fires showed that although the columns may buckle, the top of the column will remain vertically above the base and the reduction in height is unlikely to be significant. However, a reduction in height causes an increase in overturning moment on relatively tall frames. If the column adjacent to the boundary column is protected, K may be taken as 1.0 for all geometries.

(e) Bases and foundations
The holding down bolts are designed for a partial safety factor of 1.2 when subject to the moments in Equation (38). This is to avoid premature failure of the bolts which are less ductile than column or foundation failure. The overturning resistance of the foundations is assessed using the ultimate bearing strength of the soil and a partial safety factor of unity. This suggests that this is the preferential mode of failure. A figure of 1° base rotation has been assumed in the analysis. Greater rotations of the columns cause a reduction in the overturning moment. Appendix F2

12.2 Connections in frames

There are two forms of connection in framed buildings; those that resist only shear and axial forces in 'simple' construction and those that also resist moment in 'braced' or 'unbraced' frames.

Both forms of connection are generally designed to resist the forces applied to them at the ultimate limit state, rather than to be equivalent to the strength of the members which they connect. This means that the concept of 'load ratio' in the design of the beams is to be used with care. Take, for example, a beam that is understressed at the ultimate limit state, but its connection designed to be fully utilised at this load. In fire, the load ratio for the connection is therefore in the range of 0.55 to 0.65, but the load ratio for the beam is considerably less, permitting the use of higher limiting temperatures in the beam than in the connection.

Therefore, in making design recommendations for the design of connections, it is assumed that the elements of the connection (i.e. bolts, welds and plates) are fully stressed under factored loads. There is no evidence that connections are the 'weak-link' in the design of frames. Indeed, investigations of real fires show that the deformations of the main members can be gross before there is any sign of deformation in the connections.

12.2.1 Temperatures in connections

Bolted connections are of various types, the most common forms being the welded end plate with bolts to the column, and the cleated connection with bolts to the beam and column. The evidence from fire tests is that the temperatures in the region of the bolts are significantly lower than those in the members because of the 'massivity' of the plates and bolts themselves. Typically, the temperatures in the shank of the bolts could be 150°C lower than in the critical element of the member. This suggests that bolted connections perform well in fire.

The temperatures in the welds (particularly fillet welds) may in some cases be close to the temperatures of the plates which they connect. This means that in welded connections, the welds themselves may become the critical elements. Plates welded at right-angles (e.g. end plates) can offer some shielding to the weld. Similarly, the extra thickness of fire protection in corners adjacent to these welds helps to reduce their temperature relative to the other elements.

The possibility of connecting members with different section factors or 4.3.3.5
limiting temperatures is recognised by the Code. In determining the thickness of fire protection to be applied to the connection, the thickness is to be based on the member with the highest section factor at the connection.

Additional forces may be developed in the connections as a result of restraint to thermal expansion of the members. This effect generally causes compression and shear in the connections. However, there is little evidence that this form of loading has led to premature failure.

12.2.2 Strength of bolts in fire

Grade 4.6 bolts are forged from mild steel, whereas grade 8.8 bolts are manufactured from micro alloyed steel which is quenched and tempered (at around 450°C) to obtain higher strength. The 0.2% proof stress of grade 8.8 bolts is 2.33 times greater than grade 4.6 bolts whereas, the ultimate tensile strength is only 1.46 times greater.

The behaviour of these bolts subject to elevated temperatures in isothermal tests is presented in reference (16). There is a significant difference between the tensile strength and the 0.2% proof stress of the bolts for all temperatures. When 'normalised' with respect to their room temperature values, the difference between the normalised tensile strengths of the two types of bolt is small, and is similar to that of grade 43 or 50 steel.

The shear performance of bolts is similar to their tensile performance. Currently (1990), tests are underway at British Steel to establish the anisothermal performance of 20 mm diameter grade 8.8 bolts subject to shear or tension.

Friction grip bolts behave in a similar manner to other high strength bolts. There is likely to be some relaxation of friction because of expansion of the bolt, but at worst the bolt will slip into 'bearing'.

12.2.3 Bolted connections in simple construction

Connections in nominally simple frames are designed to resist shear and axial forces, but in fire can offer considerable restraint to the connected members. This reduces the moment in the members and increases their limiting temperatures and, hence, fire resistance for the same loading conditions. (Figure 12.2). This is the subject of continuing research and guidance will be published shortly by the Steel Construction Institute.

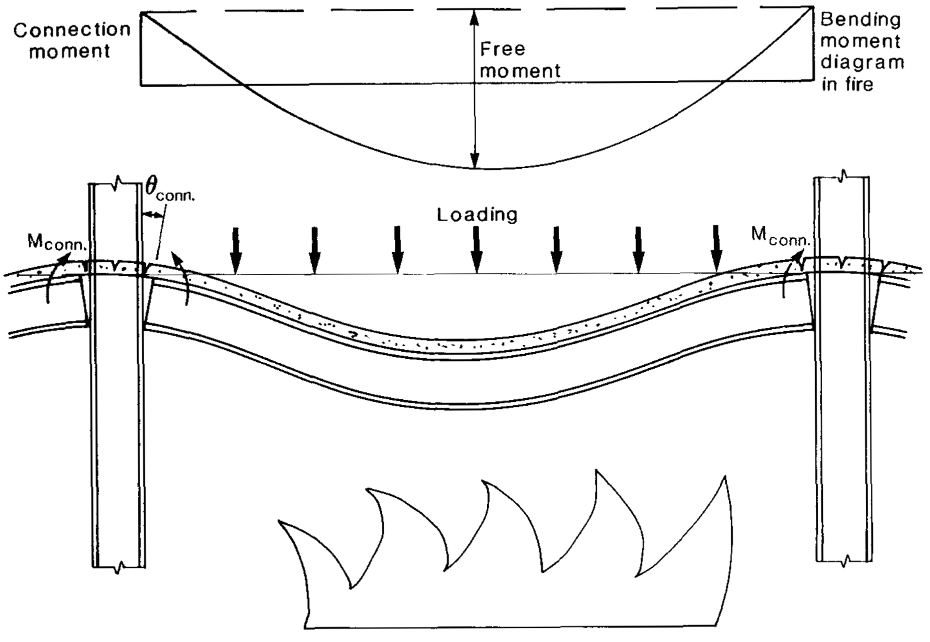


Figure 12.2 *Re-distribution of moment in simply-supported beams in fire conditions*

Continuity in fire conditions can be readily developed by internal beam to column connections in large plan buildings. End plate-type connections are capable of resisting a significant proportion of their moment capacity in fire. This is partly because the bolt shank temperatures are often up to 150°C below those of the lower flange of the adjacent beam. Extended end plates also benefit from the embedment of the upper bolts in the concrete slab.

The shear performance of the bolts may become more important in designs where the limiting temperature of the beams increases. However, it is evident from tests that shear (combined with tension) tends not to be critical. This may become a limitation in beams designed to less than 50% of their design capacity under normal conditions (i.e. load ratio ≈0.3) in which the connection is fully stressed in shear under normal loads.

12.2.4 Moment-resisting connections

Examples of moment-resisting connections are typically found in portal frames and in low rise frames. In modern frame construction for commercial buildings, moment-resisting connections are rarely used, the sway stabilty of the building being provided by bracing or shear walls.

The moments developed in the connections in fire may well exceed the limit for which they are designed initially. This is partly because of thermally induced curvature in the beam. It is important that the connection has appropriate rotational capacity, so that close to collapse it can redistribute its load to other parts of the frame.

In the design of moment-resisting sway frames it is suggested that the load-ratio method is modified to include the effect of notional forces, or alternatively, a single limiting temperature corresponding to a load ratio of approximately 0.67 is adopted. This is discussed in Section 6.5.

12.3 Castellated beams

The behaviour of castellated beams in fire has been investigated by a series of six fire tests; three on castellated sections with sprayed fire protection and three with box protection[37]. The conclusion from these tests was that the greater heated perimeter of castellated beams leads to higher temperatures in the lower section of the web and flange. One suggested approach was to increase the heated perimeter of the uncut section by 25%. However, a simplified and conservative approach as given in the Code is to increase the thickness of fire protection by 20% based on the H_p/A value of the uncut section. 4.3.3.3

12.4 Walls and roofs

4.10.3

Internal walls are often designed as compartment boundaries and therefore should possess appropriate fire resistance. However, most walls are not free standing and rely on lateral support by the structure.

The main concern of this clause is the deformation of the support structure in a fire and the effect it has on the integrity of compartment walls. The Code states that 'where a fire resisting wall is liable to be subject to significant additional vertical load due to the increased vertical deflection of a steel beam in a fire, either: 4.10

(a) provision should be made to accommodate the anticipated vertical movement of the frame or,

(b) the wall should be designed to resist the additional vertical load in fire conditions.'

Walls constructed of masonry will usually be sufficiently robust that they can support any loads transferred from a member above. The more important case is that of a tall slender fire resisting wall such as in a warehouse where compartmentation is very important. Such walls are not normally able to resist significant vertical loads.

For the purposes of this clause the anticipated vertical movement of a support beam in a fire should be taken as span/100, unless a smaller value can be justified by considering the loads applied to the member. The relevant span is that of the beam or floor in the horizontal plane at the top of the wall. This value represents the potential deformation of a protected beam subject to fire from one side only at the appropriate fire resistance of the wall (assuming the wall and beam have equivalent fire resistance).

Deflection of the support beam up to this magnitude should be accommodated by sliding restraints in the plane of the wall which permit vertical but prevent horizontal movement. These details should not permit passage of smoke or flame and are therefore crucial to maintaining the integrity of the wall. Manufacturers of purpose made systems have usually made due allowance for this effect.

An additional case referred to in the Code is that of an independent or free standing fire-resisting wall. There may be cases where thermal bowing or other thermally induced movement of the steelwork attached or adjacent to it, or thermal bowing of the wall itself may give rise to additional forces in the wall. This clause is not intended to apply to columns as part of frames which are built into walls and where the walls rely inherently on the columns for lateral support. Masonry usually has sufficient out-of-plane flexibility to accommodate thermal movement of this type. 4.10.4

Where it is necessary to calculate the magnitude of thermal bowing the guidance in reference (38) may be used. These movements are approximately

13 times greater in free standing or cantilever columns than in propped cantilever columns typical of most frames. It follows that free standing columns and walls respond much more to thermal movements than walls that are supported on four sides or columns that are connected to beams.

