

FIRE RESISTANCE DESIGN OF STEEL FRAMED BUILDINGS



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In accordance with Eurocodes and the UK National Annexes

W I Simms BEng PhD CEng MICE





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FOREWORD

This publication is based on an SCI technical report RT1100 *Fire Engineering Eurocodes*, which was issued in March 2006 as the final deliverable of the Partners in Innovation (Pil) project STBF/004/00052C cc 2512. The project was funded by UK government through the Department of Trade and Industry (DTI), with additional industry support from Corus (now Tata Steel).

The SCI was the lead partner in the project. The project team included the following representatives of the partners.

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When RT1100 was drafted, the UK National Annexes, Published Documents and other NCCI material related to the relevant Eurocode Parts were not available. The report text has been updated to reflect the requirements of the later documents. The scope of the guidance has also been adjusted to align with other SCI publications and current UK construction practice.

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SUMMARY

This publication provides a general overview of the fire design of steel and composite structures in accordance with the Eurocodes. It introduces the basis of design for fire situations, which are considered as accidental situations, and the criteria that need to be met. The direct action of a fire is essentially the heat flux transferred to the members. The basis of determining heat flux for the two options of a nominal temperature-time curve for a predetermined duration and a natural fire model for the duration of the fire is explained.

The temperature of the structural members as a result of the heat flux received may be determined using thermal analysis. A simplified method of analysis is given for protected and unprotected structural elements. These simple equations can be evaluated in a sequence of small time steps, which makes the process well suited to spreadsheet software.

The strength and stiffness of both steel and concrete are modified at elevated temperatures. Extensive research has led to standardized relationships that can be used to determine structural behaviour in fire. The variation of properties with temperature is discussed and graphical presentations given.

The Eurocodes permit fire resistance to be determined by either simple or advanced calculation models. This guide covers simple models for both steel and composite members. The simple models offer the choice between calculation in the time domain (determining the time at which the fire resistance has fallen to the point of failure) or in the temperature domain (determining a uniform 'critical temperature' at which failure occurs). It is explained that the latter choice offers the simplest method for isolated composite members.

INTRODUCTION

1.1 Scope

This guide has been prepared to help designers become familiar with the fire design of steel and composite building structures, according to the Eurocodes. It provides a general overview but does not give detailed guidance for the design of particular elements of construction. More detailed guidance is given in other design guides for specific forms of construction.

For a general introduction to the Eurocodes, see *Steel Building Design: Introduction to the Eurocodes* (SCI P361)^[1]. That publication lists the principal Eurocode documents that are relevant to steel and composite building structures, introduces the National Annexes that accompany each Part of a Eurocode, sets out the design basis that is generally adopted and the procedures for determining actions and resistances.

The present publication builds on that introduction by providing guidance on the recommendations for fire resistance design. Specifically it provides guidance on the use of:

BS EN 1991-1-2, *Eurocode 1. Actions on structures.*

General actions - Actions on structures exposed to fire^[2]

BS EN 1993-1-2, *Eurocode 3. Design of steel structures.*

General rules - Structural fire design^[3]

BS EN 1994-1-2, *Eurocode 4. Design of composite steel and concrete Structures.*

General rules - Structural fire design^[4].

1.2 The Eurocodes

A comprehensive list of Eurocode Parts relevant to the design of steel-framed and composite buildings is given in P361.

In the present publication, which makes reference to the three Eurocode Parts mentioned above, references are generally given in the form 3-1-2/3.1, meaning Clause 3.1 of BS EN 1993-1-2. Where references are made to the UK National Annexes, the clause numbers are preceded by 'NA'.

National Annexes

Where the opportunity is given in the text of the Eurocode, the National Annex will, as appropriate:

- specify the value of a factor
- specify which design method to use
- state whether an informative annex may be used.

In addition, the National Annex may give references to publications that contain non-contradictory complementary information (NCCI) that will assist the designer. A notable example of NCCI for fire design is PD 6688-1-2^[5], which provides background to the UK NA to BS EN 1991-1-2.

The guidance given in a National Annex applies to structures that are to be constructed within that country. National Annexes are likely to differ between countries within Europe.

1.3 UK Building Regulations

The performance requirements in relation to fire safety in the UK are defined in the Building Regulations, with regional variations existing between England & Wales^[6], Scotland^[7] and Northern Ireland^[8]. Each region has its own Statutory Instrument, which is the legislative document that sets out the functional requirements for buildings. These requirements are not prescriptive, which allows the designer maximum flexibility in choosing a solution for a particular building. Each region also has a guidance document that provides guidance as to how the functional requirements can be satisfied for common building types. In England and Wales the guidance document is Approved Document B^[9], which provides guidance on means of satisfying the five main performance requirements of the Building Regulations:

- B1 Means of warning and escape.
- B2 Internal fire spread (Linings).
- B3 Internal fire spread (Structure).
- B4 External fire spread.
- B5 Access and facilities for the fire service.

The main requirements in terms of structural performance are given in B3 and B4.

The guidance on structural stability is based on exposure of structural elements to the standard fire, as defined by BS EN 1363-1^[10] and is given in terms of time. However, as the guidance is performance-based, this does not exclude the use of alternative thermal actions such as the parametric fire or localised fire models provided by BS EN 1991-1-2. However, it should be noted that following the recommendations of the Approved Document B is taken as evidence of compliance with the Building Regulations, whereas if an alternative solution is proposed it is up to the designer to demonstrate that the requirements of the Regulations have been met.

Periods of fire resistance for structural elements are given in relation to building height and usage (Approved Document B, Table A2). For various elements, the performance criteria in terms of load bearing resistance (R), Integrity (E) and Insulation (I) are also specified (Approved Document B, Table A1). The fire resistance of structural elements assessed using the Eurocodes can be compared against these benchmark performance criteria.

Guidance similar to that in Approved Document B is provided by Technical Handbooks^[11] in Scotland and Technical Booklet E^[12] in Northern Ireland.

1.4 Eurocode terminology

Some of the terminology used in the Eurocodes will be new to UK designers, but terms have been chosen carefully, for clarity and to facilitate unambiguous translation into other languages. The presentation of symbols has also been rigorously defined (although not always consistently applied) and some conventions are different.

The chief differences in terminology are:

- “Actions” = loads, imposed displacements, thermal strains
- “Effects” = internal bending moments, axial forces etc.
- “Resistance” = capacity of a structural element to resist bending moment, axial force, shear, etc.
- “Verification” = check
- “Execution” = construction (fabrication, erection, etc.)

Note that in addition to the meaning of “actions” given above, actions also include the heat flux from fires; see further discussion in Section 3.

Eurocode symbols

The Eurocode system uses the ISO convention for symbols and subscripts. Where multiple subscripts occur, a comma is used to separate them. Four main subscripts and their definitions are given below:

EUROCODE SUBSCRIPT	DEFINITION	EXAMPLE	
Ed	Design value of an effect	M_{Ed}	Design bending moment
Rd	Design resistance	M_{Rd}	Design resistance for bending
el	Elastic property	W_{el}	Elastic section modulus
pl	Plastic property	W_{pl}	Plastic section modulus

For fire design, additional abbreviations are introduced into the subscripts, to distinguish them from the parameters for normal temperature design. Notably, the following subscripts are used:

EUROCODE SUBSCRIPT	DEFINITION
fi	Property in fire conditions
θ	Property at elevated temperature θ
m	Property in fire conditions when member is unprotected
p	Property in fire conditions when member is protected by insulation

BASIS OF DESIGN

2.1 Basis of structural design in BS EN 1990

The basis of structural design set out in BS EN 1990^[43] is expressed almost exclusively in relation to normal temperature design. It states that the adequacy of structures is to be verified using the principles of limit state design and gives the principles for determining actions, for modelling to determine the structural effects of actions, and for determining the resistances to those effects.

For the STR ultimate limit state (see BS EN 1990, 6.4.1), the basic requirement is expressed as:

$$E_d \leq R_d$$

where:

E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments

R_d is the design value of the corresponding resistance.

The effects of actions depend on the combinations of actions that can occur and BS EN 1990 gives expressions for the effects for the following classes of combination of actions at the ultimate limit state:

- Fundamental combinations (for persistent and transient situations)
- Combinations for accidental situations.

For fundamental combinations BS EN 1990 gives the two alternative methods to determine the design value of the effects of combined actions. The design value may be determined from either expression 6.10 or from the less favourable of expressions 6.10a and 6.10b.

The first method is to express the combination of actions as:

$$\sum_{j \geq 1} \gamma_{Gj} G_{kj} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i \geq 2} \gamma_{Qi} \psi_{0,i} Q_{k,i} \quad (6.10)$$

The second method uses the more onerous of the following two expressions for the combination of actions:

$$\sum_{j \geq 1} \gamma_{Gj} G_{kj} + \gamma_P P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i \geq 2} \gamma_{Qi} \psi_{0,i} Q_{k,i} \quad (6.10a)$$

$$\sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \quad (6.10b)$$

For accidental situations, the following expression is given:

$$\sum_{j \geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i} \quad (6.11b)$$

The occurrence of a fire within a building is considered to be an accidental design situation.

In the above expressions:

“+”	represents ‘to be combined with’
$G_{k,j}$	is the characteristic value of permanent action j
P	is the prestressing action
A_d	is the indirect thermal action due to fire (thermal expansion)
$\psi_{1,1}$	is the factor for the frequent value of the leading variable action
$\psi_{2,1}$	is the factor for the quasi-permanent value applied to the leading variable action
$\psi_{2,i}$	is the factor for the quasi-permanent value applied to the variable action i
ξ	is a reduction factor applied to unfavourable permanent actions (in 6.10b)
$Q_{k,1}$	is the characteristic value of the leading variable action
$Q_{k,i}$	is the characteristic value of the accompanying variable action i .

The choice between the use of the factor for frequent or quasi-permanent values (applied to the leading variable action) in 6.11b is given in BS EN 1991-1-2. Although it recommends the use of the quasi-permanent factor, the UK National Annex recommends the use of the frequent value. Therefore, assuming that there is no prestressing action, and the effect of indirect thermal actions can be ignored (see comment in Section 3.4.3), the combination of actions for design in the UK can be simplified to:

$$\sum_{j \geq 1} G_{k,j} + \psi_{1,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i}$$

Simultaneous occurrence with other independent actions does not need to be considered. However, where particular risks of fire arise as a consequence of other accidental actions (i.e. a fire following a gas explosion) then the overall risk should be considered when determining the overall safety concept of the building.

The above addresses only the effects side of the verification criterion. The thermal effects of the fire cause a significant reduction on the resistance side, but this is not dealt with in BS EN 1990.

2.2 Basis of design for fire situations

Because BS EN 1990 does not fully cover the basis of design in fire situations, more specific requirements are given in BS EN 1993-1-2 and in BS EN 1994-1-2. These set out the mechanical resistance and integrity criteria that need to be satisfied under nominal fire exposure and parametric fire exposure.

Additionally, these fire design Parts define the design values of mechanical and thermal material properties in relation to characteristic values. The partial factor $\gamma_{M,fi}$ is applied to characteristic values (although, since the value of $\gamma_{M,fi} = 1.0$ is recommended and accepted by the National Annexes, thermal properties are usually referred to without any designation as characteristic or design values).

The verification is expressed as the requirement, at time t during the fire exposure that:

$$E_{fi,d,t} \leq R_{fi,d,t}$$

The effects of indirect actions (internal forces and moments induced in the structure by deformations and restrained thermal expansion) do not need to be considered when fire safety is based on the standard temperature-time curve. In other cases, indirect actions need not be considered when the effect is identified as being negligible or when the boundary conditions or design model are judged to be conservative.

Simplified requirement for member analysis

As a simplification, the value of $E_{fi,d}$ for member analysis may be taken as:

$$E_{fi,d} = \eta_{fi} E_d$$

where:

E_d is the design value of the effect of the fundamental combination of actions (ultimate limit state) as given in BS EN 1990

η_{fi} is a reduction factor for the design load level.

The value of the reduction factor η_{fi} will depend on whether Equation 6.10 or 6.10a and 6.10b, given in BS EN 1990, is used for the fundamental combination.

If Equation 6.10 of BS EN 1990 is used for the fundamental combination the reduction factor η_{fi} is given by:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_K + \gamma_{Q,1} Q_{k,1}}$$

If Equations 6.10a and 6.10b are used for the fundamental combination the reduction factor η_{fi} is given by the smaller value of the following two expressions:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_K + \gamma_{Q,1} \psi_{0,1} Q_{k,1}}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_K + \gamma_{Q,1} Q_{k,1}}$$

Where all the parameters are as defined above.

2.3 Fire design procedures

BS EN 1991-1-2, 2.1 sets out four main steps in a structural fire design analysis:

- Selection of relevant design fire scenarios.
- Determination of the corresponding design fires.
- Calculation of temperature evolution within structural members.
- Calculation of the mechanical behaviour of the structure exposed to fire.

The clause notes that mechanical behaviour depends on the thermal actions and their thermal effect on material properties and indirect mechanical actions (A_d in expression 6.11b), as well as on the direct effect of mechanical actions (G_k and Q_k in expression 6.11b).

These main steps are harmonised with the requirements of national building regulations^[6,7,8], which are the statutory instruments that set out the performance based requirements that all fire design must meet.

2.3.1 Design fire scenarios

When fire design is carried out to fulfil the requirements of prescriptive rules such as those given in Approved Document B, the design fire scenario is a fully developed fire affecting one compartment at a time. The temperature of the compartment atmosphere with time is described using the standard temperature-time curve given in BS EN 1991-1-2.

For fully developed compartment fires, the temperature-time curve for the compartment atmosphere can alternatively be determined from natural fire models such as the parametric temperature-time curve given in BS EN 1991-1-2 Annex A. This simple model allows the atmospheric temperature in the compartment to be calculated as a function of compartment geometry, ventilation conditions, thermal properties of the compartment boundaries, the fire growth rate and fire load density.

Where performance-based design is undertaken, the fire scenario can be based on localized fire behaviour or on fully developed fire behaviour. The choice will depend on the geometry of the compartment being considered and the magnitude and distribution of the fire load within the compartment. The development of atmospheric temperature with time will be based on the physical characteristics of the compartment and the fire loading it contains. This temperature-time relationship can be evaluated using simple models, zone models or computational fluid dynamics (CFD). For a localised fire plume model, the model recommended by the National Annex to BS EN 1991-1-2 provides a simple calculation technique based on the geometrical size of the fire, the rate of heat release from the fire load and the geometry of the compartment. The use of zone models and CFD, although permitted by the Eurocodes, is beyond the scope of this publication. A useful introduction to these techniques can be found in *Guide to the advanced fire safety engineering of structures*^[14].