Walls close to a site boundary also need special consideration (as noted in Section 12.1 on portal frames). It may also be necessary to check against the possibility of a fire external to the building.

Generally no special considerations are necessary for roofs as they are not required to have fire resistance. However some authorities ask for the roof area in a 1.5 m wide band on either side of a compartment wall to have fire resistance. This is to prevent fire spread over the top of the wall. However many experts believe that this is an unnecessary provision and it is much more important to fire stop any voids or movement gaps that may exist at the junction of the wall and the roof. Material used for fire stopping must be capable of taking up any movement that may occur in fire conditions. 4.11

12.5 Ceilings

Suspended ceilings may in some cases be used to provide additional fire resistance to the floors above. The main considerations in the effective use of fire protective suspended ceiling systems are that the tiles or boards do not become dislodged in a fire, and that the suspension hangars do not lose strength. To overcome the possible disruptive effect of the thermal expansion, 'cut-outs' are introduced in the ceiling grid. Ventilation ducts and other openings should not impair the integrity of the ceiling. The design of suspended ceilings is covered in CP290[39]. 4.12 4.12.2.1

12.6 Bracing

Bracing provides the stability of simple frames and should be fire protected accordingly as in Section 6.4.3. An example of fire protection to bracing members is shown in Figure 12.3. If possible, bracing should be built into fire resisting walls so that the bracing needs no applied fire protection. Similarly bracing within fire protected stairways needs no additional protection (other than that provided around the stairway). 3.4

The lateral loading on low rise structures in a fire is relatively small and indeed, in low rise structures, wind forces in fire conditions are ignored. In other cases it is possible to develop sway resistance through the continuity of nominally simple connections. Other alternative load paths may be devised when considering the stability of the structure.

Figure 12.3 *Fire protection to bracing members*

12.7 External steelwork

The Code does not give specific guidance on the design of steelwork external to the building. This is covered in an SCI publication[40] which is referred to directly. In principle, the method may be used to calculate the temperature in the external steel members, taking account of the flame plume through any openings in the external walls. The effect of wind on the direction of the flame is to be considered. 4.8

12.8 Escape stairways

Escape stairways should be part of a fire-protected envelope as they are needed for safe evacuation of the building and subsequent fire-fighting. Steelwork contained completely within the fire-protected envelope does not need additional fire protection.

13 NATURAL FIRES

'Natural fires' are those where a fire builds up and decays in accordance with the mass and energy balance within a compartment. They are significantly different in behaviour from 'standard fires' where gas is the fuel and where temperatures increase at a gradually decreasing rate. The prediction of the temperatures that are reached in natural fires has been the subject of considerable research. The CIB (Conseil International du Batiment) have developed predictive methods by which the severity of natural fires can be related to the standard fire.

Consideration of the effect of natural fires can result in considerable benefit, particularly in buildings where the amount of combustible contents are small. Examples are buildings of large volume, such as sports halls, some retail premises, car parks and rail or air terminals. Some relaxation in the regulations for fire resistance is often possible, particularly where a 'trade-off' between 'active' and 'passive' protection measures is envisaged.

Although calculation procedures for natural fires are not included in *BS 5950: Part 8,* the limiting temperature method is applicable to any fire. Therefore, there may be circumstances where it is possible to compute the rise in temperature of the steel section in natural fires and then to follow the approach in Section 6 to establish its load carrying capacity. A review of fire engineering methods is given by Kirby[41].

13.1 Important parameters in determining fire temperatures

13.1.1 Rate of burning

Temperatures in natural fires as compartments are dependent on: the fire load, the ventilation conditions and other properties such as the thermal conductivity of the compartment walls and roof. The 'fire load' should take into account the calorific value of all the combustible components and is usually expressed in terms of kg of wood-equivalent per unit area of the floor.

After 'flash-over', or rapid expansion of the fire from its source (after 2 to 5 minutes), the rate or burning of the fuel (i.e. wood) depends on whether the fire is 'fuel' or 'ventilation' controlled. If sufficient air is available, the rate of burning is influenced by the characteristics of the fuel. For 'fuel-controlled' fires, which in fire tests are characterized by wooden slats or cribs (normally 40 mm square at 40 mm spacing), the rate of burning is almost a linear function of fire load. A typical rate of burning would be 50–60 kg/m^2/hour for a fire load of 20 kg/m^2. This suggests that a fire of this intensity would consume most of the fuel in roughly 30 minutes.

If there is insufficient air brought into the compartment through the openings to maintain the combustion process, the rate of burning will reduce. This is known as 'ventilation controlled' burning. The transition between these two regimes is shown in Figure 13.1.

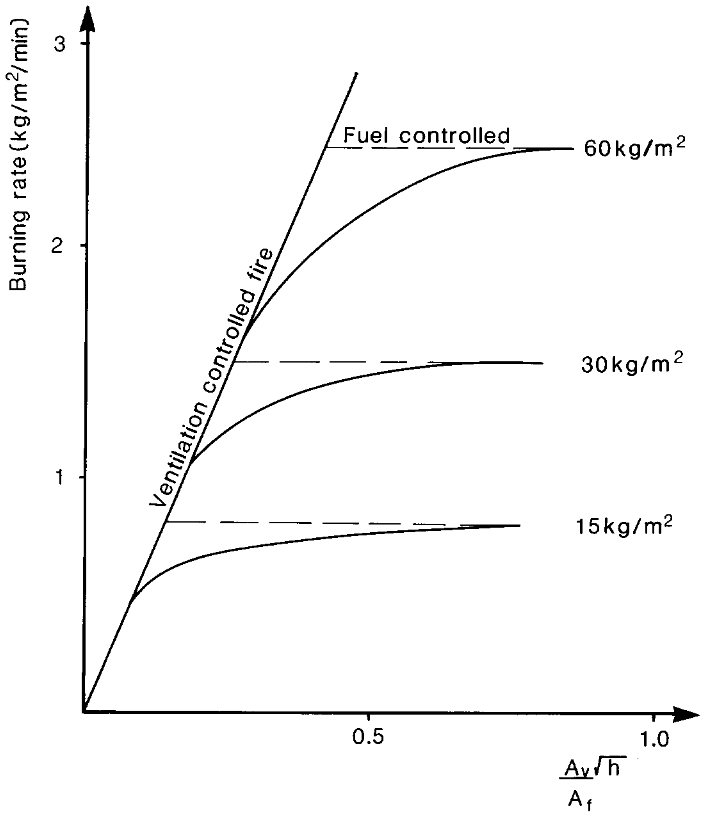


Figure 13.1 *Rate of burning in ventilation and fuel controlled fires*

Typically, 5 kg of air is required for the combustion of 1 kg of wood, whereas 13 kg of air is required for 1 kg of polystyrene. Other data is presented in Table 13.1.

Table 13.1 *Heat output and air demand of some common materials and gases*

Material	Heat output kWh per: kg (material)	Heat output kWh per: kg (air)	Air demand: kg (air) / kg (material)
Wood	4.6	0.93	5.0
Polystyrene	11.6	0.88	13.1
Benzene	11.6	0.88	13.1
Methane	15.1	0.90	16.2
Hydrogen	33.6	0.98	34.5

For wood fires the maximum rate of ventilation-controlled burning is given by the empirical formula:

$$R = 0.09\, A_v \sqrt{h} \ \text{kg/sec} \qquad (39)$$

where A_v = total ventilation area (m^2)
h = weighted mean height of the openings (m)

The height h is important because of considerations of gas flow into and out of the compartment. The hot gases are expended through the upper part of the opening, and the air necessary to maintain the fire is drawn in through the lower part of the opening.

In ventilation-controlled fires the rate of burning of the combustible contents is approximately constant during a period in which the total weight of fuel reduces from 80% to 30% of its original value. The maximum temperature is experienced when around 50% of the fire load is consumed. For fuel-controlled fires the behaviour is similar, but the maximum temperature occurs when 60 to 70% of the fire load is consumed.

13.1.2 Fires in small compartments

In a small compartment the temperature conditions at a given time are considered to be uniform. Temperatures are determined from the energy and mass balance equations of the form:

(a) Heat increase of air in the compartment = Heat produced (by fuel burning) − Heat lost (through walls, openings etc.)

(b) Mass of air leaving the compartment (through openings) = Mass of air drawn in + Mass of fuel consumed.

Peak temperatures in a compartment are usually experienced when there is just sufficient air for fuel controlled burning, but the size of the openings does not permit excessive heat loss. Openings in the roofs of compartments have a much greater effect than vertical openings (of the same size) in allowing the hot gases to escape. It is well known that roof-venting is a good method of reducing peak temperatures in fires. Various computer programs have been developed for predicting the temperature-time relationship in natural fires in small compartments.

A recent series of fire tests was carried out by British Steel and the Fire Research Station[42]. 21 tests were carried out in a brick and concrete compartment of 9.7 m length, 6.85 m width and 3.9 m height using a range of fire loads up to 20 kg/m^2 and a variable width facade opening. An example of a typical temperature-time curve is shown in Figure 13.2. The case that most closely matches the early stages of a standard fire test is a fire load of 15 kg/m^2 and a single opening of area equivalent to one quarter of one facade. This is shown as the curve identified by: 15(¼). The rates of burning were measured

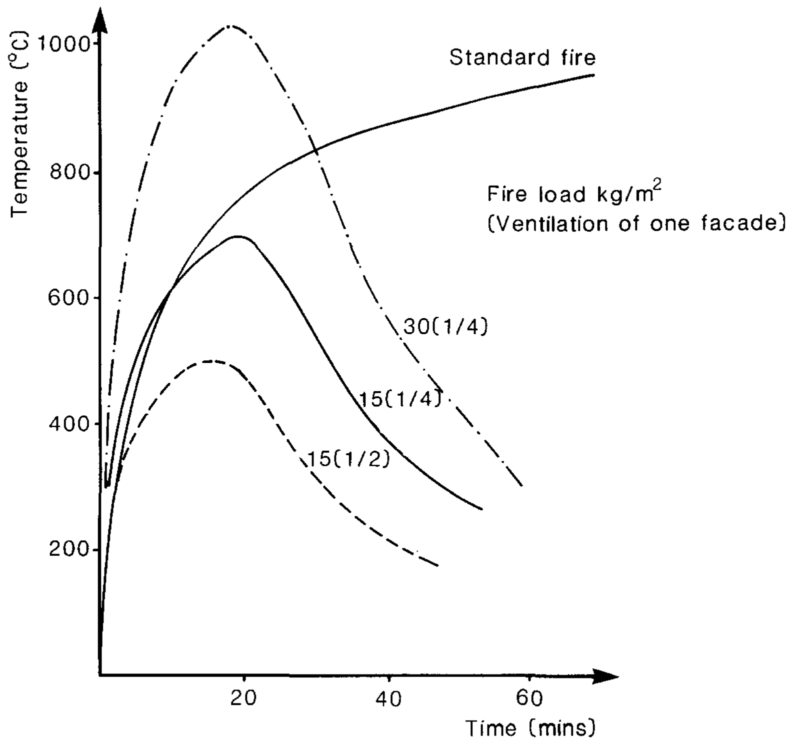


Figure 13.2 *Temperature-time curves for standard and natural fires in small compartments*

and agreed well with the theoretical values. All but one test was fuel-controlled.

13.1.3 Fires in large compartments

Fires in large compartments do not generally engulf the whole compartment. The same heat and mass balance equations apply as for a small compartment but it would be necessary to include the air and heat exchange between the fire volume (plume) and the remainder of the compartment. The fire plume (considered to be an inverted truncated cone in shape) over the area of burning fuel, expands in size according to an empirical formula. Air is drawn into the burning fuel to be replaced by air drawn in from the openings.

The above approach is a two zone model and is illustrated in Figure 13.3.

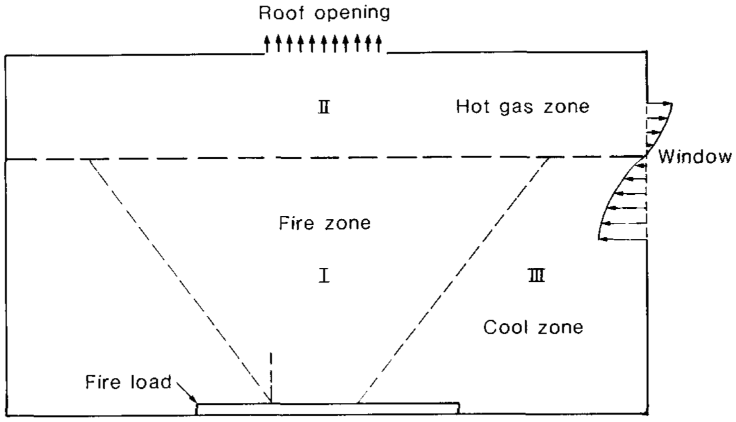


Figure 13.3 *Simple zone model for fire in large compartment*

More sophisticated zone models than this two zone model have been developed, but these are not yet practical methods. The single zone (small compartment) model would over-predict the temperatures experienced in a large compartment.

The spread of fire from its source is normally based on empirical observations in real fires. A doubling of the fire area in a given time is one approach (i.e. exponential increase), but a linear increase of 2 to 3 m/s has also been used.

Fire tests on large compartments are few in number. It is known that a fire test (as yet unreported) on a composite steel framed building was carried out by the American Iron and Steel Institute. Various tests on open and closed car park structures have also been carried out in the UK, USA and Australia[43,44] to demonstrate that relatively low intensity fires are experienced in such buildings, leading to a relaxation in the regulation of requirements for fire protection of this form of structure.

The most important series of fire tests[45] has been carried out recently by the Centre Technique Industriel de la Construction Metallique (CTICM) on a disused electricity generating hall in Paris (La Villette). In this series of tests, fires were propogated in a small area (10% to 25%) of a 28 m × 39 m bay of the building. Various steel sections were suspended over the centre of the fire. Adequate air was available for the combustion process. The temperatures recorded near the roof of the building (at 9 m height) in three of the tests are shown in Figure 13.4. In test 4, polystyrene replaced 25% of the calorific value of the wood. The effect of this was to dramatically increase the temperatures developed.

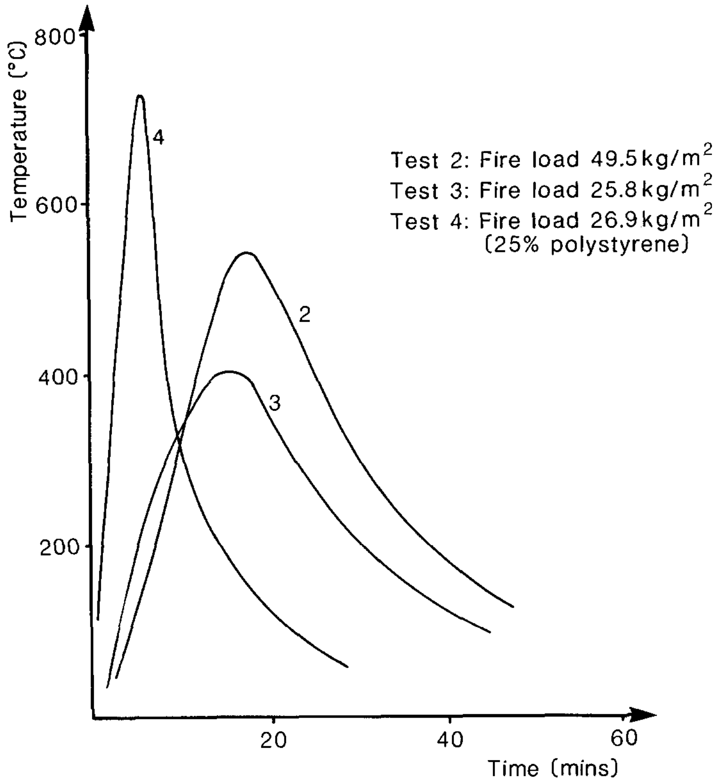


Figure 13.4 *Comparison of temperatures at 9 m above floor in large compartment fire test*

13.2 Concept of time-equivalent

The concept of 'time-equivalent' has been widely used as means of measuring the intensity of a natural fire in relation to a standard fire. In principle, the greater the time equivalent the greater the severity of the fire, and hence the greater the required fire resistance of the structure. The definition of the time equivalent is that time in a standard fire corresponding to the same temperature experienced in a structural member in both the standard fire and at the peak of the natural fire.

Consider the rise in temperature of a steel section, as in Figure 13.5. This is

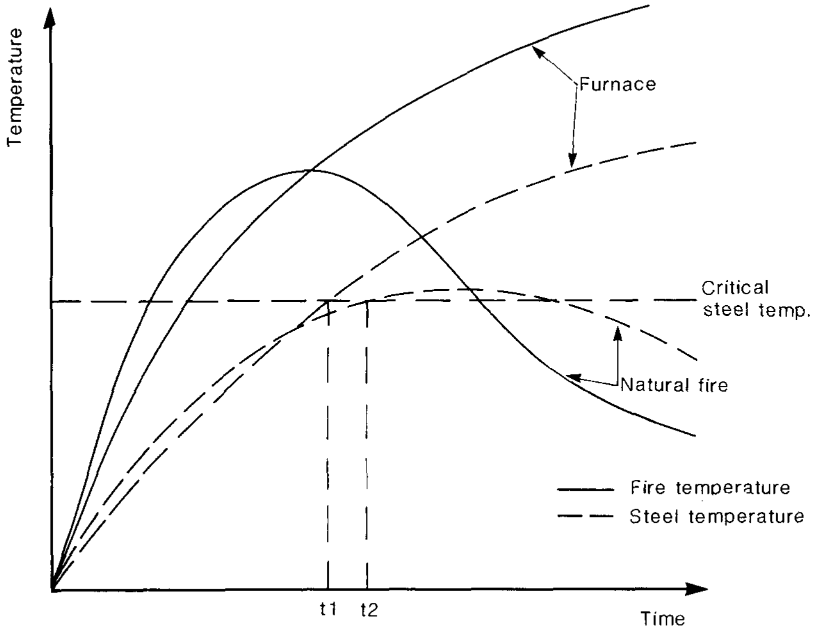


Figure 13.5 *Idealized behaviour of steel sections in natural and standard fires*

also a function of the area under the temperature time curve. The limiting steel temperature might be reached at time t_2 in a natural fire and t_1 in a standard fire (see Section 13.4).If a protected section just reaches a peak temperature of 550°C in the natural fire then the time-equivalent is given by t_1. Repeating this for a range of natural fires and different protected sections leads to a family of time equivalents as a function of the fire load and other parameters.

From an extensive series of fire tests in small compartments carried out worldwide, an empirical formula for the time equivalent t_e has been proposed. This was first established by considering the equivalent degradation of concrete elements in natural and standard fires. It includes the term $A_v\sqrt{h}$ which broadly defines the rate of burning in ventilation controlled fires. The formula is[6]:

$$t_e = \frac{0.067\ q_f\ A_f}{A_t(A_v\sqrt{h}/A_t)^{0.5}} \text{ minutes} \qquad (40)$$

where q_f = the fire load expressed as MJoules/m^2 of floor area (18 MJ = 1 kg wood),
A_f = floor area
A_t = total surface area of the compartment.

In many references q_f is expressed in terms of the total surface area, rather than the floor area. This formula applies to enclosures of brick or concrete.

Equation (40) has been subsequently revised further to give:

$$t_e = c.w.\ q_f \quad \text{minutes} \qquad (41)$$

where ω = ventilation factor
c = is a factor accounting for the properties of the walls.