2.3.2 Temperature analysis

BS EN 1991-1-2 provides three main methods for determining the fire development. These are the standard fire curves, the parametric fire curves and localised fire curves. The standard fire curves are the temperature-time relationships used to control the furnace gas temperature in standard fire resistance test to BS EN 1363-1^[10]. Parametric fire curves allow the gas temperature within a real fire compartment to be estimated for the post flash-over condition. Finally the localised fire curves are used when the size of the fuel bed involved in the fire is small relative to the size of the compartment. This technique can also be used to model pre flash-over conditions. BS EN 1991-1-2 also permits the use of advanced analysis techniques such as CFD or zone models.

2.3.3 Mechanical analysis

There are three main design methods for evaluating structural performance in fire conditions that can be adopted for fire resistance design in accordance with the Eurocodes:

- Tabular data.
- Simple calculation models.
- Advanced calculation models.

Although unusual for individual projects, the performance of individual elements could also be evaluated by fire testing.

The structural analysis provided by rules in BS EN 1993-1-2 is based on simple design models that can be used to determine the resistance of individual structural members subject to tension, compression and bending. BS EN EN 1993-1-2 also provides a critical temperature method similar to the limiting temperature method previously provided in BS 5950-8^[15].

BS EN 1994-1-2 also provides simple calculation models but in addition includes tabular data that defines suitable geometry of composite structural elements for standard periods of fire resistance. Tabular data and simple calculation models are normally applied to the analysis of individual structural members with simple boundary conditions.

Advanced calculation models are normally used to evaluate the performance of a sub-frame or the entire structure where the benefit of interaction between structural elements can be considered. Although it is possible to carry out advanced analysis of the structure for exposure to the standard temperature-time curve, this method typically forms part of a design to performance-based requirement that considers natural fire behaviour.

A detailed treatment of advanced analysis is beyond the scope of the Eurocodes and of this design guide, although the design principles in the Eurocodes may inform an advanced analysis. A brief introduction to the use of advanced analysis may be obtained from a guide published by the Institution of Structural Engineers^[14].

The load bearing resistance of structural elements can be assessed by adjusting the strength and stiffness of the materials to allow for temperature effects and using simple engineering models similar to those used in normal temperature design.

Temperature development in structural members

BS EN 1993-1-2 and BS EN 1994-1-2 provide simple heat transfer models that can be used to determine the temperature rise in protected and unprotected steel sections in fire conditions. However, for protected sections, information on the temperature-dependent material properties of fire protection materials is required. As fire protection manufacturers do not make this information readily available in the public domain, it is more practical for the evaluation of members exposed to the standard temperature-time curve to use manufacturers' data derived from testing and assessment to BS EN 13381-4^[16] for non-reactive fire protection and to BS EN 13381-8^[17] for intumescent coatings. BS EN 1993-1-2 also provides a method of determining the heat transfer to structural steelwork located outside the building envelope.

The Informative Annexes to BS EN 1994-1-2 also contain simple models to allow for the effect of temperature on the cross sectional properties. Annex D gives a simple thermal model for composite slabs that permits the evaluation of insulation performance and the calculation of the temperature for key elements on the cross section. Annex F for partially encased beams and Annex G for partially encased columns also provide a method of calculating the effect of temperature on the cross sectional properties of these members. The advantage of these models is that they permit direct calculation of the temperature distribution on the cross section, for a given fire resistance period.

Design procedures for determining fire resistance

Evaluation of the performance of a structural element is usually based on its resistance to loads at elevated temperature. In some cases, where the structural element also forms part of a fire resisting compartment, the integrity and insulation performance must also be assessed. The specific requirements for common elements in building construction can be found in Approved Document B.

For uniformly heated members, it is possible to calculate the variation of member resistance with temperature, independent of time. Using this approach, the critical temperature of the member can be established for a given level of load. This approach is recommended when the members are to be fire protected: given the critical temperature, the section factor and the period of fire resistance required, an appropriate thickness of fire protection material (that has been tested and evaluated in accordance with BS EN 13381) can be specified.

Where the members are not uniformly heated, it will be necessary to determine the temperature-time development in the structural member first and then calculate the member resistance for a given time. This will be the case when the member is being designed for exposure to a natural fire model, such as the parametric fire curve model

given in BS EN 1991-1-2 Annex A. In this case, the thermal response of the member cannot be predetermined and must be calculated as it develops with time. The member resistance is then checked for the maximum temperature attained during the fire event.

For composite members, where significant thermal gradients exist, regardless of the fire scenario being considered, it is difficult to determine a single value of reference temperature that describes failure. In these circumstances, it will be necessary to calculate the development of temperature with time before conducting member resistance calculations based on the distribution of temperature in the member at the required fire resistance time.

Deflection limits for members in fire are not specified, except in testing standards. However, deflection will influence the performance of compartments and some forms of fire protection material. In most cases, the ability of fire protection to stay attached to the structural element at large deformations, often referred to as 'stickability', is evaluated by loaded fire tests in which the test specimens attain a level of deflection that is compatible with the 2% strain level at which steel yield strength is evaluated. In these cases, no further evaluation of deflection is required. Where load bearing elements interact with other fire resistance elements, such as compartment walls, the interaction between these elements should be considered. It is often convenient to assume that a loaded structural element subject to bending will attain a mid span deflection in fire equal to span divided by 20. The designer needs to consider how this deformation can be accommodated at the interface between elements or the how the load will be shared between the elements.

THERMAL ACTIONS

The direct action of a fire is essentially the heat flux transferred from the fire to the structural members. It is generally presumed that there are no direct mechanical actions due to the fire (i.e. there are no vertical or horizontal loads) but there may be indirect mechanical actions arising from constraints to the change in structural geometry (increase in length, curvature due to temperature gradient). This Section discusses the temperature evolution in fire and the consequent heat flux that is transferred to the members. Indirect actions are discussed in Section 3.4.3.

3.1 Thermal actions - heat flux

The thermal action on a structural member is the heat flux into the member. The net heat flux \dot{h}_{net} to the surface of a member is given in 1-1-2/3.1 as the sum of the heat transfers by convection $\dot{h}_{\text{net,c}}$ and by radiation $\dot{h}_{\text{net,r}}$, expressed as:

$$\dot{h}_{\text{net}} = \dot{h}_{\text{net,c}} + \dot{h}_{\text{net,r}} \quad [\text{W/m}^2]$$

with

$$\dot{h}_{\text{net,c}} = \alpha_c \cdot (\theta_g - \theta_m) \quad [\text{W/m}^2]$$

and

$$\dot{h}_{\text{net,r}} = \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma [(\theta_r + 273)^4 + (\theta_m + 273)^4] \quad [\text{W/m}^2]$$

where:

Φ	is the configuration factor	
ε_m	is the surface emissivity of the member	
ε_f	is the emissivity of the fire	
α_c	is the co-efficient of heat transfer by convection	$[\text{W/m}^2\text{K}]$
θ_g	is the gas temperature	$[\text{°C}]$
θ_m	is the surface temperature of the member	$[\text{°C}]$
θ_r	is the effective radiation temperature of the fire environment.	$[\text{°C}]$

3.2 Temperature analysis

BS EN 1991-1-2 provides two options for temperature analysis of members:

- Use of a nominal temperature-time curve, for a specified period of time, without any cooling phase.
- Use of a fire model for the full duration of the fire, including the cooling phase.

The two options are discussed below.

3.3 Nominal temperature-time curves

3.3.1 Standard temperature-time curve

The standard temperature-time curve is given in 1-1-2/3.2.1, expressed as:

$$\theta_g = 20 + 345 \log_{10}(8t + 1) \quad [^{\circ}\text{C}]$$

where:

θ_g is the gas temperature in the fire compartment [$^{\circ}\text{C}$]

t is the time. [min]

The coefficient of heat transfer by convection for this curve is:

$$\alpha_c = 25 \text{ W/m}^2\text{K}$$

This curve is often referred to as the cellulosic heating curve. Although it does not represent an actual fire, it forms the basis on which the fire resistance performance of load-bearing elements is evaluated and the curve is referred to in the Approved Document B.

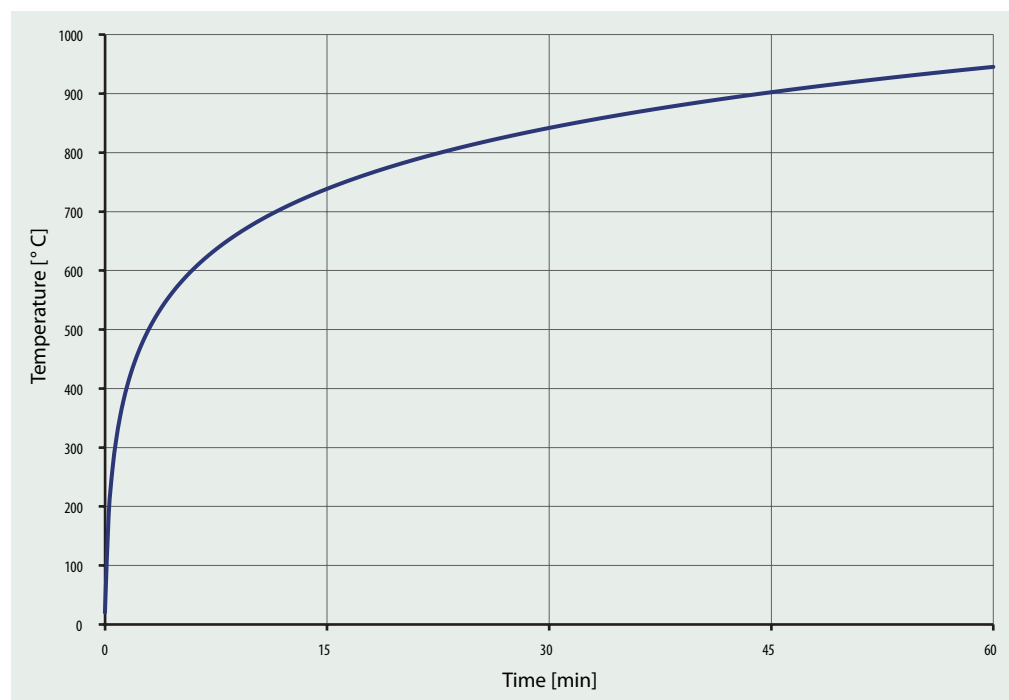


Figure 3.1
Standard temperature-time curve

The curve is shown in Figure 3.1.

3.3.2 External fire curve

To characterise a less severe fires immediately outside a building, or a fire emanating from within a building, for example to determine the condition of flames issuing from adjacent windows, 1-1-2/3.2.2 provides a temperature-time curve expressed as:

$$\theta_g = 660(1 - 0.687e^{-0.32t} - 0.313e^{-3.8t}) + 20 \quad [^{\circ}\text{C}]$$

where:

θ_g is the gas temperature in the fire compartment [$^{\circ}\text{C}$]

t is the time. [min]

The coefficient of heat transfer by convection for this curve is:

$$\alpha_c = 25 \text{ W/m}^2\text{K}$$

The curve is shown in Figure 3.2.

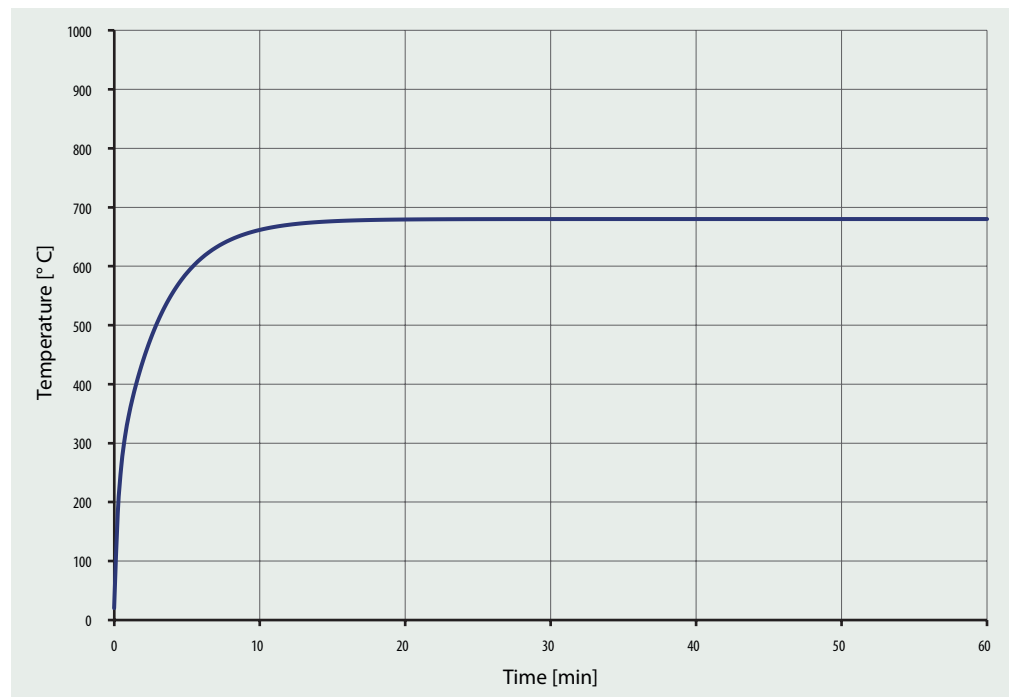


Figure 3.2
External fire curve

3.3.3 Hydrocarbon curve

The hydrocarbon temperature-time curve is given by 1-1-2/3.2.3 as:

$$\theta_g = 1080(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) + 20 \quad [^{\circ}\text{C}]$$

where:

θ_g is the gas temperature in the fire compartment [$^{\circ}\text{C}$]

t is the time. [min]

The coefficient of heat transfer by convection for this curve is:

$$\alpha_c = 50 \text{ W/m}^2\text{K}$$

This particular curve is representative of the heating rate of hydrocarbon pool fire.

This curve is shown in Figure 3.3.