For large openings (greater than 10% of the floor area), the fire is fuel-controlled and ω may be taken as 1.5. c may be conservatively taken as 0.09 for highly insulating materials reducing to 0.05 for less insulating materials.

Using either of these equations, it is possible to evaluate the fire resistance required for a particular compartment within a building. Such an approach demands that the building should be unchanged during its life, particularly in terms of fire load and ventilation area.

13.3 Fire loads in buildings

The main problem relating to the use of the time-equivalent method is the selection of the fire load or fire load density that should be adopted. A number of surveys of fire loads have been conducted worldwide. Typically, the data have been presented in terms of mean 'fire loads' and 80% fractile values i.e. 80% of rooms within the various occupancy groups have fire loads less than the values quoted.

All values are expressed in terms of MJoules/unit floor area. This can be related to kg of wood-equivalent (18 MJ = 1 kg wood). The fire load is determined by summing the mass of each object in the compartment times the calorific value of the materials. The fire loads typical of the following types of buildings are:

Hospitals: Studies have shown that the 80% fractile fire load density is around 400 MJ/m^2, but it should be noted that in certain rooms, such as stores and laundries mean values approaching 2000 MJ/m^2 can be encountered.

Hotels: The 80% fractile value is typically 400 MJ/m^2. The same comments, as above, apply to store rooms and linen cupboards.

Offices: Values of average fire load density vary depending on the function of the office, varying from less than 100 MJ/m^2 in the lobby to 1500 MJ/m^2 in file storage rooms. 80% fractile values are between 520 to 720 MJ/m^2 for offices of normal usage.

Schools: Studies have shown that the 80% fractile values vary according to the age-group of the students. The range is from 270 to 420 MJ/m^2.

Department stores: The 80% fractile value is typically 600 MJ/m^2, although this depends on the materials sold, and may increase in storage areas.

Restaurants and theatres: The 80% fractile value is typically 450 MJ/m^2.

Further work is needed to verify these figures. If a fire load of 450 MJ/m^2 is inserted into Equation (41) (with $c = 0.09$; and $\omega = 1.5$), the time-equivalent is approximately 60 minutes.

13.4 Temperatures in steel sections in natural fires

The temperatures in uniformly-heated steel sections θ_s may be determined as in Equation (11) in Section 4.3 for protected members or Equation (8) in Section 4.2 for unprotected members. Examples of uniformly heated members are columns, or beams not supporting concrete floors. These equations are based on one-dimensional heat flow into the section. More sophisticated finite element methods are required to determine the heat-flow in more complex sections comprising different materials (see Section 4.6.).

The furnace or natural fire temperatures θ_f may be input in Equations (8) and (11). In a standard fire, the steel temperature rise follows the same form of curve as the fire temperature, the temperature difference between the furnace and the steel temperature being principally dependent on the section factor H_p/A and the thermal resistivity (inverse of conductivity) of the protective material.

In a natural fire, the steel temperature rises and decays but the maximum steel temperature does not occur at the maximum fire temperature. Rather it occurs on the declining portion of the temperature curve shortly after the point at which the fire and steel temperatures are equal. This is because when θ_f exceeds θ_s heat enters the section and when θ_f exceeds θ_s, heat leaves the section. The slight delay is because of the storage of heat in the fire-protective material. This behaviour is illustrated in Figure 13.5.

The effect of the fire protection is both to slow down the rate of temperature rise and to 'smooth-out' any early peaks in the temperature-time history. In test 4[46] of Figure 13.4 polystyrene as fuel caused a very rapid rise of temperature early in the fire but this also decayed quickly. However, steel temperatures in a well-protected member were not significantly different from the case where wood was the only fuel (for the same fire load density).

13.5 Influence of active protection measures

Passive protection refers to the fire resistance of members, whereas active protection measures are those that contribute to a reduction in the severity of a fire. The most commonly-used active system is sprinklers. These are characterized in terms of their speed of reaction, rate of wetting, and total discharge capacity.

There is a body of statistical evidence to indicate that sprinklered buildings have suffered less fire damage than non-sprinklered buildings[41,46]. For steel buildings, such active systems are attractive because some trade-off between active and passive measures could result in the elimination of passive fire protection to the steelwork in many categories of buildings of low fire load. A secondary benefit is the increase in permissible compartment areas and volumes.

There are now many examples of buildings in the UK and Europe where such appraisal methods have been used. Examples in the UK are: Gatwick airport terminal, Ibrox and Murrayfield stadia, and the National Exhibition Centre. A major example in Europe is the Palais des Expositions in Geneva where sprinklers in the roof zone were designed primarily to cool the steelwork in the event of a fire.

The Swiss(47) were the first nation to introduce a risk evaluation matrix for buildings in fire (known as the Gretener system). This is administered by a monopoly-insurance bureau who offer building-structure insurance on a national basis. The method is based on a points system that heavily reflects the benefits of active measures. A similar calculation method is now used for industrial buildings in West Germany. This is based on statistical evidence of fire severity in sprinklered buildings.

In principle, sprinklers, detection and other active measures are designed to prevent small fires from becoming large ones. There is some small risk (around 2% commonly quoted) of the systems mal-functioning or being inoperational (because of poor maintenance). The benefits of sprinklers are covered in a report by Stirland.(46)

It is possible to modify the deterministic approach considered earlier to include the effect of sprinklers in the energy balance. The heat required to vaporize the discharged water is partly removed from the fire, thereby reducing the peak temperatures experienced. Even if the sprinklers were only operational for a short duration (say 10 minutes) this would be well within normal fire-brigade response times and would reduce the potential damage. One possible approach is to reduce the equivalent fire resistance (or time equivalent) by up to 50% based on equation (41) when sprinklers are introduced.

14 RE-USE OF STEEL AFTER A FIRE

Fire is a rare occurrence and it would normally not be economically viable to design for damage limitation and repairability after a severe fire. Indeed, regulations and fire tests are not concerned with re-use of the structure, provided it retains its load carrying capacity for the appropriate period of fire resistance. Appendix C

The potential for re-use of structural steel after a moderate to severe fire is a matter of engineering judgement. It may be expected that unprotected elements have distorted, and that the fire protection to some elements has become detached on cooling. Spalling of concrete encasement may have exposed the steel section.

Outside the zone of the building subject to the most severe part of the fire, there may be secondary damage because of thermal expansion of the floors or thermal bowing towards the fire. The criteria for re-use are the residual strength of the materials, and the distortion of the members. This second aspect is more important for members subject to compression e.g. columns or struts.

14.1 Mechanical properties

Data on the mechanical properties of steel and other materials after being subject to elevated temperatures for up to 4 hours is presented by Kirby Lapwood and Thomson(16). The following sections summarize their observations and more recent research findings.

14.1.1 Structural steel

The mechanical strength properties of grade 43 steel are unaffected following a fire if temperatures did not exceed 600°C. However, exposure of grade 43 steel to temperatures above 600°C may result in a reduction in strength of up to 10% below the minimum specified values. The mechanical strength properties of grade 50 steels are also unaffected for temperatures below 600°C. Exposure to higher temperatures may result in greater strength reductions than in grade 43 steel. Appendix C2

The notch toughness of structural steels is unaffected by temperatures up to 600°C. Above 600°C there are temperature regimes which can result in the properties falling to below the minimum requirements.

14.1.2 Cold formed steel

Cold formed members are likely to have distorted and suffered high temperatures even in a moderate fire making them unsuitable for re-use. As a simple rule, the residual strength of cold formed steel members up to grade Z35 may be taken as 90% of the specified strength, reflecting the loss of the beneficial effects of cold-working.

14.1.3 Cast and wrought iron

The residual strength of cast iron is similar to steel up to 600°C except that above 600°C a more severe strength reduction can be expected. However, due to the massivity of cast iron members and their low design stresses, they often perform better than steel under the same heating conditions.

The available evidence indicates that fire does not permanently affect the strength properties of wrought iron and if anything, a small improvement may occur.

14.1.4 Bolts and welds

Appendix C3

The tensile strength reduction for grade 4.6 bolts follows that of grade 43 steel. For grade 8.8 bolts, the residual strength falls more markedly after exposure to temperatures exceeding 450°C. The residual strength after exposure to higher temperatures reduces to about 80% at temperatures of 600°C and 60% at 800°C. High strength friction grip bolts behave in a similar manner.

The residual strength of welds is not significantly affected by the exposure to high temperatures.

14.2 Inspection and appraisal

The initial inspection and assessment of any fire damaged structure is visual. This is followed by measurements of the out-of-straightness of linear elements, particularly those that may be subject to compression. Finally, it may be necessary to carry out hardness tests on certain sections of steel to gain a measure of the residual strength of the steel.

Firstly, the visual assessment would include signs of gross deformation, particularly of the connections and welds. Distress of non-structural elements is often caused in thermal expansion and bowing. Out of straightness or non-verticality of columns will reduce their load carrying capacity, but this is less important for beams provided the compression flanges are laterally restrained.

Secondly, hardness tests using for example a portable Brinell Hardness Tester, can be used to provide an indication of the ultimate strength of the steel and whether it is likely to meet the appropriate specification.

Comparisons may be made between fire damaged and only slightly affected or non-damaged members. Variations of 10% in Brinell Hardness Number can be regarded as significant. Where changes have occurred and the corresponding values of ultimate tensile strengths are below 5% of the minimum requirements for Grade 43 steel, or 10% for Grade 50 steels, the mechanical strength properties of the steel may not meet the appropriate specification. In such cases, it may be necessary to seek advice from a metallurgist and to check the tensile properties of the steel by removing a coupon from a heat affected member and subjecting it to a tensile test (to *BS 18*).

14.3 Re-use of steel structures

14.3.1 Unprotected sections

Members that have not distorted beyond the out-of-straightness limits in *BS 5950: Part 2* can be re-used without further checking provided the connections have not suffered damage. Members that have distorted slightly may be re-used provided they are laterally restrained or the stresses to which they are subjected are small. This is a matter of a further structural assessment. It may be necessary to replace bolts if they have been subject to high temperature(16).

14.3.2 Protected sections

Fire protection generally takes the form of spray or box protection or concrete encasement. Spray protection can be re-sprayed having removed any damaged or friable material. However, some materials undergo chemical changes at 200–300°C and advice should be sought about their removal. Gaps may have developed between the individual panels of box protection as a result of the distortion of the structure, and it would be necessary to replace the affected panels.

Concrete is likely to have spalled from the steel and it will normally be necessary to replace the cover of the steel by a cement rich mortar or sprayed concrete (gunite) having broken back the damaged or cracked concrete.

14.3.3 Composite deck slabs

Some permanent deflection of all concrete or composite deck slabs would be experienced after a severe fire(29). In composite deck slabs it would normally be necessary to cut-away any sections of decking that have lost contact or debonded from the slab, and replace it by reinforcing bars attached by shot-fired clips, between the ribs. The bars could then be 'gunited' in situ to achieve the appropriate bond. In severely damaged slab, replacement of the complete section of slab between the steel beams should be considered.

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Appendix DESIGN EXAMPLES

These Design Examples cover the following cases:

1. **Simple beams**
2. **Columns in simple construction**
3. **Shelf angle floor beams**
4. **Concrete filled rectangular hollow sections**

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CALCULATION SHEET

Job No. PUB 7800		Sheet 1 of 6	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 1, Simple Beam			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

The following calculations are used to illustrate the application of the code. This covers:

(1) Methods of determining the thickness of fire protection.
(2) Load ratio method as applied to columns in simple construction.
(3) Moment capacity method as applied to shelf angle floor beam.
(4) Load ratio method as applied to concrete filled hollow section.

METHODS OF CHECKING A SIMPLE BEAM

Various aspects of the fire resistance of a simple beam are demonstrated:

(a) Section factor H_p/A
(b) Performance without protection.
(c) Performance with fire protection.

Design parameters

Beam	406 × 178 × 54 UB	
Grade	50B	
Design strength, p_y	355 N/mm^2	
Span	6 m	
Loading:	Distributed load over a beam spacing of 3.9 m	
	Imposed	5 kN/m^2
	Dead	3.5 kN/m^2
	Ceiling and Services	1.5 kN/m^2

The beam is assumed to be laterally restrained by the floor.

Section factor (Section 4.1, Code 4.2)

$$\text{Thermal response factor (section factor)} = \frac{H_p}{A} \text{ metres}^{-1}$$

H_p = Heated perimeter (m)
A = Cross section area (m^2)

In calculating the heated perimeter of the steel section the upper surface may be ignored because it is protected by the precast concrete. It is normal to ignore the effect of fillet radii in the steel section.

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CALCULATION SHEET

Job No. PUB 7800		Sheet 2 of 6	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 1, Simple Beam			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

For sprayed protection which follows the profile of the section.

$H_p = 2D + 3B - 2t$

$= 1322.8$ mm

$= 1.3228$ m

$A = 68.4$ cm^2 (from tables)

$= 0.00684$ m^2

$$\frac{H_p}{A} = \frac{1.3228}{0.00684}$$

$= 193.4$ m^{-1}

If the radii are taken into account this value would be slightly reduced. Expressed to the nearest 5 (which is often done in published tables), Reference 2 quotes a value of 190 m^{-1}.

For board protection:

$H_p = 2D + B$

$= 982.8$ mm

$= 0.9828$ m

$A = 0.00684$ m^2

$$\frac{H_p}{A} = 143.7 \text{ m}^{-1}$$

This would normally be expressed as 145 m^{-1}.

FIRE RESISTANCE WITHOUT APPLIED PROTECTION

Beams are assessed on their load carrying capacity only. Integrity and insulation are not normally considered. The criterion for load carrying capacity for simple beams, is that at the end of the fire resistance period the design temperature does not exceed the limiting temperature given in Code Table 5. For a beam the temperature of the bottom flange is considered to represent the limiting temperature.

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CALCULATION SHEET

Job No. PUB 7800		Sheet 3 of 6		Rev.
Job Title Handbook to BS5950: Part 8				
Subject Design Example 1, Simple Beam				
Client SCI	Made by GMN		Date 30 Mar 90	
	Checked by RML		Date 18 Apr 90	

In practice an unprotected beam will rarely have more than 30 minutes fire resistance, so 30 minutes will be considered.

For a flange thickness of 10.9 mm the design temperature is 767°C. (Code Table 7).

The load ratio, R, at the fire limit state is to be calculated from:

$$R = \frac{\text{Moment at fire limit state}}{\text{Moment capacity at } 20°\text{C}}$$

The moment at fire limit state, M_f is obtained from:

Load factors:	Permanent	1.0	(Code Table 2)
	Non-Permanent	0.8	

Assume that 4 kN/m² of the imposed load is non-permanent.

$$M_f = \frac{L^2 b}{8} \times (1.0 \times \text{Permanent Load} + 0.8 \times \text{Non-Permanent Load})$$

$$= \frac{6^2 \times 3.9}{8} \times (1.0 \times 6 + 0.8 \times 4)$$

$$= 161.5 \text{ kNm}$$

As the section is compact the moment capacity is equal to the plastic moment capacity.

$$M_c = p_y S$$
$$= 355 \times 744 \times 10^{-3}$$
$$= 274.8 \text{ kNm}$$

$$R = \frac{161.5}{274.8}$$

$$= 0.587$$

The limiting temperature for unprotected members is 624°C (by linear interpolation, from Code Table 5). This is less than the design temperature of 767°C so the unprotected beam will have less than 30 minutes fire resistance.

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CALCULATION SHEET

Job No. PUB 7800	Sheet 4 of 6	Rev.	
Job Title Handbook to BS5950: Part 8			
Subject Design Example 1, Simple Beam			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

FIRE RESISTANCE WITH APPLIED PROTECTION

(a) Using published data (Reference 2, Code 4.3)

When using proprietary data the restrictions on the use of any product should always be noted. These include the use of paint primers and exposure to weather.

(i) Spray applied protection
Section factor, H_p/A, 3 sided profile = 190 m^{-1}
For an H_p/A of 190 m^{-1} the following data can be obtained for a typical sprayed fire protection material.

Fire resistance	1 hr	1½ hr	2 hr	3 hr	4 hr
Protection thickness	17 mm	24 mm	32 mm	46 mm	–

These are based on a limiting temperature of 550°C.

Considering the actual load on the beam, a limiting temperature of 624°C can be used to assess the fire protection thickness. The protection manufacturer should be consulted for this information, or if the thermal properties of the insulation are available, method (b) below can be used.

(ii) Use of board protection
Section factor, H_p/A, 3 sided box = 145 m^{-1}
For an H_p/A of 145 m^{-1} the following data can be obtained for a typical box fire protection.

Fire resistance	½hr.	1hr	1½hr	2hr	3hr	4hr
Protection thickness	20mm	20mm	25mm	35mm	50mm	70mm

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	Job Title Handbook to BS5950: Part 8		
	Subject Design Example 1, Simple Beam		
	Client SCI	Made by GMN	Date 30 Mar 90
		Checked by RML	Date 18 Apr 90

(6) Calculation of protection thickness for sprayed protection (Section 8.3, Code Appendix D)

$$t = k_i \, I_f \, F_\omega \left(\frac{H_p}{A}\right) \times 10^{-6}$$

where t = protection thickness (m)

k_i = function of thermal properties of insulation (Wm^{-1})

I_f = fire protection material insulation factor (m^3kW^{-1})

F_ω = fire protection material density factor

Assume a spray protection

k_i = 0.17 Wm^{-1} (obtained from manufacturer)

I_f = 1350 m^3kW^{-1}, assuming a limiting temperature of 624°C and 2 hours fire resistance. (Code Table 16)

$$F_\omega = \frac{(1 + 4\mu)^{1/2} - 1}{2\mu}$$

$$\text{where } \mu = \frac{k_i \, \rho_i \, (1 + 0.03c)}{\rho_s} \times \frac{I_f}{10^6} \times \left(\frac{H_p}{A}\right)^2$$

where ρ_i = density of protection material (kg/m^3)

= 400 kg/m^2 (obtained from manufacturer)

c = moisture content of protection material (wt %)

= 5% (obtained from manufacturer)

ρ_s = density of steel (kg/m^3)

= 7850 kg/m^3

This gives for the above data:

μ = 0.485

F_ω = 0.736 (F_ω could have been obtained from Code Table 17).

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CALCULATION SHEET

Job No. PUB 7800	Sheet 6 of 6	Rev.
Job Title Handbook to BS5950: Part 8		
Subject Design Example 1, Simple Beam		
Client SCI	Made by GMN	Date 30 Mar 90
	Checked by RML	Date 18 Apr 90

$$\therefore \quad t = 0.17 \times 1350 \times 0.736 \times 190 \times 10^{-6}$$
$$= 0.0321 \text{ m}$$
$$= 32.1 \text{ mm}$$

This calculated thickness must be checked to see if it complies with any restrictions that the manufacturer or material assessor may impose on its use. For example, it may require wire reinforcement, or it may exceed the maximum allowable thickness.

The above figure is not significantly different from the tabulated value. However, it would be possible in using this method to modify the thickness of fire protection with respect to limiting temperature, by considering the variation of the term l_f.