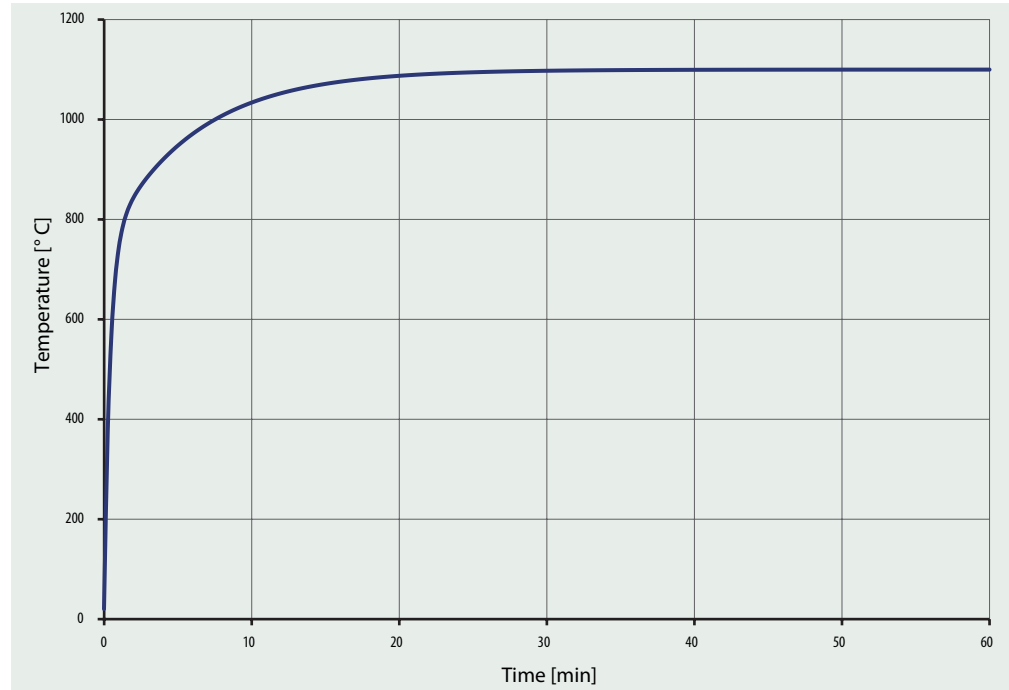


Figure 3.3
Hydrocarbon curve

3.4 Natural fire models

Three natural fire models are presented in BS EN 1991-1-2 and their definitions are given in Informative Annexes. (Informative Annexes may be adopted through the National Annex, or the National Annex may provide alternative non-contradictory complementary information.)

3.4.1 Compartment fires - parametric temperature-time curve

Annex A of BS EN 1991-1-2 presents a temperature-time relationship for members inside a compartment for post-flash-over fires. The temperature rises until a time t_{max} (which depends on whether the compartment is ventilation controlled or fuel controlled) and then falls.

In the heating phase, the temperature-time relationship is given by Annex A as:

$$\theta_g = 20 + 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) \quad [^{\circ}\text{C}]$$

where:

$$\theta_g \quad \text{is the gas temperature in the fire compartment} \quad [^{\circ}\text{C}]$$

$$t^* = t \cdot \Gamma \quad [\text{h}]$$

in which:

t	is the time	[h]
Γ	$= [O/b]^2 / (0.04/1160)^2$	[-]
O	is the opening factor: $O = A_v \sqrt{h_{eq}} / A_t$ Within the limits $0.02 \leq O \leq 0.20$	[m ^{1/2}]
A_v	is the total area of vertical openings	[m ²]
h_{eq}	is the weighted mean height of the vertical openings	[m]
A_t	is the total area of the compartment (walls, floor and ceiling including the openings)	[m ²]
b	is the thermal diffusivity: $\sqrt{(\rho c \lambda)}$ within the limits: $100 \leq b \leq 2200$ The lower value would be representative of a compartment or building with a controlled environment whereas the upper value is representative of a very poorly insulated structure.	[J/m ² s ^{1/2} K]

In the cooling phase, the temperature-time curve is given by:

$$\theta_g = \theta_{max} - 625(t^* - t_{max}^* \cdot x) \quad \text{for } t_{max}^* \leq 0.5$$

$$\theta_g = \theta_{max} - 250(3 - t_{max}^*)(t^* - t_{max}^* \cdot x) \quad \text{for } 0.5 < t_{max}^* < 2$$

$$\theta_g = \theta_{max} - 250(t^* - t_{max}^* \cdot x) \quad \text{for } t_{max}^* \leq 2$$

where:

$$x = 1.0 \quad \text{if } t_{max} > t_{lim}, \quad \text{or } x = \Gamma / t_{max}^* \quad \text{if } t_{max} = t_{lim}$$

The UK NA adopts the Annex A relationship but modifies some of the limiting values. For example, the minimum value for the opening factor is extended down to 0.01m^{1/2}, based on historical data and calibration^[5].

For a wall with different layers of material, limits are given for b , for example when the layer exposed to the fire has a greater diffusivity than the inner layer, a limiting thickness s_{lim} applies, given by:

$$s_{lim} = \sqrt{\frac{3600 t_{max} \lambda_1}{c_1 \rho_1}} \quad \text{[m]}$$

$$\text{With } t_{max} = \max \left[(0.2 \cdot 10^{-3} \cdot q_{t,d} / O); t_{lim} \right] \quad \text{[h]}$$

where:

c_1	is the specific heat of the exposed layer	[J/kgK]
ρ_1	is the density of the exposed layer	[kg/m ³]

$q_{t,d}$ is the design fire load density, within the limits $50 \leq q_{t,d} \leq 1000$ [MJ/m²]
 t_{lim} = 25 min, 20 min, and 15 min for fast, medium and slow fire growth rates respectively (see 1-1-2/Annex E for definitions). [h]

To account for different factors in walls, ceiling and floor, b should be taken as:

$$b = (\sum(b_j A_j)) / (A_t - A_v)$$

where:

A_j is the area of surface j , openings not included

b_j is the thermal diffusivity of surface j .

The comparison between atmospheric temperature measured in a fire compartment and the parametric fire curve from 1-1-2/Annex A, calculated for the compartment, is shown in Figure 3.4. The test data comes from a series of 21 natural fire tests carried out by British Steel^[18]. The compartment's internal dimensions were 8.66 m × 5.87 m × 3.9 m high with a two windows in each long wall measuring 3.67 m × 2.3 m. In this test, shutters were used to limit the ventilation area, to give an opening factor of 0.06 m^{1/2}. The fire load was provided by timber cribs arranged to give a fire load density of 15 kg/m² (equivalent to 270 MJ/m²).

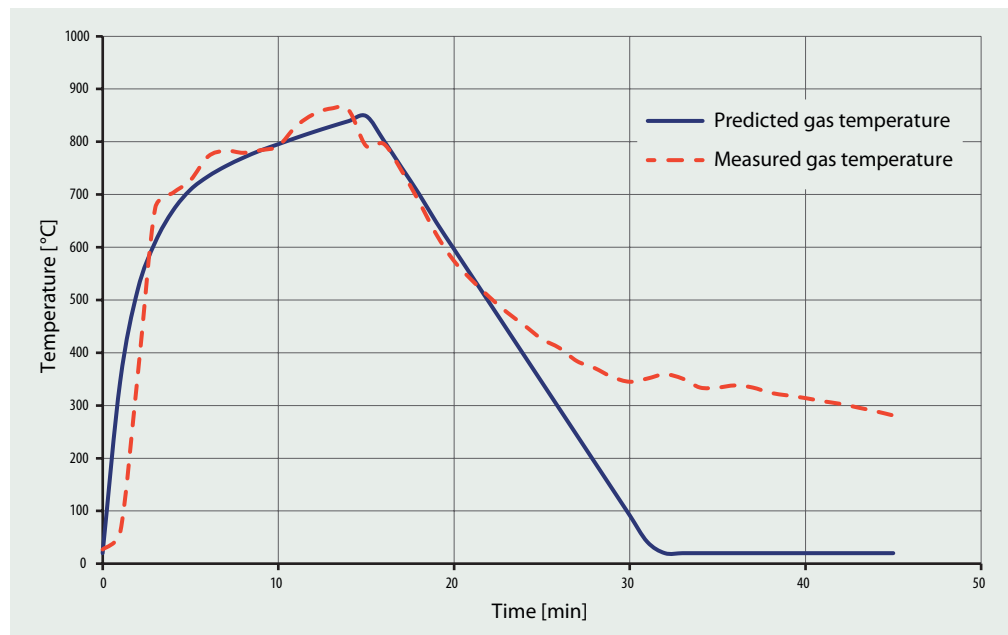
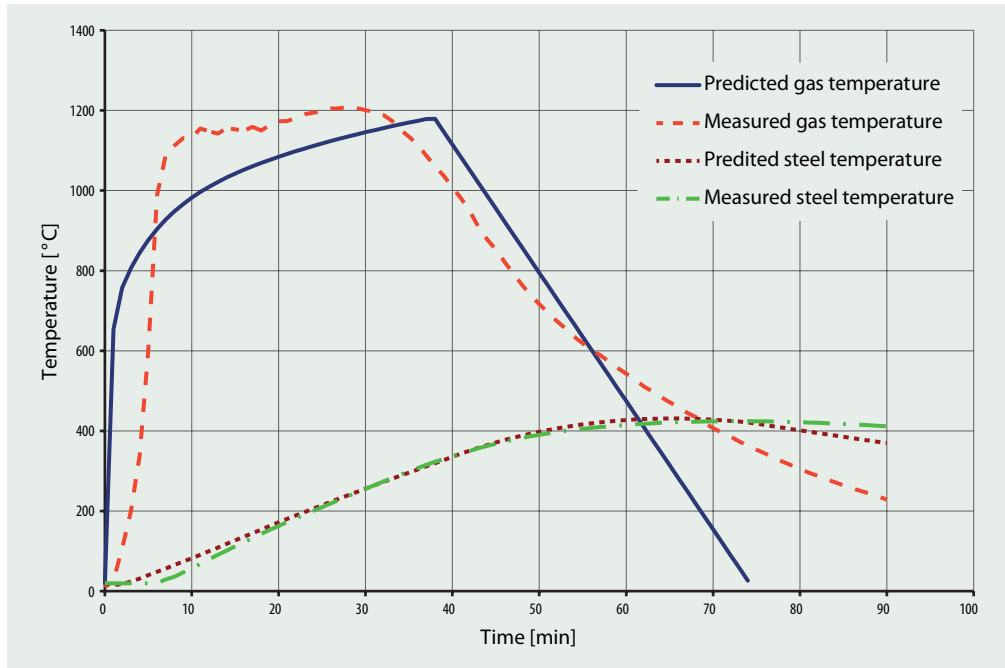


Figure 3.4
 Comparison
 of measured
 and predicted
 atmospheric
 temperatures

In general, good agreement has been observed between predicted and measured atmospheric temperatures. Even for cases where the correlation is less good, the predicted member temperatures were found not to suffer significant variations from measured values. One such case occurred for a fire compartment with a fire load of 40 kg/m² of wood and an opening factor of 0.07 m^{1/2}. The measured and predicted atmospheric temperatures are shown in Figure 3.5 alongside the measured and predicted member temperatures. Despite the differences between the atmospheric temperatures, the difference in member temperatures is not significant.

Figure 3.5
Comparison of
measured member
temperatures and
predicted member
temperatures based on
a parametric fire curve



3.4.2 Compartment fires - external members

Annex B introduces a methodology for calculating the fire and flame temperature for structural steel members external to the building façade.

It provides:

- the maximum temperatures of a compartment fire
- the size and temperature of the flames from the openings
- radiation and convection parameters.

The UK NA generally adopts Annex B but does qualify that adoption by stating that it should be used in conjunction with the complementary information in PD 6688-1-2, which expresses some caution about results in certain situations.

The analysis can be applied to fire compartments up to 70 m long, 18 m wide and 5 m high.

Two situations are considered:

- no forced draft
- forced draft.

No forced draught

The rate of burning or the rate of heat release is given in 1-1-2/B.4.1 by:

$$Q = \min \left\{ (A_f \cdot q_{f,d}) / \tau_F; 3.15(1 - e^{-0.036/O}) A_v \left\{ \frac{h_{eq}}{D/W} \right\}^{1/2} \right\} \quad [\text{MW}]$$

where:

τ_F is the period of free burning (usually 1200 seconds) [s]

D is the depth of the compartment (perpendicular to W) [m]
 W is the width of the wall containing the openings (windows). [m]

The temperature of the fire in the compartment is given by:

$$T_f = 6000(1 - e^{-0.1/O})O^{1/2}(1 - e^{0.00286\Omega}) + T_0 \quad [^\circ\text{K}]$$

where:

Ω is the fire load density as a function of the geometric parameters, given by $(A_f \cdot q_{f,d}) / (A_v \cdot A_t)^{1/2}$. [MJ/m²]

The dimensions of the flames issuing from the window are shown in Figure 3.6. Two different cases are considered, depending on whether there is a wall above the opening, and the aspect ratio of the opening.

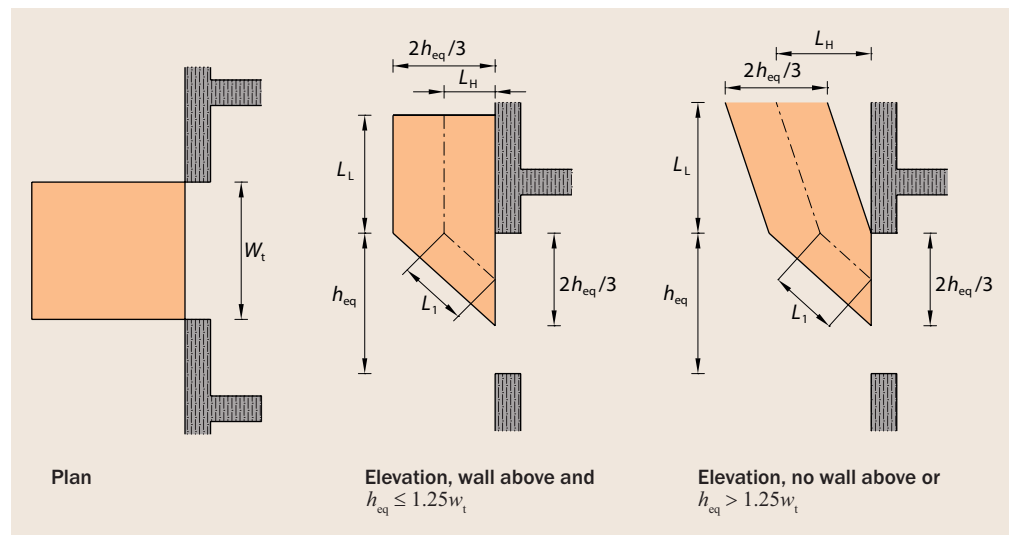


Figure 3.6
 Flame dimensions for compartments with no through draught

The height of the flame (L_L) above the top edge of the opening may be calculated as follows:

$$L_L = \max \left\{ 0; h_{eq} \left[2.37 \left(\frac{Q}{A_v \rho_g (h_{eq} g)^{1/2}} \right)^{2/3} - 1 \right] \right\} \quad [m]$$

With $p_g = 0.45 \text{ kg/m}^3$ and $g = 9.81 \text{ m/s}^2$, the equation may be simplified to:

$$L_L = 1.9 \left(\frac{Q}{w_t} \right)^{2/3} - h_{eq} \quad [m]$$

The width of the flame is equal to the width of the window and the flame depth at the opening is taken as two thirds of the opening height.