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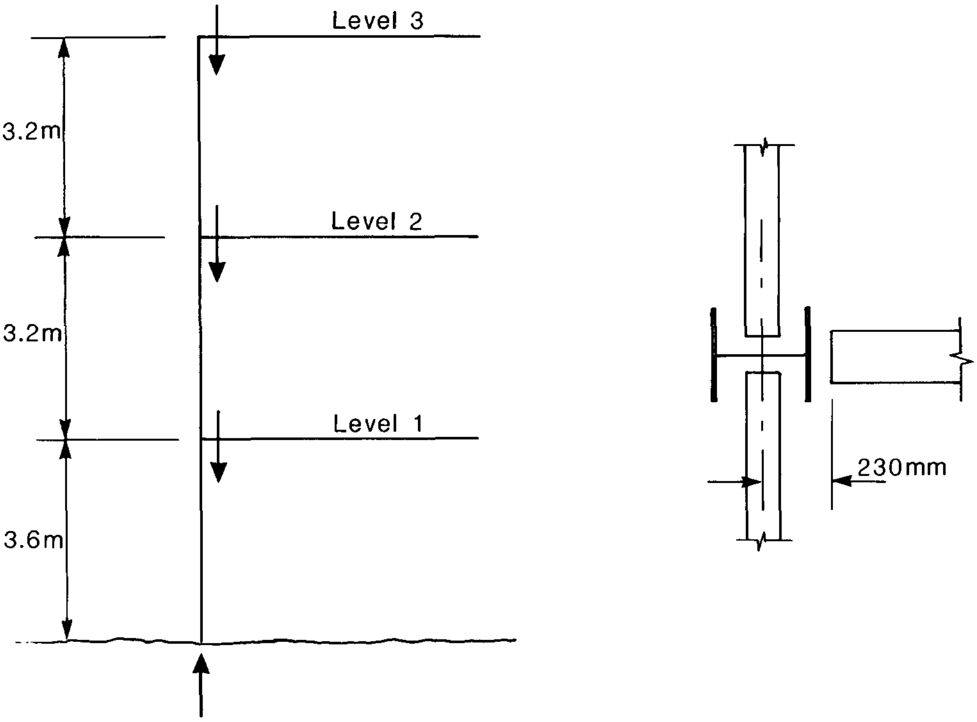
CALCULATION SHEET

Job No. PUB 7800		Sheet 1 of 3	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 2, Column			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

COLUMN IN SIMPLE CONSTRUCTION

The limiting temperature method is used to illustrate the effect of load ratio on a three storey column in simple construction.



Elevation

Plan

Design parameters

Column	254 × 254 × 89 UC
Design strength, p_y	275 N/mm^2
Floor loading:	
Imposed	5 kN/m^2
Dead	3.5 kN/m^2
Ceiling and Services	1.5 kN/m^2

For simplicity, assume that a column receives load from two edge beams each equivalent to 6 m^2 of floor at each level and receives load from an internal beam equivalent to 24 m^2 of floor at each level.

The Steel Construction Institute	Job No. PUB 7800	Sheet 2 of 3	Rev.
Silwood Park Ascot Berks SL5 7QN Telephone: (0990) 23345 Fax: (0990) 22944 Telex: 846843	Job Title Handbook to BS5950: Part 8		
	Subject Design Example 2, Column		
CALCULATION SHEET	Client SCI	Made by GMN	Date 30 Mar 90
		Checked by RML	Date 18 Apr 90

For a column in simple construction the load ratio is given by:

$$R = \frac{F}{A_g\, p_c} + \frac{M_x}{M_b} + \frac{M_y}{p_y Z} \qquad \text{(Code 4.4.2.3)}$$

F, M_x and M_y are to be calculated at the fire limit state.

Load factors: Permanent 1.0 (Code Table 2)
Non-Permanent 0.8

Assume that the whole of the imposed load is non-permanent.
The loads are therefore:

		Unfactored (kN)	Factored (kN)
Dead	Edge beam	60	60
	Internal beam	120	120
Imposed	Edge beam	60	48
	Internal beam	120	96

	Dead Loads				Imposed Loads				Totals	
Level	Edge	Int	Total	M_x	Edge	Int	Total	M_x	$\sum F$	$\sum M_x$
3	60	120	240	28	48	96	192	22	432	50
2	60	120	480	14	48	96	$192 \times 2 \times 0.9 = 346$	11	826	25
1	60	120	720	14	48	96	$192 \times 3 \times 0.8 = 461$	11	1181	25

Notes:

(1) As the loads are balanced about the yy axis $M_x = 0$

(2) M_x moments are based upon a 230 mm offset of the beam from the column centre

(3) The imposed loads at levels 2 and 3 are reduced in accordance with BS6399

(4) Units are kN and kNm

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Job No. PUB 7800		Sheet 3 of 3	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 2, Column			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

From the design of the column under normal loads (in accordance with BS5950: Part 1)

p_c = 215 N/mm^2 for top and middle storeys

p_c = 205 N/mm^2 for bottom storey

M_b = 325 kNm

A_g = 114 cm^2

l/r_y = 49 (Top and middle storeys)

l/r_y = 52 (Bottom storey)

The load ratio can now be calculated for each storey height.

Upper storey:

F = 432 kN $\quad M_x$ = 50 kNm

$$R = \frac{432 \times 10^3}{114 \times 10^2 \times 215} + \frac{50}{325} = 0.33$$

Middle storey:

F = 826 kN $\quad M_x$ = 25 kNm

$$R = \frac{826 \times 10^3}{114 \times 10^2 \times 215} + \frac{25}{325} = 0.41$$

Bottom storey:

F = 1181 kN $\quad M_x$ = 25 kNm

$$R = \frac{1181 \times 10^3}{114 \times 10^2 \times 215} + \frac{25}{325} = 0.58$$

The limiting temperatures for each storey height may be obtained from Code Table 5. The lower slenderness values may be used.

			Limiting temperature
Upper storey	$R = 0.33$	→	643°C
Middle storey	$R = 0.41$	→	611°C
Bottom storey	$R = 0.58$	→	548°C

The fire protection thickness may be varied with respect to these temperatures.

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CALCULATION SHEET

Job No.	PUB 7800	Sheet 1 of 8		Rev.	
Job Title	Handbook to BS5950: Part 8				
Subject	Design Example 3, Shelf Angle				
Client	SCI	Made by GMN		Date	30 Mar 90
		Checked by RML		Date	18 Apr 90

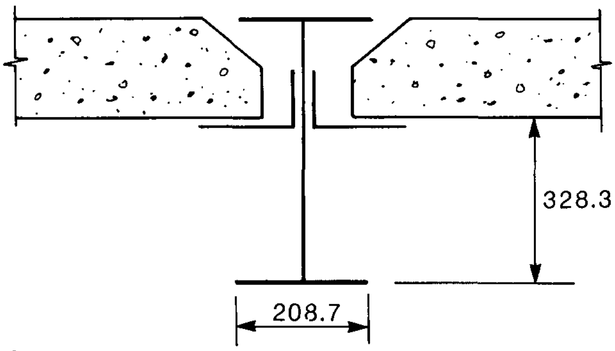
SHELF ANGLE FLOOR BEAM

This example illustrates the use of the moment capacity method (Section 7, Code 4.4.4) to check the fire resistance of a shelf angle floor beam (Section 10, Code Appendix E).

Design parameters

Beam	533 × 210 × 82 UB
Grade	50B
Angle	125 × 75 × 12
Grade	50B
Span	6 m

Span of precast slab	4.5 m
Precast slab	200 mm deep
Screed	40 mm



Loading:

Imposed	$5\ kN/m^2$		
Dead	$5\ kN/m^2$		
Fire resistance	1 hour		
Load factors	Permanent	1.0	
	Non-Permanent	0.8	(Code Table 2)

The maximum beam bending moment and the vertical load per unit length on each angle are calculated as follows:

$$\text{Moment} = \frac{6^2 \times 4.5}{8} (1.0 \times 5 + 0.8 \times 5)$$

$$= 182.3\ kNm$$

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CALCULATION SHEET

Job No. PUB 7800		Sheet 2 of 8	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 3, Shelf Angle			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

Load on leg of angle

$$= 6 \times \frac{4.5}{2} (1.0 \times 5 + 0.8 \times 5)$$

$= 121.5$ kN

$= 20.25$ kN per metre of span

<u>Temperature distribution after 1 hour</u>

(Table 10.2 Code Appendix E3)

Bottom flange

Thickness $= 13.2$ mm

Code, Table 7 $\theta_1 = 936°C$

Check D_e/B_e in Code Table 8

Exposed depth, $D_e = 528.3 - 200$

$= 328.3$ mm

$B_e = 208.7$ mm

$= 1.57$ i.e. > 1.5

Therefore, Table 8 gives no correction to flange temperature

Lower web:

Code, Table 18 $\theta_2 = \theta_1 = 936°C$

Exposed horizontal leg of angle:

Code, Table 18 $\theta_3 = 765°C$

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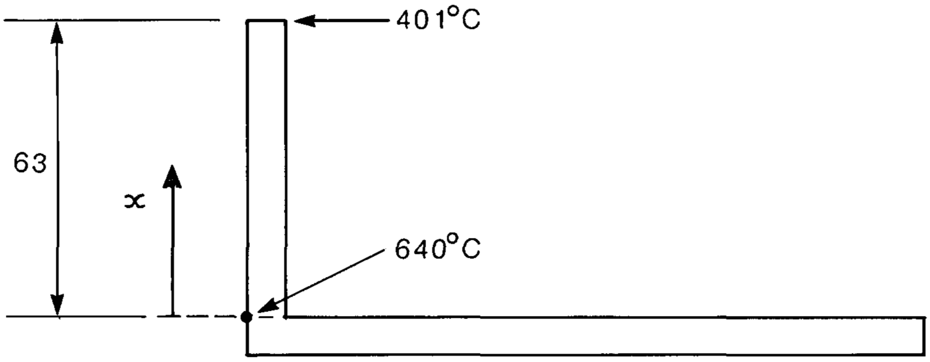
CALCULATION SHEET

Job No. PUB 7800	Sheet 3 of 8		Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 3, Shelf Angle			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

Vertical leg of angle and upper web:



Code, Table 18	θ_{root}	$= 640°C$
Code, Table 19	Gradient	$= 3.8°C$ per mm
Top of angle	θ	$= 640 - 63 \times 3.8$
		$= 401°C$
∴ Average of angle leg	θ_4	$= (640 + 401)/2$
		$= 520°C$

The position of the 300°C boundary is found by linear interpolation from:

$$x = (640 - 300)/3.8$$
$$= 89.5 \text{ mm}$$

The upper web is therefore considered as two elements

Lower part –	Depth	$= 89.5 - 63 \times 26.5 = 26.5$
	θ_5	$= \frac{1}{2}(401 + 300) = 350°C$
Upper part –	Depth	$= 97.3$ mm
	θ_6	$\ngtr 300°C$
Top flange	θ_7	$\ngtr 300°C$