The horizontal projection of the flames, L_H , is influenced by the aspect ratio of the opening and its proximity to other openings. Three cases are considered when there is a wall above the window:

$$L_H = h_{eq} / 3 \quad \text{if } h_{eq} \leq 1.25w_t$$

$$L_H = 0.3h_{eq} (h_{eq} / w_t)^{0.54} \quad \text{if } h_{eq} > 1.25w_t \text{ and the distance to any adjacent opening is greater than } 4w_t$$

$$L_H = 0.454h_{eq} (h_{eq} / 2w_t)^{0.54} \quad \text{in other cases}$$

If there is not a wall above the window, the horizontal projection of the flames is given by:

$$L_H = 0.6h_{eq} (L_L / h_{eq})^{1/3} \quad [\text{m}]$$

The total length of the flame along its axis, L_f , will be zero when the height of the flame above the top of the opening, L_L , is zero.

If flame height L_L is greater than zero, then the flame length is given by one of the following two equations, depending upon whether there is a wall above the window and on the aspect ratio of the opening.

$$L_f = L_L + h_{eq} / 2 \quad \text{if there is a wall above the opening or if } h_{eq} \leq 1.25w_t$$

$$L_f = (L_L^2 + (L_H - h_{eq} / 3)^2)^{1/2} + h_{eq} / 2 \quad \text{if there is no wall above the opening or if } h_{eq} > 1.25w_t$$

The flame temperature at the window (K) is given by:

$$T_w = 520 / (1 - 0.4725(L_f \cdot w_t / Q)) + T_0 \quad [\text{K}]$$

The temperature of the flame at its tip is assumed to be 520 K above ambient (293 K) and this allows the temperature distribution along the flame axis to be calculated at any point from:

$$T_z = (T_w - T_0)(1 - 0.4725(L_x \cdot w_t / Q)) + T_0 \quad [\text{K}]$$

Where there is an awning or balcony projection this would affect the length of the flame by reducing its height or increasing its horizontal projection.

The emissivity of the flames at the window (ϵ_f) is taken as 1.0.

The emissivity of the flame along its length is given by:

$$\epsilon_f = 1 - e^{-0.3d_f} \quad [-]$$

Where d_f is the flame thickness. [m]

The convective heat transfer coefficient is given by:

$$\alpha_c = 4.67(1/d_{eq})^{0.4}(Q/A_v)^{0.6}$$

Forced draught

Where there are openings on two or more walls of a compartment it can be assumed there will be a preference for flames to emerge from one set of openings. This gives rise to forced draught conditions which influence:

- the rate of heat release
- the temperature of the fire within the compartment
- the flame height
- the horizontal projection of the flame
- the flame width
- the flame length along its axis
- the flame temperature emerging from the window.

For forced draught conditions, the rate of heat release is given by:

$$Q = (A_f q_{f,d})/\tau_F \quad \text{[MW]}$$

The temperature of the fire compartment, T_f is given by:

$$T_f = 1200(1 - e^{-0.00228\Omega}) + T_0 \quad \text{[K]}$$

The following flame dimensions are shown in Figure 3.7. The flame height, L_L above the top of the opening is given by:

$$L_L = \left(1.366 \left(\frac{1}{u} \right)^{0.43} \frac{Q}{A_v^{1/2}} \right) - h_{eq} \quad \text{[m]}$$

The horizontal projection of the flames is given by:

$$L_H = 0.605(u^2/h_{eq})^{0.22}(L_L + h_{eq}) \quad \text{[m]}$$

The flame width, w_f , is given by:

$$w_f = w_t + 0.4L_H \quad \text{[m]}$$

The flame length along the axis is given by:

$$L_f = (L_L^2 + L_H^2)^{1/2} \quad \text{[m]}$$

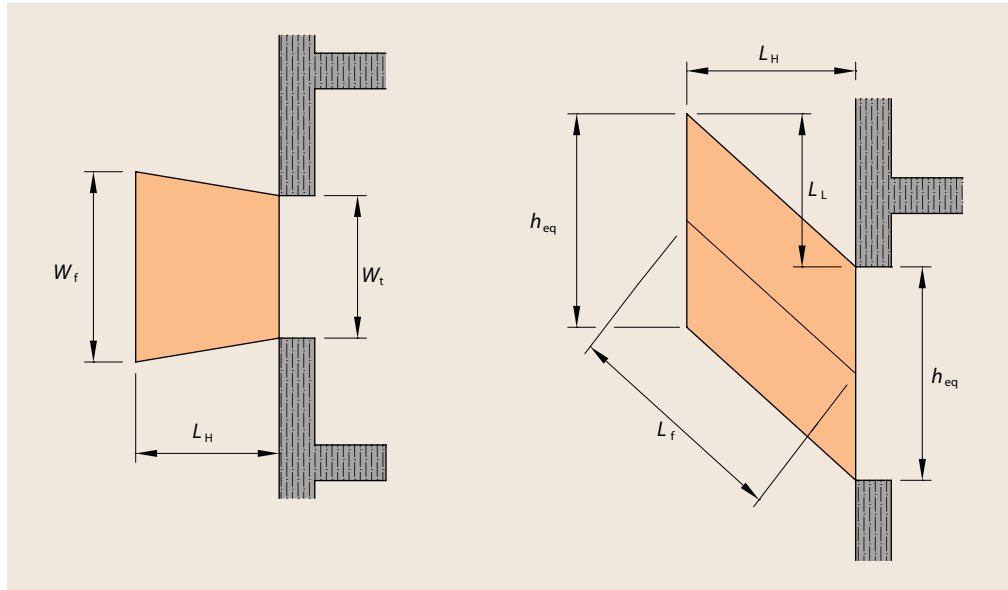


Figure 3.7
Flame dimensions,
forced draught
conditions

The flame temperature at the window is given by:

$$T_w = 520 / (1 - 0.3325 L_f (A_v)^{1/2} / Q) + T_0 \quad [\text{K}]$$

with $L_f (A_v)^{1/2} / Q < 1$.

The temperature of the flame at any point along its axis is given by:

$$T_z = \left(1 - 0.3325 \frac{L_x (A_v)^{1/2}}{Q} \right) (T_w - T_0) + T_0 \quad [\text{K}]$$

where:

L_x is the axis length from the window to the point at which the calculation is made. [m]

The emissivity of the flame at the window ε_f is taken as 1.0.

The emissivity of the flame along its length, outside the compartment, is given by:

$$\varepsilon_f = 1 - e^{-0.3d_f}$$

where:

d_f is the flame thickness.

The convective heat transfer coefficient is given by:

$$\alpha_c = 9.8 (1/d_{eq})^{0.4} (Q / (17.5 A_v) + u / 1.6)^{0.6}$$

Once the temperatures and profiles of the flames emerging from openings have been established, the heat transfer to the external structure is considered. This is achieved by

considering the configuration (or view) factor, which is determined using a set of geometric calculations. These are given in Annex B but are described more fully in Annex G.

In carrying out the analysis, it is possible to obtain temperature outputs that are unrealistic for a building fire. In this respect, an upper limit is placed upon the fire and flame temperatures of 1650 K and 1750 K respectively.

It is also possible to obtain negative flame heights when certain compartment and window geometries are selected. This indicates that the tip of the flame is below the top of the window.

3.4.3 Localised fires

Annex C of BS EN 1991-1-2 provides information for calculating the flame length and temperatures in the plume for localised fires. The UK NA does not adopt Annex C and instead refers to NCCI in PD 6688-1-2 for actions in localised fires; this in turn refers to PD 7974-1^[19].

Flame length for axi-symmetric fire source

For an axi-symmetric fire source, on the floor and away from walls, air is entrained from all sides and along the entire height of the plume until the plume becomes submerged in the smoke layer beneath the ceiling.

The height of the flaming region of such a fire is given by:

$$z_{fl} = 0.2Q^{2/5} \quad [m]$$

where:

z_{fl} is the mean height of luminous flames above the fuel surface [m]

Q is the total rate of heat release. [kW]

This equation is valid only for fires where:

$$z_{fl}/D_s > 3$$

where:

D_s is the diameter of the source. [m]

Limits for equation

The above expression for z_{fl} was originally developed for horizontal-surface fires without substantial in-depth combustion, such as liquid pool fires. Experience has shown that many practical fuel sources such as wood pallet fires can be predicted by this equation (with the exception of very openly constructed pallet stacks). Flame height is then measured from the top surface.

It should be noted that for some purposes, such as the design of fire detection systems, this equation will over-predict the flame height. It will therefore not be a conservative estimate for this purpose.

Flame length for line sources

A line source is a rectangular source where the longer side, D_s is greater than three times the shorter side. For a line source originating on the floor away from the walls, the mean height of luminous flames above the fuel surface is given by:

$$z_{fl} = \frac{0.035Q^{2/3}}{(d_s + 0.074Q^{2/5})^{2/3}} \quad [m]$$

where:

d_s is the dimension of the longer side of the source [m]

Q is the total heat release rate. [kW]

However, where the ratio of the length to width of the source is greater than about 5, z_{fl} is given by:

$$z_{fl} = \frac{0.035Q^{2/3}}{d_s^{2/3}}$$

Limits for equation

In practice, the equation may represent a flame that emerges from an open-fronted compartment fire, horizontal conveyor or cable fires, etc.

Flame lengths for corner room and wall fires

The presence of walls near the source of a plume can strongly influence the entrainment rate and other properties of the plume. One of the main influences that it can have is that of the flame height.

It should be appreciated that when using a design fire that is in the corner or against a wall of a room, the flame length will increase, compared to the same fire with the same heat release rate in the centre of the room.

Corner room fires are likely to increase flame heights by 75%; wall fires are likely to increase flame heights by 32%.

Flame emissivity

For a luminous flame, the emissivity may be taken as:

$$\varepsilon_f = 1 - \exp(-K\lambda_f)$$

where:

ε_f is the emissivity of the flame

K is the effective emission coefficient – see Table 3.1 [m⁻¹]

λ_f is the thickness of the flame. [m]

MATERIAL	EFFECTIVE EMISSION COEFFICIENT, K , (m^{-1})
Wood Cribs	0.51 – 1.1
Assorted Furniture	1.13
Diesel Oil	0.43
Polypropylene	1.8
Polystyrene	5.3
PMMA	1.3
Kerosene	2.6
Petrol	2.0
Alcohol	0.37

Table 3.1
Effective emission
coefficient for
various materials

If the flame length $z_f > 1$ m and the flame is luminous, it is common to assume black body behaviour and that the emissivity of the flame $\varepsilon_f = 1$.

Limits for equation

Flame emissivity is used in calculation procedures for thermal radiation fluxes. It is highly questionable whether alternative approaches are not better suited for the calculations of thermal radiative fluxes.

Calculation of radiative heat flux from flames requires as input data flame emissivity, effective values of flame temperature and the flame geometry, idealised as a simple shape such as a plane layer or an axi-symmetric cylinder or cone. Calculation procedures may be found in the *Handbook of Fire Protection Engineering*^[20]. The simplifying assumptions that are necessary make calculations difficult to use with confidence in practice. A simpler model, based on the assumption that radiation accounts for 20% to 30% of total heat release, works satisfactorily in many practical cases.

Flame radiation

The radiant heat flux from a flame depends on a number of factors and is given by:

$$Q_R = \phi \varepsilon_f \sigma \bar{T}^4 \quad [\text{Wm}^{-2}]$$

where:

ϕ is the configuration factor (geometric relationship between the flame and receiving object)

ε_f is the emissivity of the flame

σ is the Stefan-Boltzmann constant [5.67×10^{-11} W/m.K]

\bar{T} mean temperature of the flame. [K]

The configuration factor ϕ enables the calculation of radiant intensity at a point remote from the radiator. For the purposes of calculating ϕ , the flame is typically approximated to a simple geometric shape such as a rectangle or cone. If the flame is influenced by external air flows or fire induced flows, an appropriate configuration factor must be found.

3.4.4 Advanced fire models

Annex D covers a basic description of the different approaches that can be adopted in fire analysis.

The UK NA states that this Annex may be used but notes that “This Annex introduces the concept of heat balance in compartment fires and a very brief description of the fundamentals and complexities behind each of the types of fire models ranging from single zone models up to CFD analyses. However, apart from a statement on principles, there is little in terms of calculations that would enable an engineer to carry out any detailed analysis.”

3.5 Fire load densities

Informative Annex E of BS EN 1991-1-2 offers rules for determining fire load densities. These rules have not been adopted by the UK NA because the experts responsible for the UK NA have a fundamental disagreement about the use of factors that multiply the design fire load density to provide fires of much lower severity.

Instead, the UK NA refers to NCCI in Annex A of PD 6688-1-2. This will enable the fire load densities to be determined for use in calculating fire scenarios such as parametric temperature-time relationships, or time equivalent assessments for estimating the period of heating in the standard furnace test (to BS EN 1363-1).

3.6 Equivalent time of fire exposure

For the nominal temperature-time curves, a specified period is required. Annex F of BS EN 1991-1-2 offers recommendations for determining this period but these have not been adopted by the UK NA.

Instead, the UK NA refers to NCCI in Annex B of PD 6688-1-2.

3.7 Configuration factor

Annex G of BS EN 1991-1-2 provides a set of geometric relationships for use when determining the heat transfer (radiation) from the fire to the structural element. The UK NA adopts this Annex.

The information provided is based on simple geometry. It was intended this Annex primarily related to the radiation from a fire within a building to externally-placed structural members. However, the relationships can be used inside fire compartments for situations where part of a member’s profile creates a shadow effect and the shadowed part therefore receives a lower level of thermal radiation.