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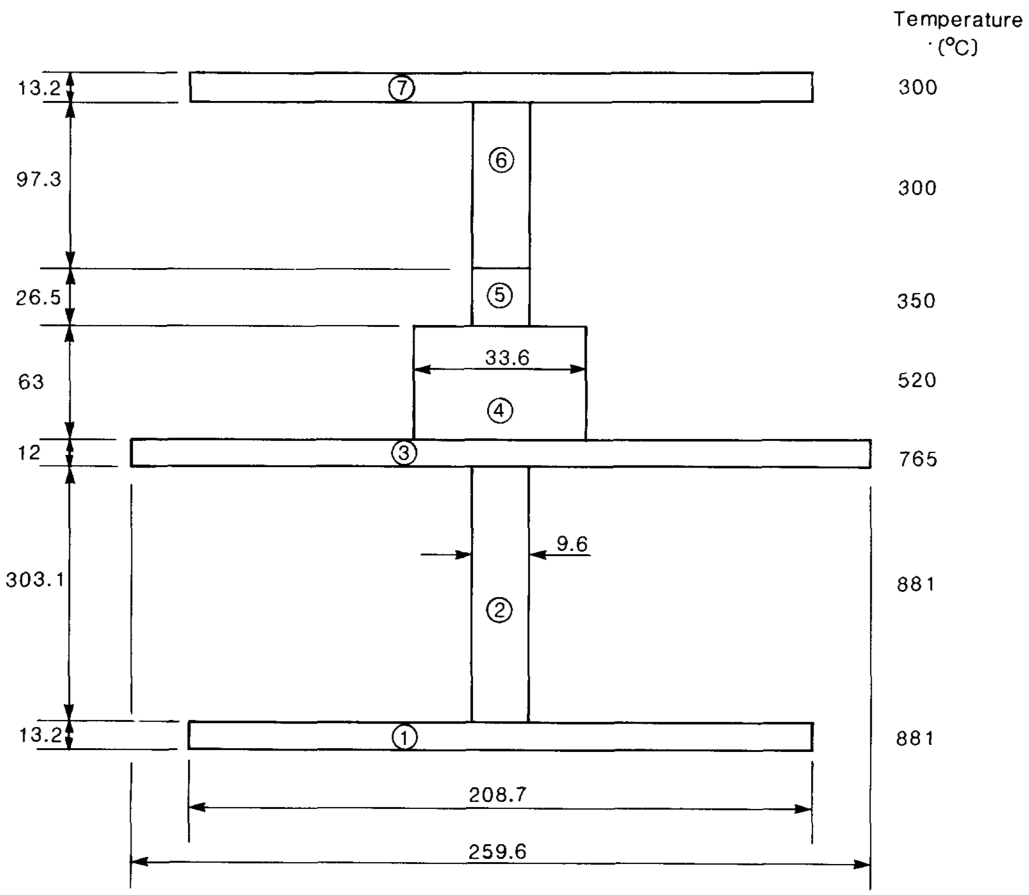
	Job Title Handbook to BS5950: Part 8		
	Subject Design Example 3, Shelf Angle		
	Client SCI	Made by GMN	Date 30 Mar 90
		Checked by RML	Date 18 Apr 90

The beam is divided into temperature zones as follows:



The Code specifies that in calculating the moment capacity of a shelf angle floor the steel strength should be based upon a strain of 1.5% (Code 2.3).

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Job Title Handbook to BS5950: Part 8		
Subject Design Example 3, Shelf Angle		
Client SCI	Made by GMN	Date 30 Mar 90
	Checked by RML	Date 18 Apr 90

For each of the 7 zones the resistance is to be calculated. A tabular method is preferred.

Zone	Width mm	Depth mm	Temp °C	SRF+	Strength N/mm²	Stress N/mm²	Resistance N
1	208.7	13.2	936	0.050	355	17.6	48546
2	9.6	303.1	936	0.050	355	17.6	51546
3	259.6	12.0	765	0.139	355	49.3	153579
4	33.6	63.0	520	0.698	355	247.8	524543
5	9.6	26.5	350	0.968	355	343.6	87412
6	9.6	97.3	300	1.00	355	355	331598
7	208.7	13.2	300	1.00	355	355	977968
							2175192N

+ SRF – Strength reduction factor from Code Table 1 based on 1.5% strain.

The total resistance of the section is thus 2175 kN. In calculating the moment capacity it follows that half this resistance is in tension and half in compression.

Half resistance = 1087.5 kN

Zone 7 and part of Zone 6 are in compression

$$\text{Fraction of Zone 6} = \frac{1087.5 - 978.0}{331.6} \times 97.3$$

$$= 32.2 \text{ mm in compression}$$

The neutral axis is therefore 13.2 + 32.2 = 45.4 mm below the top flange. Zone 6 is to be divided into compression and tension. The centroid of each zone is then calculated and moments of each zone taken about any convenient axis to obtain the moment capacity.

Zone	Resistance N	Tension or Compression	Centroid+ mm	Moment kNm
1	48.5	T	521.7	25.32
2	51.5	T	363.6	18.74
3	153.5	T	206.0	31.64
4	524.5	T	168.5	88.39
5	87.4	T	123.8	10.82
6	222.0	T	77.9	17.30
6	109.6	C	29.3	–3.21
7	978.0	C	6.6	–6.45
				182.5

+ Measured to upper surface of top flange, Zone 7

The moment capacity is 182.5 kNm which is greater than the required moment capacity of 182.3 kNm.

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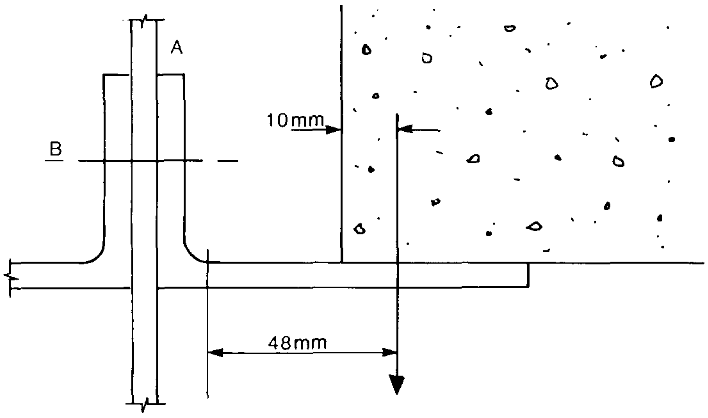
CALCULATION SHEET

Job No. PUB 7800		Sheet 6 of 8		Rev.	
Job Title Handbook to BS5950: Part 8					
Subject Design Example 3, Shelf Angle					
Client SCI	Made by GMN		Date 30 Mar 90		
	Checked by RML		Date 18 Apr 90		

Transverse bending of angle



The load is assumed to act 10 mm in from the edge of the precast unit. The lever arm on the leg of the angle depends on the actual details and 48 mm is taken as a typical dimension.

Angle temperature = 765°C

Strain = 1.5%

Reduction factor = 0.193 (Code, Table 1)

Bending capacity = $1.2 p_y Z$ (Section 10.3.2 Code E1 (f))

$$= 0.193 \times 1.2 \times 355 \times \frac{12^2}{6} \times 1000 \times 10^{-6} \text{ kNm per m}$$

= 1.97 kNm per metre

Applied load = 20.25 kN

∴ Moment = $20.25 \times 48 \times 10^{-3}$

= 0.972 kNm per metre

The angle is therefore satisfactory in transverse bending.

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CALCULATION SHEET

Job No. PUB 7800		Sheet 7 of 8	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 3, Shelf Angle			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

Angle to beam connection

The force carried by each angle will be a proportion of the resistance of Zones 4 and 3. This can be calculated as a fraction of the width of those zones.

Force on angle $= \dfrac{12}{33.6} \times$ resistance of Zone 4 $+ \dfrac{125}{259.6} \times$ resistance of Zone 3

$= 187.0 + 73.9$ kN

$= 261.3$ kN

As this force occurs at mid span the welds or bolts must transmit this force over a length of half the span. This analysis assumes a uniform transfer of longitudinal force among the welds or bolts, as follows:

Force/metre $= \dfrac{261.3}{{}^1/_2 \times 6}$

$= 87.1$ kN/metre

(i) Use of bolts

The centre of the angle vertical leg is at 520°C. The Code specifies that a strength reduction factor should be taken as 80% of that given for steel based on 0.5% strain. This is conservative pending further research information.

SRF $= 0.8 \times 0.57$ (Code E.1 (g))

$= 0.456$

Strength of 16 mm dia grade 8.8 bolt $= 58.9$ kN

Reduced strength $= 0.456 \times 58.9$

$= 26.86$ kN

The bolts must carry both the longitudinal and vertical force.

Combined load $= \left(261.3^2 + \left(\dfrac{121.5}{2}\right)^2\right)^{1/2}$

$= 268.3$ kN

Therefore 10 16 mm dia bolts per half span are required.

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CALCULATION SHEET

Job No. PUB 7800	Sheet 8 of 8		Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 3, Shelf Angle			
Client SCI	Made by GMN	Date 30 Mar 90	
	Checked by RML	Date 18 Apr 90	

(ii) Use of welds

The top of the angle is at 401°C. The Code specifies similar reductions in strength for welds and bolts.

SRF $= 0.8 \times 798$ (Code E.1(g))
$= 0.638$

For a 6 mm fillet weld

Capacity $= 0.638 \times 1.07$
$= 0.68$ kN/mm
$= 680$ kN/m

For a 4 mm fillet weld

Capacity $= 0.638 \times 0.714$
$= 0.46$ kN/mm
$= 455$ kN/m

Capacity required

$= (87.1^2 + 20.25^2)^{1/2}$
$= 89.4$ kN/m

Clearly both 4 mm and 6 mm intermittent fillet welds could be used. Final choice would be governed by what is economic and practical.