3.8 Indirect (mechanical) actions

Indirect actions can arise from:

- restrained longitudinal thermal expansion
- differing thermal expansion within statically indeterminate members
- thermal gradients through the cross-section inducing thermal stresses
- thermal expansion causing displacement of adjacent members (i.e. the lateral displacement of column heads caused by thermal expansion of the adjacent beams)
- thermal expansion of heated members affecting other members outside the fire compartment.

It is general practice to ignore indirect actions for the design of members subjected to nominal temperature-time curves. In the case where indirect actions arising from thermal gradients through the cross-section cannot be ignored, they are taken into account in the simple calculation models by reducing the resistance of the member. For example, the simple calculation models for composite steel and concrete columns consider the effect of induced thermal stresses caused by thermal gradients by applying reduction factors to the design resistance of the member.

THERMAL ANALYSIS

The same calculation rules are given in both BS EN 1993-1-2 and BS EN 1994-1-2 for calculating the temperatures of unprotected and protected steel beams.

The heat transfer to the member is predominantly by two mechanisms; radiation and convection. The member temperature is related to time via a fairly complex differential equation. This is simplified in BS EN 1993-1-2 and BS EN 1994-1-2 by linearising the temperature increments over small time steps. The method is still impractical for hand calculation but is ideal for setting up in spreadsheet software.

The temperatures of the lower and upper flanges may differ considerably, so it is very important that they should be calculated properly in order to obtain an accurate estimate of the bending resistance of the section, especially if a concrete slab is supported on the top flange.

4.1 Unprotected steel sections

For an unprotected steel section, the increase of temperature $\Delta\theta_{a,t}$ in a small time interval Δt (up to 5 seconds) depends on the net amount of heat which the section acquires during this time; the increase is given by 3-1-2/4.2.5 as:

$$\Delta\theta_{a,t} = k_{sh} \frac{1}{c_a \rho_a} \frac{A_m}{V} \dot{h}_{net} \Delta t \quad [\text{K}]$$

where:

\dot{h}_{net}	is the design value of net heat flux per unit area	[W/m ²]
c_a	is the specific heat of steel	[J/kgK]
ρ_a	is the density of steel	[kg/m ³]
$\frac{A_m}{V}$	is the section factor of the member (see 3-1-2/Table 4.3)	[1/m]
k_{sh}	is a correction factor, commonly attributed to the shadow effect of flanges.	

The correction factor k_{sh} reduces the calculated temperature gain for sections whose flanges cause a 'shadow effect' on the inner perimeter areas, in order to achieve better agreement between this calculation method and test results.

For I sections under normal fire conditions

$$k_{sh} = 0.9 \left(\frac{A_m}{V} \right)_b / \left(\frac{A_m}{V} \right)$$

Where $(A_m/V)_b$ is the factor for a notional box around the section ($= 2(h + b)$ for a section exposed on four sides or $= 2h + b$ for a section exposed on three sides).

For all convex shapes, $k_{sh} = 1.0$.

For all other cases:

$$k_{sh} = \left(\frac{A_m}{V} \right)_b / \left(\frac{A_m}{V} \right)$$

4.2 Protected steel sections

For steelwork insulated by fire protection material, the basic mechanisms of heat transfer are identical to those for unprotected steelwork, but the surface covering of material of very low conductivity achieves a considerable reduction in the heating rate of the steel section. Also, the insulating layer itself has the capacity to store a certain amount of heat.

The expression in 3-1-2/4.2.5.2 for the uniform temperature increase of the steel assumes that the exposed insulation surface is at the fire atmosphere temperature (since the conduction away from the surface is low and very little of the incident heat is used in raising the temperature of the surface layer of insulation material).

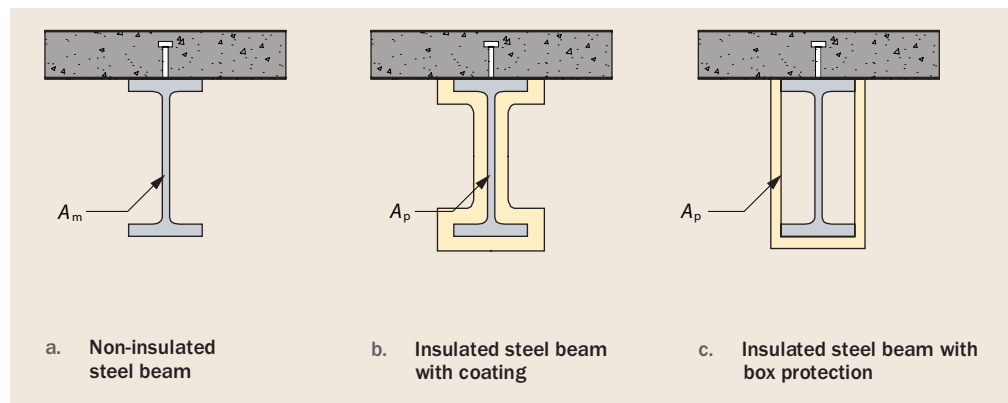


Figure 4.1
Section factors for
unprotected and
protected steel beams

The expression for temperature rise $\Delta\theta_{a,t}$ in a small time increment Δt (up to 30 s) now depends on balancing the heat conduction from the exposed surface with the heat stored in the insulation layer and the steel section. The increase is given in 3-1-2/4.2.5.2 as:

$$\Delta\theta_{a,t} = \frac{\lambda_p / d_p}{c_a \rho_a} \frac{A_p}{V} \left(\frac{1}{1 + \phi / 3} \right) (\theta_{g,t} - \theta_{a,t}) \Delta t - (e^{\phi/10} - 1) \Delta\theta_{g,t} \quad [\text{K}]$$

but $\Delta\theta_{a,t} \geq 0$ if $\Delta\theta_{g,t} > 0$

where:

ϕ	is the relative heat storage in the protection material, given by:	
	$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p \frac{A_p}{V}$	
A_p/V	is the section factor for a protected steel member, with A_p taken as the inner perimeter of the protection material	[m ² /m]
c_a, c_p	are the specific heats of steel and protection materials	[J/kgK]
d_p	is the thickness of fire protection material	[m]
$\theta_{a,t}, \theta_{g,t}$	are steel and gas temperatures at time t	[°C]
$\Delta\theta_{g,t}$	is the increase of gas temperature during the time step Δt	[K]
λ_p	is the thermal conductivity of the fire protection material	[W/mK]
ρ_a, ρ_p	are the densities of steel and fire protection material.	[kg/m ²]

Fire protection materials often contain a certain percentage of moisture; this evaporates at about 100°C, with considerable absorption of latent heat. This causes a 'dwell' in the heating curve for a protected steel member at about this temperature. The incremental temperature-time relationship above does not model this effect (this is a conservative simplification).

4.3 Composite beams

For composite beams where the steel beam has no concrete encasement, the cross section should be divided into parts and the temperature increase for each part determined according to 4-1-2/4.3.4.2.2.

4.3.1 Steel parts of a composite beam

For unprotected sections, the expression for temperature increase is similar to that for steel but with a different shadow factor k_{shadow} and with different values of section factor for each part of the cross section. It is assumed that no heat transfer takes place between the various parts or between the top flange and the concrete slab.

The increase is given by:

$$\Delta\theta_{a,t} = k_{\text{shadow}} \frac{1}{c_a \rho_a} \frac{A_i}{V_i} \dot{h}_{\text{net}} \Delta t \quad [\text{K}]$$

where A_i/V_i is the section factor for the part of the cross section. [m²/m]

For the cross section shown in Figure 4.2, the section factor for the bottom flange is given by:

$$A_i/V_i = \frac{2(b_1 + e_1)}{b_1 e_1}$$

When a composite slab (constructed with profiled steel sheeting, commonly referred to as decking) forms the flange of a composite beam, the concrete slab is not in continuous

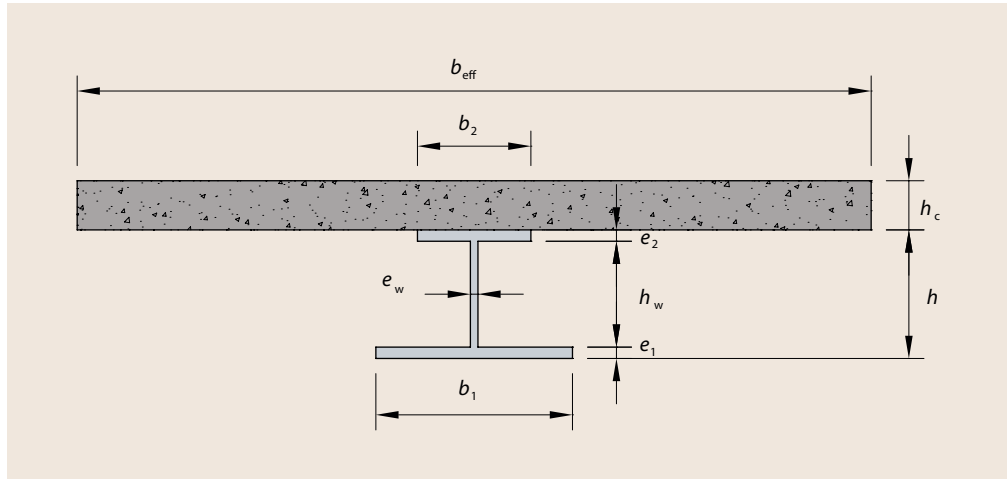


Figure 4.2
Elements of the
cross section

contact with the top surface of the top flange of the steel section (see Figure 4.3). In accordance with 4-1-2/4.3.4.2.2(9), the section factor for the top flange, when at least 85% of the top surface of the top flange is in contact with the composite slab or when any voids formed by the steel profile are filled with non-combustible material, is given by:

$$A_i/V_i = \frac{(b_2 + 2e_2)}{b_2 e_2}$$

where b_2 and e_2 are as defined in Figure 4.2.

For the top flange, when less than 85% of the top surface is in contact with the composite slab and the voids are not filled, the section factor is given by:

$$A_i/V_i = \frac{2(b_2 + e_2)}{b_2 e_2}$$

If the beam depth does not exceed 500 mm, the temperature of the web may be taken as equal to that of the bottom flange.

For protected sections, the expression for temperature increase for the various parts is similar to that given above in Section 4.2 except that the factor for relative heat storage

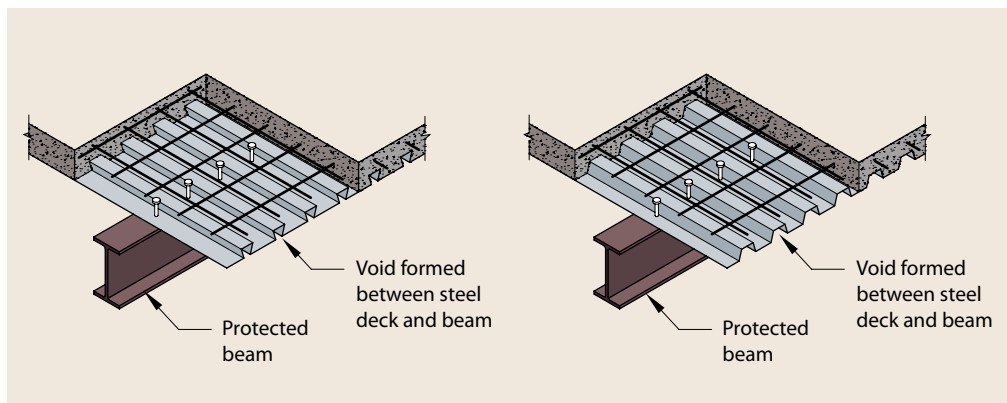


Figure 4.3
Composite beam
and slab using
steel sheeting with
re-entrant and
trapezoidal profiles

(now symbolized as w rather than ϕ) depends on the section factor for the particular part of the cross section.

For members with box-protection, 4-1-2/4.3.4.2.2(11) permits a uniform temperature increase to be assumed for the steel beam. In that case, the section factor is given by:

$$A_p/V = \frac{2h+b}{A}$$

where:

A is the cross sectional area of the steel section

b is the width of the steel section

h is the height of the steel section.

4.3.2 Concrete flange of a composite beam

According to 4-1-2/4.3.4.2.2(15), the temperature of the concrete at a given level in the slab may be assumed to be uniform over the effective width and, the variation of the temperature of the concrete through the depth of the compression zone may be determined on the basis of the tabulated data in 4-1-2/Table D.5.

However, informative Annex D of BS EN 1994-1-2 has been rejected by the UK NA and instead the UK NA refers to NCCI at www.steel-ncci.co.uk. For this purpose, guidance is given in document PN005^[21].

Using PN005, the temperature of the concrete in the compression zone may be determined based on the distance from the soffit of the slab to the level within the zone. If the temperature of all the concrete in the compression zone is below 250°C, no reduction of concrete strength is considered. If some of the concrete in the compression zone is at a temperature above 250°C, the depth and strength of the compression zone can be determined using an iterative process. In this case, the concrete can be divided into a number of strips 10 mm deep, as shown in Figure 4.4. Each strip can be assigned a temperature, based on the distance of its centroid above the soffit of the slab. The temperature of each strip is assumed to be uniform.

No strength reduction is required for the concrete slab if:

$$(h_s - h_u) \geq h_{cr}$$

where:

h_{cr} is determined from PN005 for a limiting temperature of 250°C

h_s is the overall depth of the composite slab

h_u is the depth of the compression zone (for the non-reduced strength of the concrete, to give a compression resistance equal to the tension resistance of the tension zone in the fire situation).

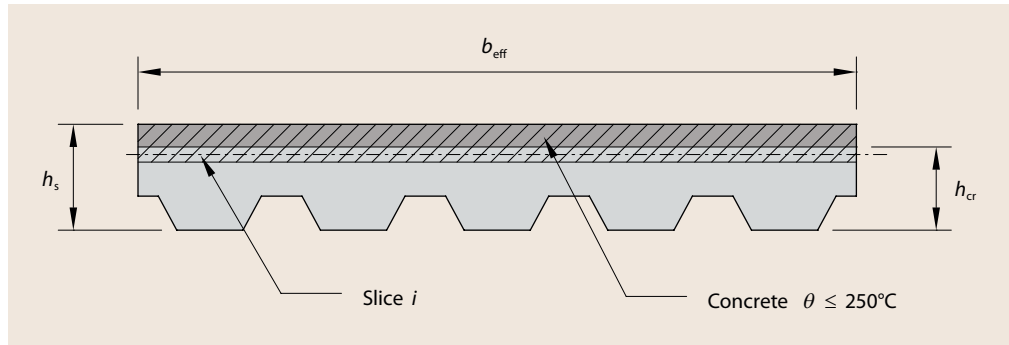


Figure 4.4
Concrete
compression zone

4.3.3 Shear connectors

Clause 4-1-2/4.3.4.2.5 states that the temperature of the shear connector θ_v , may be taken as equal to 80% of the temperature (in $^\circ\text{C}$) of the top flange of the beam and the concrete temperature may be taken as 40% of the temperature of the top flange. These temperatures determine the values of strength reduction factors $k_{u,\theta}$ and $k_{c,\theta}$ that are applied when determining the connector resistance $P_{fi,Rd}$.

MATERIAL PROPERTIES

All materials lose strength at elevated temperatures and, in order to calculate the variation of member resistance with temperature, the strength reduction of the material must be known. It is also important to know how quickly a structural member reaches a temperature at which it will no longer be able to support the loads to which it is exposed in fire conditions. This will require a heat transfer analysis, for which the thermal properties of the material must be known. Often the properties of a material vary with temperature and it is important that information is available for the temperature range that may be experienced in building fires.

The two important thermal properties of materials that will be required for heat transfer analysis are the specific heat and the thermal conductivity, which are defined as follows.

The specific heat of a material is the heat energy required to raise the temperature of a unit mass by 1 K (measured in J/kgK).

The thermal conductivity of a material is the amount of heat energy per second that passes through a unit cross-sectional area of the material for a unit temperature gradient (measured in W/mK).

5.1 Properties of carbon steel

5.1.1 Thermal properties

The properties of thermal expansion, thermal conductivity and specific heat capacity of steel are dependent on steel temperature.

The nominal value of the coefficient of thermal expansion of steel increases from 1.2×10^{-5} per °C at 0°C to 1.5×10^{-5} per °C at 750°C, as given by 3-1-2/3.4.1.1. At a temperature of approximately 730°C, steel undergoes a phase change from ferrite-pearlite to austenite. This phase change results in a denser molecular structure and causes a marked change in expansion characteristics represented by a plateau between 750°C and 860°C, as shown in Figure 5.1. This phase change requires significant amounts of energy causing a significant spike in the specific heat capacity in this temperature range, see Figure 5.2. Above 860°C, further expansion occurs, at a rate of 1.2×10^{-5} per °C.

Expressions for the nominal (characteristic) values of specific heat of steel c_a at elevated temperatures are given in 3-1-2/3.4.1.2 and are shown graphically in

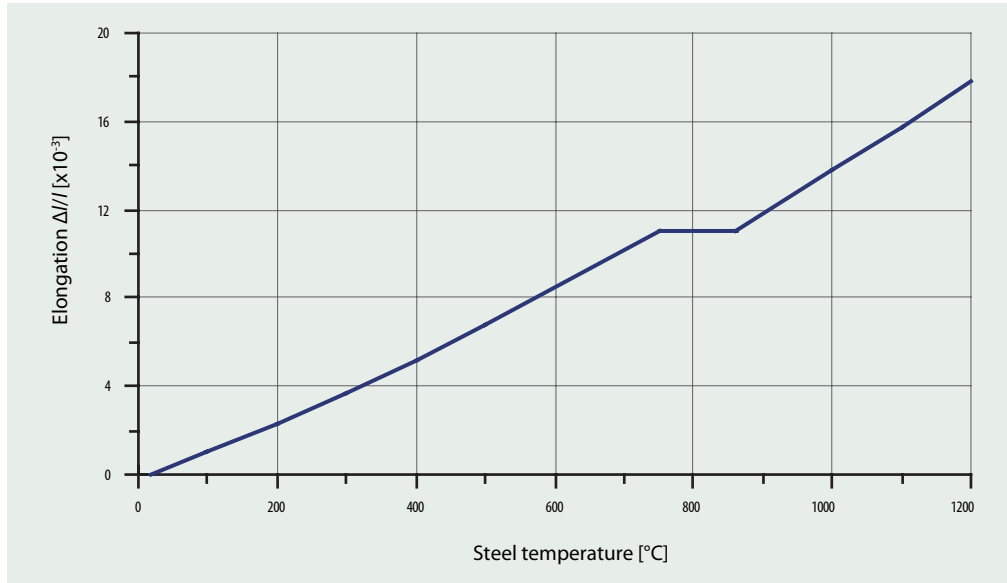


Figure 5.1
Thermal elongation
of carbon steel
as a function of
temperature (from
EN 1993-1-2,
Figure 3.3)

Figure 5.2. For simple calculation models, it is not necessary to consider the variation of specific heat capacity with temperature and 4-1-2/3.3.1(6) recommends a temperature-independent value of 600 J/kg K.

Expressions for the nominal (characteristic) values of thermal conductivity at elevated temperatures λ_a [in W/mK] are given in 3-1-2/3.4.1.3 and are shown diagrammatically in Figure 5.3. For simple calculation models, the thermal conductivity may be considered to be independent of steel temperature; a value of 45 W/mK is recommended for this purpose in 4-1-2/3.3.1(9).

5.1.2 Mechanical properties

The values of strength reduction of carbon steels at elevated temperatures are very similar when normalised with respect to their yield strength at normal temperature. Examples of normalised data obtained from tests are presented in Figure 5.4.

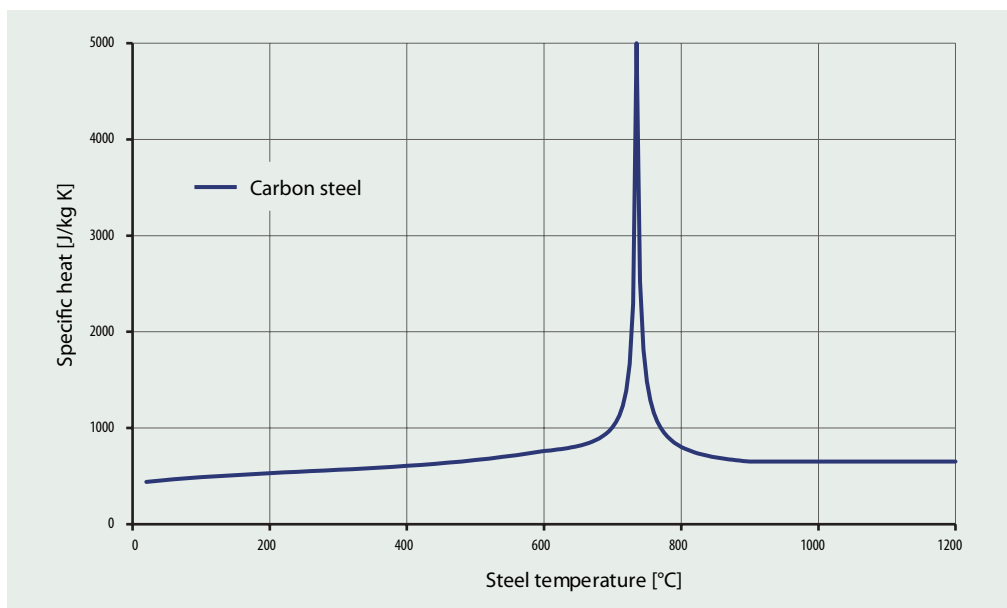


Figure 5.2
Specific heat
of carbon steel
as a function of
temperature

Figure 5.3
Thermal conductivity
of carbon steel
as a function of
temperature (from
EN 1993-1-2,
Figure 3.5)

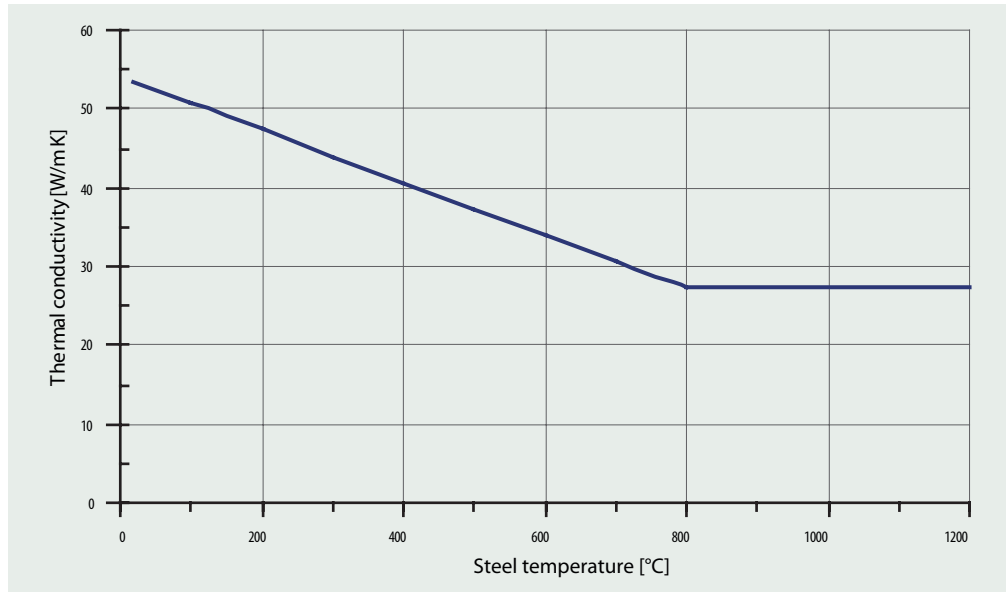
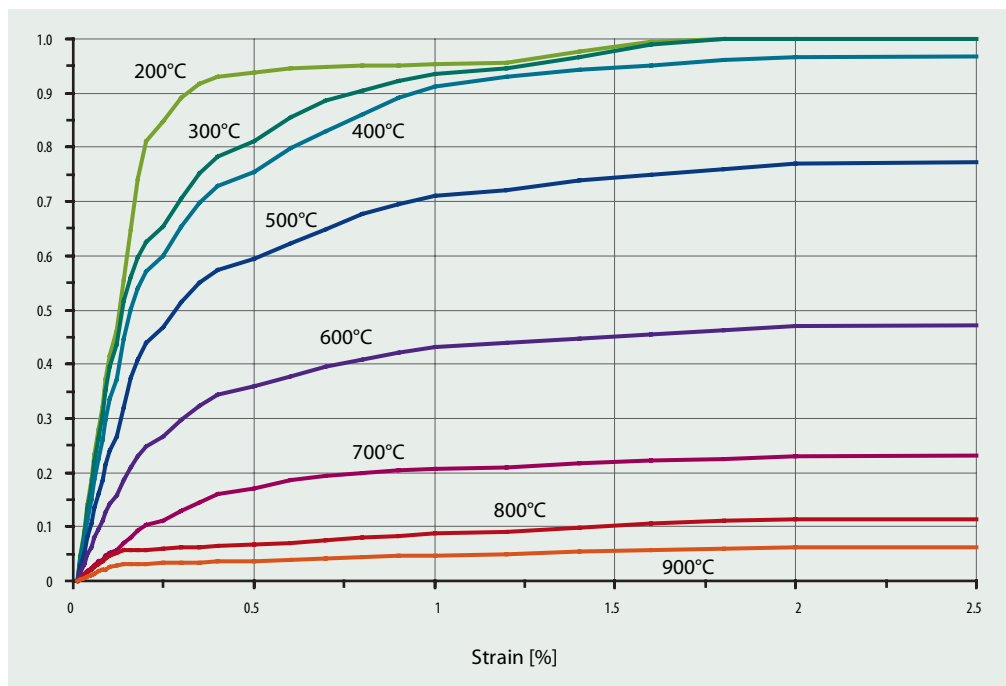


Figure 5.4
Normalised stress-
strain curves for
steel at elevated
temperature (as
measured in tests).



The shape of the stress-strain curve for carbon steel at elevated temperature differs from that at normal temperature. Most notable is the lack of a defined yield point.

The strength of steel at elevated temperature becomes dependant on the strain limit considered: in BS EN 1993-1-2 and BS EN 1994-1-2 a strain limit of 2% is adopted, which is reasonable, as fire tests have shown that steel members can have strains in excess of 3% when they reach the limiting deflection. However, for Class 4 sections, where local buckling could limit the strain achieved by the steel section, Annex C gives strength reduction factors based on a strain limit of 0.5%, resulting in much lower strengths.

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. For this reason,

the stress-strain data on which the recommendations of BS EN 1993-1-2 and BS EN 1994-1-2 are based has been obtained from anisothermal testing at different heating rates. The stress-strain data is therefore considered to include the effects of creep deformation. The recommended values are appropriate for heating rates between 2 and 50 K/min.

BS EN 1993-1-2 provides values of material properties at elevated temperature for carbon steel of grades S235 to S460, for steels in accordance with BS EN 10025^[22], BS EN 10210^[23] and BS EN 10219^[24].

In 3-1-2/3.2.1, the stress-strain relationship at elevated temperatures is idealised, as shown in Figure 5.5.