The Steel Construction Institute	Job No. PUB 7800	Sheet 1 of 6	Rev.
Silwood Park Ascot Berks SL5 7QN	Job Title Handbook to BS5950: Part 8		
Telephone: (0990) 23345	Subject Design Example 4, Concrete Filled SHS		
Fax: (0990) 22944 Telex: 846843	Client SCI	Made by GMN	Date 9 Feb 88
CALCULATION SHEET		Checked by RML	Date 18 Apr 90

CONCRETE FILLED RECTANGULAR HOLLOW SECTION (Section 11.2)

(A) Plain or fibre reinforced concrete

The use of a 300 × 200 × 6.3 thick RHS column, grade 43C, is considered in a situation where 30 minutes fire resistance is required.

Design conditions:

Height	3.5 m (effective)
Factored load at fire limit state	1200 kN
Concrete grade	40 N/mm²

(i) No applied moments

Clause 4.6.2.1 states that the fire resistance dependent load ratio for a concrete filled hollow section is:

$$\eta \geq \frac{F + 6\left(\dfrac{M_x}{d_x} + \dfrac{M_y}{d_y}\right)}{0.83 \times f_{cu} \times A_c \times K}$$

where F = Compressive load at fire limit state

M_y = Applied moment at fire limit state about xx axis

M_y = Applied moment at fire limit state about yy axis

d_x = Depth of concrete normal to xx plane

d_y = Depth of concrete normal to yy plane

f_{cu} = Cube strength of concrete

K = Buckling factor (Code Table 12)

η = Fire resistance dependent limiting load ratio (Code Table 11)

A_c = Cross-sectional area of concrete

$A_c = (300 - 2 \times 6.3)\,(200 - 2 \times 6.3)$

$= 53859\ \text{mm}^2$

Buckling factor, K, is a function of L_e/r_c

L_e = effective length

$= 3500$ mm

Discuss me ...

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	Job Title Handbook to BS5950: Part 8			
	Subject Design Example 4, Concrete Filled SHS			
	Client SCI	Made by GMN	Date 9 Feb 88	
		Checked by RML	Date 18 Apr 90	

If buckling is considered about the weak axis, then for a rectangular section:

$$r_c = \left[\frac{I_{yy}}{A}\right]^{1/2} = \frac{d_y}{\sqrt{12}}$$

$d_y = 200 - 2 \times 6.3 = 187.4$ mm

$\therefore r_c = 54.1$ mm

$$\frac{L_e}{r_c} = \frac{3500}{54.1} = 64.7$$

$\therefore K = 0.745$ (Code Table 12)

$M_x = 0$ and $M_y = 0$

$$\therefore \eta = \frac{1200 \times 10^3}{0.745 \times 0.83 \times 40 \times 53859} = 0.901$$

From Code Table 11, for 30 minutes fire resistance, η must not exceed 1.00 for both plain and fibre reinforced concrete. Therefore in this case plain concrete will be adequate.

(ii) Applied moments

A nominal moment of 4 kNm is assumed to act about the weak axis (yy)

$d_y = 187.4$ mm

$M_y = 4$ kNm

$M_x = 0$

$$\therefore \eta = \frac{1200 \times 10^3 + \dfrac{4 \times 10^6}{187.4} \times 6}{0.745 \times 0.83 \times 40 \times 53859}$$

$$= 0.997$$

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CALCULATION SHEET

Job No. PUB 7800		Sheet 3 of 6	Rev.
Job Title Handbook to BS5950: Part 8			
Subject Design Example 4, Concrete Filled SHS			
Client SCI	Made by GMN	Date 9 Feb 88	
	Checked by RML	Date 18 Apr 90	

Check for concrete in tension. This does not occur provided:

$$6\left(\frac{M_x}{d_x}+\frac{M_y}{d_y}\right) \le F \text{ (Code 4.6.2.1)}$$

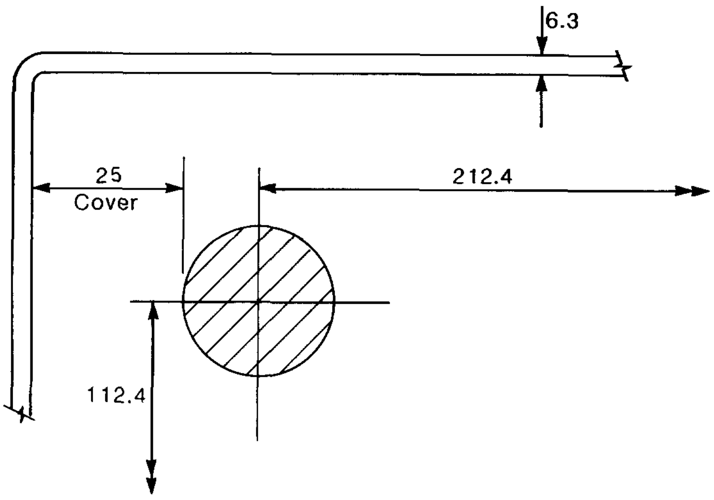
$$6 \times \frac{4 \times 10^6}{187.4} \times \frac{1}{10^3} = 128.1 \text{ kN}$$

but $F = 1200 \text{ kN} > 128.1 \text{ kN}$

$\therefore$ Again plain concrete is adequate.

(B) Bar reinforced concrete

The RHS used in the previous example has reinforcement in the form of 4×25 mm dia. high yield bars placed in the corners.



(i) Compressive capacity for 60 minutes fire resistance. (Clause 4.6.2.2).

$$\eta \ge \frac{F}{K\,(0.83\, f_{cu}\, A_c + f_{yr}\, A_r)\left(1 - \frac{M_x}{M_{px}} - \frac{M_y}{M_{py}}\right)}$$

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	Job Title Handbook to BS5950: Part 8		
	Subject Design Example 4, Concrete Filled SHS		
	Client SCI	Made by GMN	Date 9 Feb 88
		Checked by RML	Date 18 Apr 90

f_{yr} = Yield strength of reinforcement = 460 N/mm²

A_r = Area of reinforcement

M_{px} = Plastic moment of resistance of reinforcement about xx axis

M_{py} = Plastic moment of resistance of reinforcement about yy axis

E_r = 200,000 N/mm²

I_c about weak axis, (gross area)

$$= 287.4 \times \frac{187.4^3}{12} \times 10^{-4}$$

$$= 15762.1 \text{ cm}^4$$

$$I_r = 2 \times \frac{\pi \times 25^2}{4} \times \frac{112.4^2}{2} \times 10^{-4}$$

$$= 620.1 \text{ cm}^4$$

$$I_{c,\ nett} = 15762.1 - 620.1$$

$$= 15142 \text{ cm}^4$$

$$r_c = 0.043 \times \left(\frac{450 \times 40 \times 15142 \times 10^4 + 200 \times 10^3 \times 620.1 \times 10^4}{0.83 \times 40 \times 51895 + 460 \times 1963.5} \right)^{1/2}$$

$$= 52.84 \text{ mm}$$

$$\frac{L_e}{r} = \frac{3500}{52.84} = 66.23$$

$\therefore K = 0.74$ (Code Table 12)

Determine the load that can be carried with no moments for 60 minutes fire resistance.

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CALCULATION SHEET

Job No. PUB 7800	Sheet 5 of 6	Rev.
Job Title Handbook to BS5950: Part 8		
Subject Design Example 4, Concrete Filled SHS		
Client SCI	Made by GMN	Date 9 Feb 88
	Checked by RML	Date 18 Apr 90

From Code Table 11, $\eta = 0.509$

$$A_c = 287.4 \times 187.4 - 4 \times \pi \times \frac{25^2}{4}$$
$$= 51895 \text{ mm}^2$$

$$A_r = 4 \times \pi \times \frac{25^2}{4}$$
$$= 1963 \text{ mm}^2$$

$$\therefore F = 0.509 \times 0.74\,(0.83 \times 40 \times 51895 + 460 \times 1963) \times 10^{-3}$$
$$= 989.1 \text{ kN}$$

Calculate the effect of the maximum axial load if moments of 20 kNm about the xx axis and 8 kNm about the yy axis are applied.

M_{px} and M_{py} are to be calculated.

For a pair of bars the plastic moment capacity is given by

M_p = Tensile capacity of bar x spacing of bars

$$\therefore M_{px} = 2\left(460 \times \frac{1963}{4} \times 212.4\right) \times 10^{-6}$$
$$= 95.9 \text{ kNm}$$

$$\therefore M_{py} = 2\left(460 \times \frac{1963}{4} \times 112.4\right) \times 10^{-6}$$
$$= 50.7 \text{ kNm}$$

The Steel Construction Institute Silwood Park Ascot Berks SL5 7QN Telephone: (0990) 23345 Fax: (0990) 22944 Telex: 846843 **CALCULATION SHEET**	Job No. PUB 7800		Sheet 6 of 6	Rev.
	Job Title Handbook to BS5950: Part 8			
	Subject Design Example 4, Concrete Filled SHS			
	Client SCI	Made by GMN	Date 9 Feb 88	
		Checked by RML	Date 18 Apr 90	

The load capacity is reduced by the following factor to allow for the effects of biaxial bending:

$$1 - \frac{M_x}{M_{px}} - \frac{M_y}{M_{py}}$$

$$= 1 - \frac{20}{95.9} - \frac{8}{50.7}$$

$$= 0.63$$

$$\therefore F = 989.1 \times 0.63$$

$$= 626.8 \text{ kN}$$

(C) Reduced fire protection thickness for concrete filled rectangular hollow section

Unfilled section $200 \times 100 \times 5$ RHS

$H_p/A =$ 205 m^{-1}

For 60 minutes fire resistance, assume that 17 mm of sprayed vermiculite cement is required.

Filled section

Code Table 13 gives a modification factor of 0.69 for H_p/A of 200 m^{-1} and 0.55 for H_p/A of 260 m^{-1}

$\therefore$ modification factor for $H_p/A = 205 \text{ m}^{-1}$

$$= 0.69 - \frac{5}{60} \times 0.06$$

$$= 0.685$$

$\therefore$ Thickness of protection may be reduced to

$0.685 \times 17 = 11.6$ mm, say 12 mm

This thickness will provide adequate thermal insulation. However, when using proprietary data the restrictions on the use of any product should always be noted.