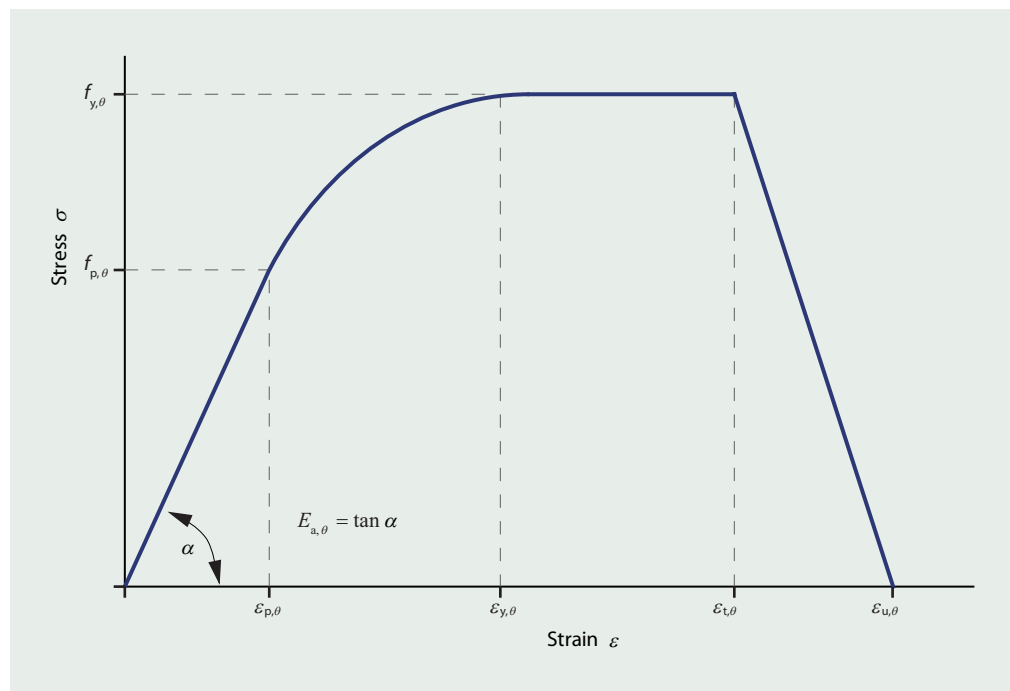


Figure 5.5
Stress-strain
relationship for
carbon steel
at elevated
temperatures

Particular values to note in this Figure are:

- $f_{y,\theta}$ is the effective yield strength at elevated temperature θ
- $f_{p,\theta}$ is the proportional limit at elevated temperature θ
- $E_{a,\theta}$ is the slope of the linear elastic range at elevated temperature θ
- $\epsilon_{p,\theta}$ is the strain at the proportional limit at elevated temperature θ
- $\epsilon_{y,\theta}$ is the yield strain at elevated temperature θ
- $\epsilon_{t,\theta}$ is the limiting strain for yield strength at elevated temperature θ
- $\epsilon_{u,\theta}$ is the ultimate strain at elevated temperature θ .

The Figure shows that the first part of the relationship is linear up to the proportional limit $f_{p,\theta}$ and the elastic modulus $E_{a,\theta}$ is equal to the slope of this segment. The second part depicts the transition from the elastic to the plastic range. This region is formulated by an elliptical progression up to the effective yield strength $f_{y,\theta}$. The third part of the curve is a flat yield plateau up to a limiting strain. The last part of the curve is characterised by a linear reduction to zero stress at the ultimate strain.

Table 3.1 of 3-1-2 gives expressions for $f_{y,\theta}$, $f_{p,\theta}$ and $E_{a,\theta}$ in terms of reduction factors applied to the nominal values at normal temperature (which are given in BS EN 1993-1-1); these are shown diagrammatically in Figure 5.6. Note that these factors are applicable for all grades of carbon steel, for steels to BS EN 10025, BS EN 10210 and BS EN 10219.

3-1-2/Figure 3.1 gives values of $\varepsilon_{y,\theta} = 0.02$, $\varepsilon_{t,\theta} = 0.15$ and $\varepsilon_{u,\theta} = 0.2$ at all temperatures. The effect of creep is implicitly included in the values and the values are applicable for heating rates between 2 and 50 K/min.

For Class 4 sections, reduction factors are given in 3-1-2/Annex E, which covers hot-rolled and cold formed sections (see Section 5.3). The UK NA adopts this informative Annex.

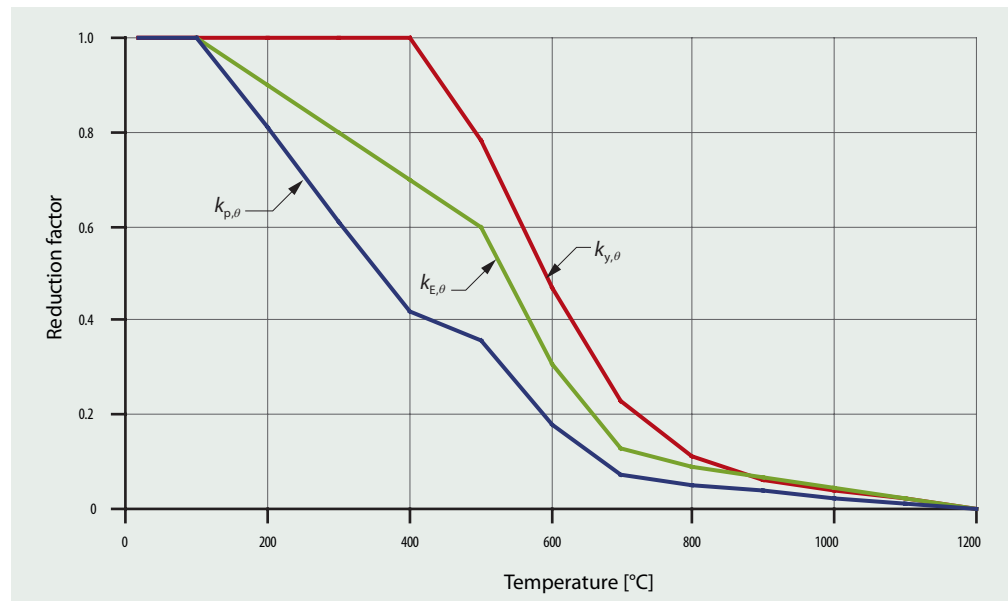


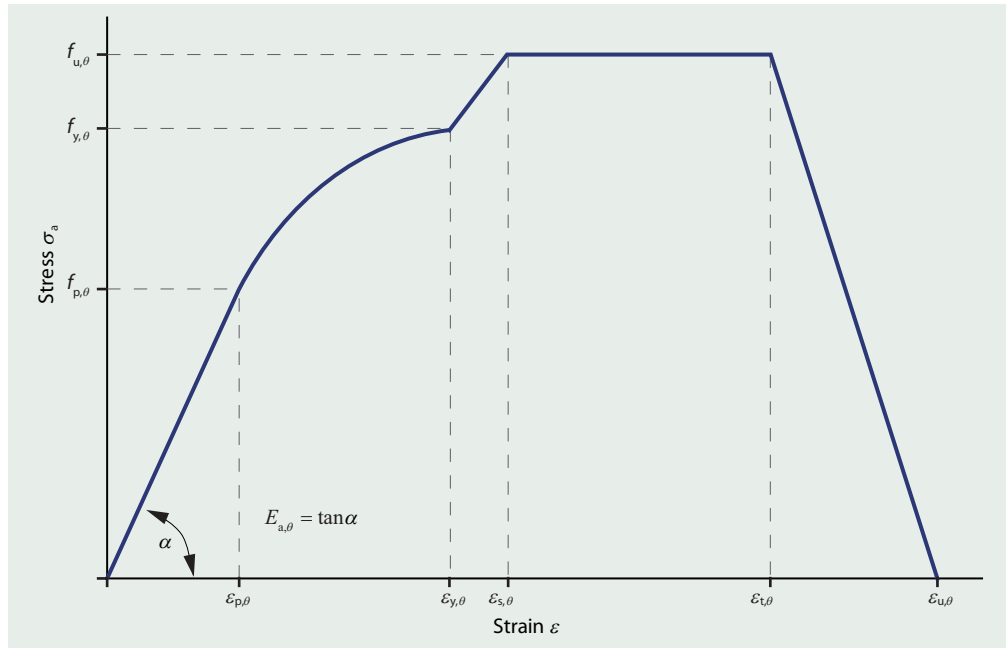
Figure 5.6
Reduction factors
for stress-strain
relationship of carbon
steel at elevated
temperatures

Figure 5.6 shows that carbon steel becomes inelastic at temperatures above 100°C and for a strain limit of 2% the yield strength reduces for temperatures above 400°C. The reduction in strength continues at a fairly steady rate up to 800°C. The well-defined yield plateau at normal temperatures (characterised by $k_{p,\theta} = 1.0$) is replaced by a gradual increase of strength with increasing strain, at elevated temperatures (where a 2% strain limit is used to define the yield point).

Comparing these reduction factors, it can be seen that the stiffness of steel reduces more rapidly than the strength. This indicates that the failure mode of steel members may change at elevated temperatures. For instance, a steel beam comprising a slender I section that is designed for plastic failure at ULS at normal temperature may experience premature failure due to web buckling at elevated temperatures.

3-1-2/3.2.1 allows the option of an alternative stress-strain relationship for steel temperatures below 400°C that includes the effects of strain-hardening, provided that local or overall buckling does not lead to premature collapse. The idealised stress-strain relationship including strain hardening is shown in Figure 5.7 and the strain-hardened values are given in 3-1-2/Annex A.

Figure 5.7
Stress-strain
relationship for carbon
steel at elevated
temperatures allowing
for strain hardening



5.2 Stainless steel

The grades of stainless steel covered by BS EN 1993-1-4 include a wide range of corrosion and heat-resistant iron-based materials containing at least 10% chromium, a maximum 1.2% carbon and other alloying elements. There are five basic groups of stainless steel, classified according to their metallurgical structure, namely austenitic, ferritic, austenitic-ferritic (duplex), martensitic and precipitation-hardening groups. BS EN 1993-1-4 covers only the first three groups; it covers steels to BS EN 10088^[25]. Austenitic and duplex stainless steels are the most widely used in structures, mainly due to their good weldability.

BS EN 1993-1-2, Annex C provides characteristic values of mechanical properties at elevated temperatures, for stainless steel Grades 1.4301, 1.4401, 1.4571, 1.4003 and 1.4462 and characteristic values of thermal properties that may be used for all grades of stainless steel. For grades of stainless steel other than those listed, the Annex says that the mechanical properties may be taken as those for carbon steel.

5.2.1 Thermal properties

The thermal properties of stainless steel are quite different from those of carbon steel. The magnitude of thermal expansion of stainless steel is greater than the thermal expansion of carbon steel. As there is no phase change in stainless steel at elevated temperature the rate of thermal expansion of stainless steel remains relatively constant up to 1200°C; there is no plateau as for carbon steel. The specific heat of stainless steel is relatively constant, increasing only slightly with elevated temperatures.

At normal temperature, stainless steel has a much lower thermal conductivity than carbon steel. However, the thermal conductivity of stainless steel gradually increases with elevated temperatures and exceeds the value for carbon steel above 1000°C.

The comparisons between values for carbon and stainless steels are shown in Figure 5.8, Figure 5.9 and Figure 5.10.

Figure 5.8
Comparison between the thermal elongation of stainless steel and carbon steel as a function of temperature

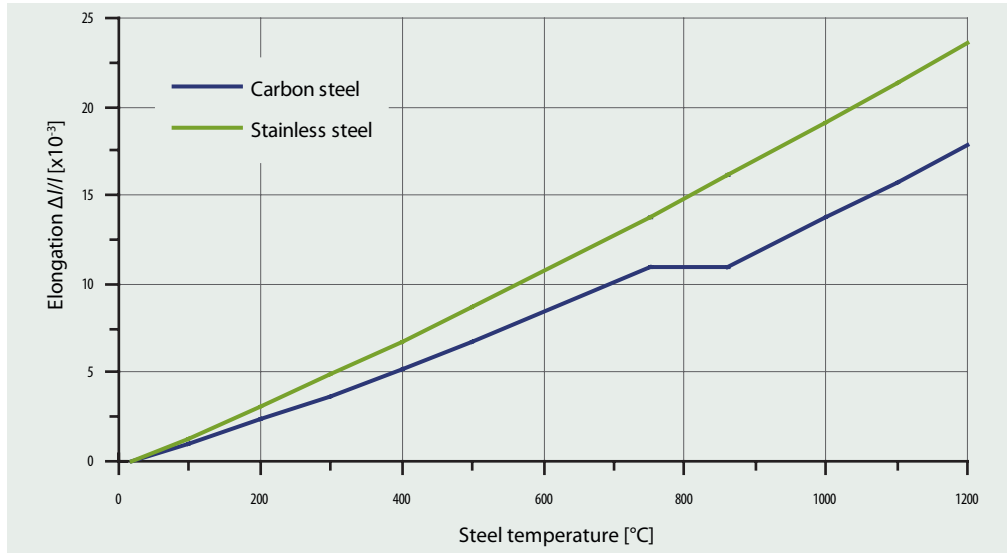


Figure 5.9
Comparison between the specific heat of stainless steel and carbon steel as a function of temperature

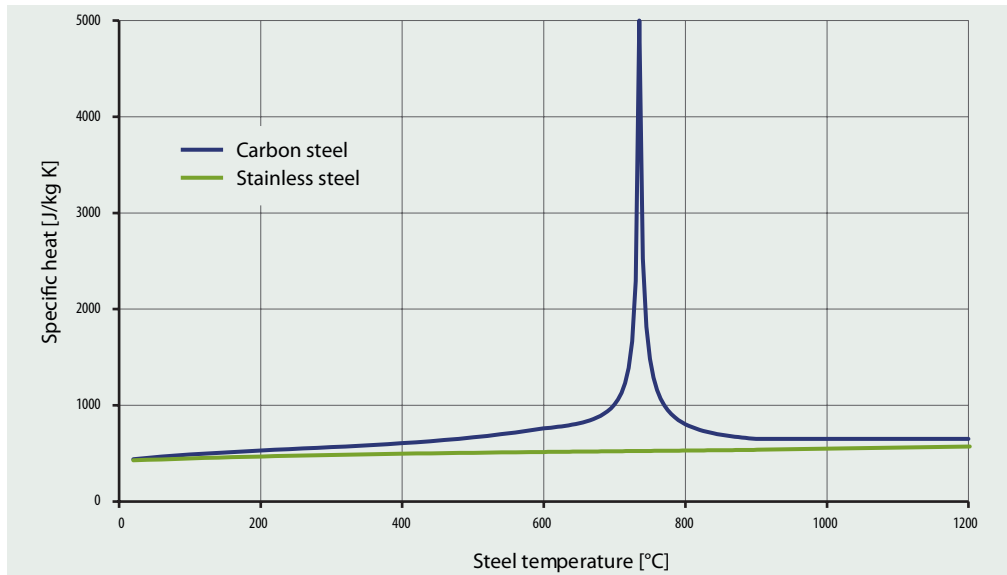
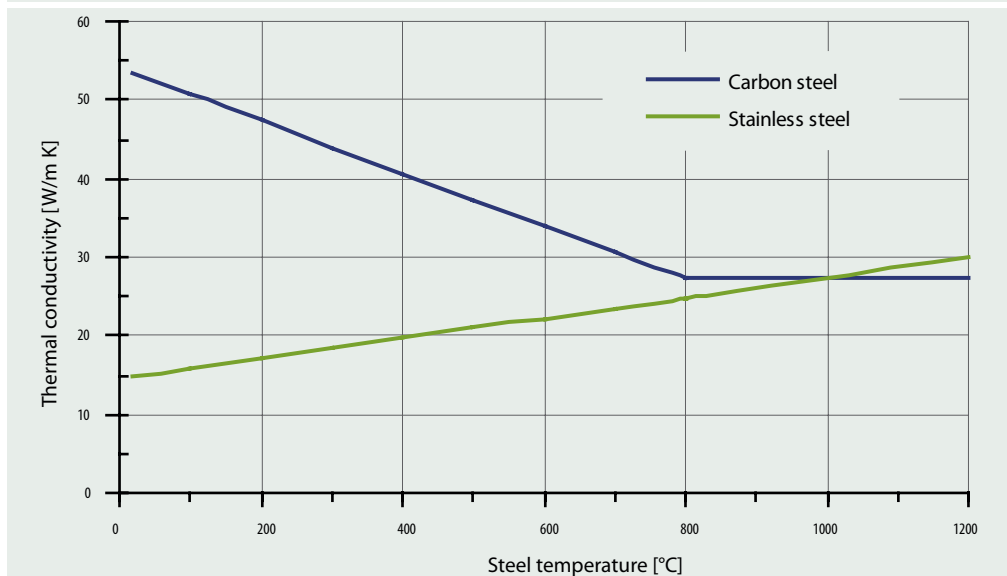


Figure 5.10
Comparison between the thermal conductivity of stainless steel and carbon steel as a function of temperature



5.2.2 Mechanical properties

For stainless steel grades 1.4301, 1.4401, 1.4571, 1.4003 and 1.4462, the stress-strain relationship at elevated temperatures is characterised diagrammatically in 3-1-2/Figure C.1, as shown in Figure 5.11.

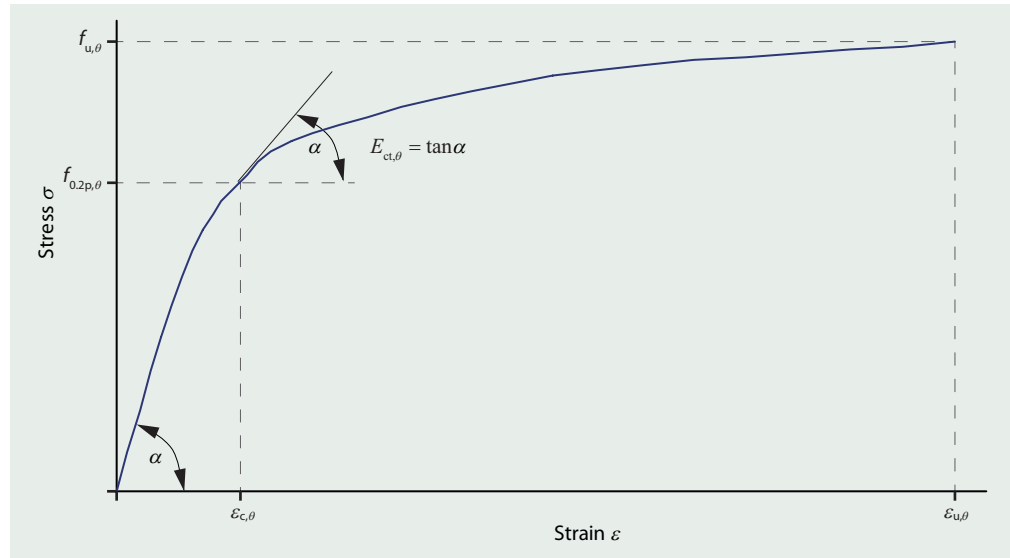


Figure 5.11
Stress-strain relationship for stainless steel at elevated temperatures

The key values in this Figure are:

- $f_{u,θ}$ is the tensile strength at elevated temperature $θ$
- $f_{0.2p,θ}$ is the proof strength at 0.2% plastic strain at elevated temperature $θ$
- $E_{a,θ}$ is the slope of the linear elastic range at elevated temperature $θ$
- $E_{ct,θ}$ is the slope (tangent) at proof strength at elevated temperature $θ$
- $ε_{c,θ}$ is the total strain at proof strength at elevated temperature $θ$
- $ε_{u,θ}$ is the ultimate strain at elevated temperature $θ$.

As for carbon steel, values of key parameters are given in terms of reduction factors applied to characteristic values at normal temperature (3-1-2/Table C.1) and the values are applicable for heating rates between 2 and 50 K/min.

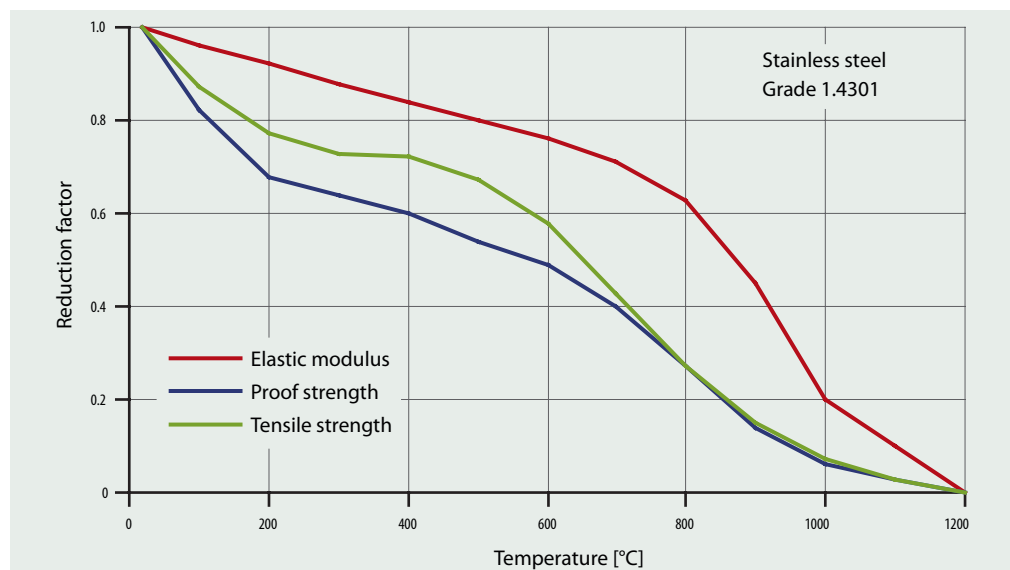


Figure 5.12
Reduction factors for stress-strain relationship of stainless steel Grade 1.4301 at elevated temperatures

For use in simple calculation methods 3-1-2/C.2.1 gives an expression for the determination of nominal yield strength at elevated temperature; the value is between the values of proof strength $f_{0.2p,\theta}$ and tensile strength $f_{u,\theta}$.

Figure 5.12 illustrates the variation of the reduction factors for Grade 1.4301 stainless steel. Reduction factors for all the above grades are in 3-1-2/Table C.1.

5.3 Cold formed steel members

The design of cold formed members at normal temperatures is covered by BS EN 1993-1-3; that Part covers a range of steel grades and product standards, in addition to grades to BS EN 10025. Traditionally, 'light steel sections', such as cold-formed C or Z shapes, are commonly used as roof purlins and side rails; the advantage of these sections is a high strength-to-weight ratio, and this is achieved by the use of Class 4 sections. BS EN 1993-1-2 treats all Class 4 sections in a similar manner regardless of whether they are hot-rolled, welded or cold formed. The yield strength in fire conditions is based on the 0.2% proof stress of the material. This takes account of the tendency for these section types to be subject to local buckling when subjected to high strains.

The influence of cold working on the material strength of cold formed sections should be relatively small. Comparing hot rolled and cold formed steels, an additional reduction in the strength of cold formed steel would be noticed in the temperature range 400°C to 600°C.

Although widely used in the UK, the performance of light gauge steel in fire is only briefly described in Eurocode 3. The only modification to material properties is in the determination of yield strength; the value to be used in design is given by 3-1-2/Annex E. All other properties, thermal and mechanical are as for carbon steel.

The variation of yield strength with temperature is given in terms of a reduction factor $k_{p0.2,\theta}$ and this is illustrated in Figure 5.13.

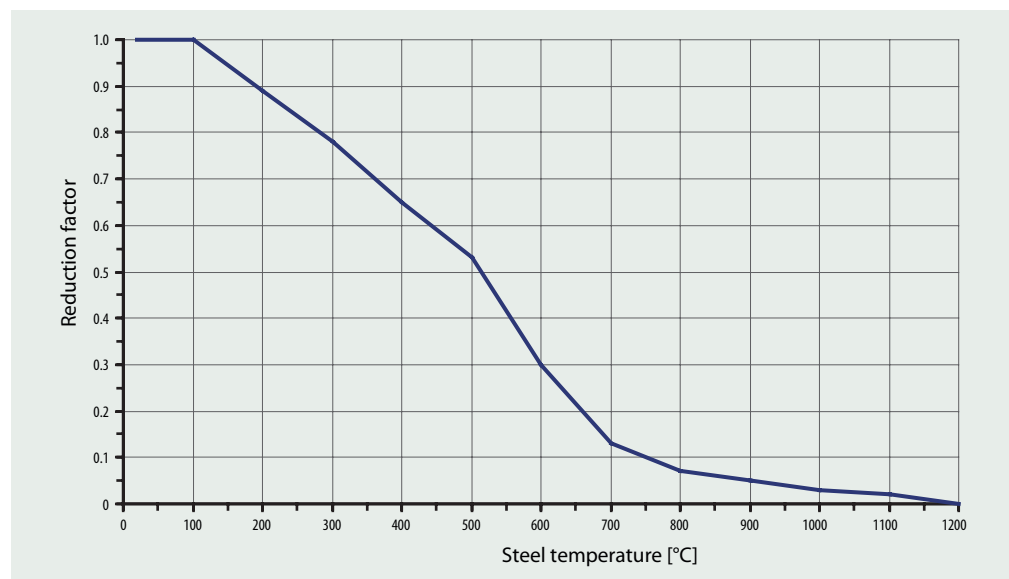


Figure 5.13
Strength reduction
factor $k_{p0.2,\theta}$ for
cold-formed
steel at elevated
temperatures

5.4 Concrete

Normal weight concrete can be composed of either siliceous or calcareous aggregate; the different aggregates result in slightly different properties at elevated temperature. The properties given in BS EN 1994-1-2 are for siliceous aggregate; references are made to BS EN 1992-1-2 for properties for calcareous aggregate. Lightweight concrete used in structural applications consists of pelletized aggregate made from sintered fuel ash or expanded clay.

In some cases, a simplified relationship is given for some properties, for use with the simple calculation model in BS EN 1994-1-2 (see Section 7).

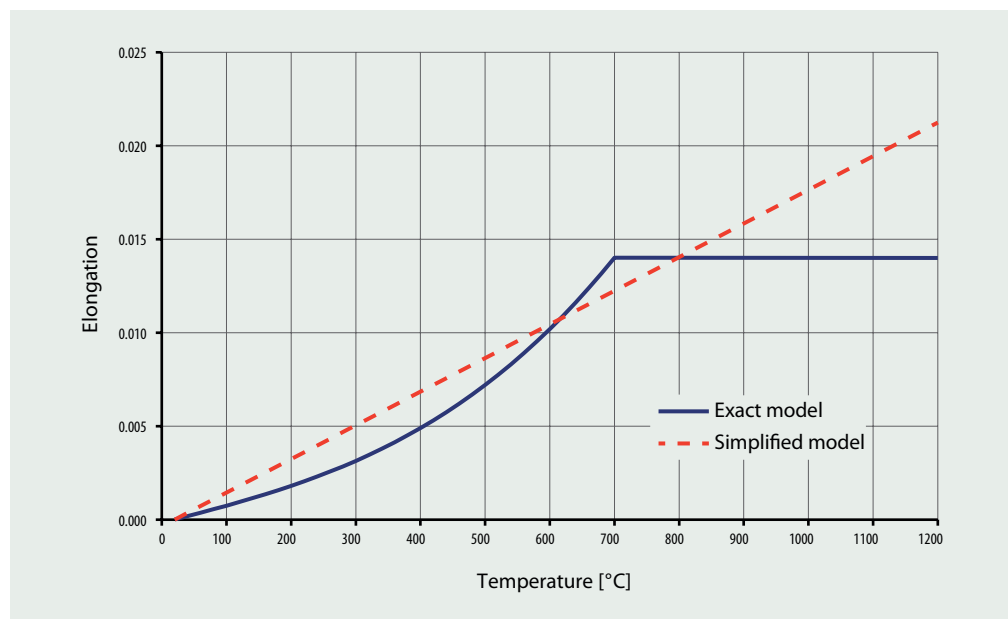
BS EN 1994-1-2 does not cover the design of composite structures with concrete strength classes lower than C20/25 and LC20/22 or higher than C50/60 and LC50/55. However, 4-1-2/1.1(16) notes that information is given in 2-1-2/6 for strengths higher than C50/60 and 4-1-2/NA.2.2 permits the use of 2-1-2/6.

5.4.1 Thermal properties of normal weight concrete

The thermal elongation of normal weight concrete with siliceous aggregates is given in 4-1-2/3.3.2. For calcareous aggregate concrete, reference is made to 2-1-2/3.3.1(1). In simple calculation models, the thermal elongation may be considered to vary linearly with temperature. Figure 5.14 shows the thermal elongation of normal weight concrete according to both the 'exact' relationship and that for the simple calculation model.

The specific heat of dry siliceous and calcareous concrete $c_{c,\theta}$ is given in BS 4-1-2/3.3.2(4). For simple calculation models, the specific heat may be considered to be independent of concrete temperature, with an assumed value of 1000 J/kgK. Where the moisture content u is not considered explicitly in analysis, the specific heat of concrete may be modified by adding a peak value at 115°C. At that temperature, the value for dry

Figure 5.14
Thermal elongation
of normal weight
siliceous aggregate
concrete as
a function of
temperature



concrete is 915 J/kgK (given by the expressions in 4-1-2/3.3.2) and for the following moisture contents, the peak values are:

$$c_c^* = 1470 \text{ [J/kgK]} \quad \text{for } u = 1.5\% \quad \text{(this value has been interpolated between 915 and 2020 J/kgK)}$$

$$c_c^* = 2020 \text{ [J/kgK]} \quad \text{for } u = 3.0\%$$

$$c_c^* = 5600 \text{ [J/kgK]} \quad \text{for } u = 10.0\%$$

For other moisture contents, linear interpolation may be used between the above values. A 10% moisture content may occur for hollow sections filled with concrete. The peak values of specific heat are shown in Figure 5.15.

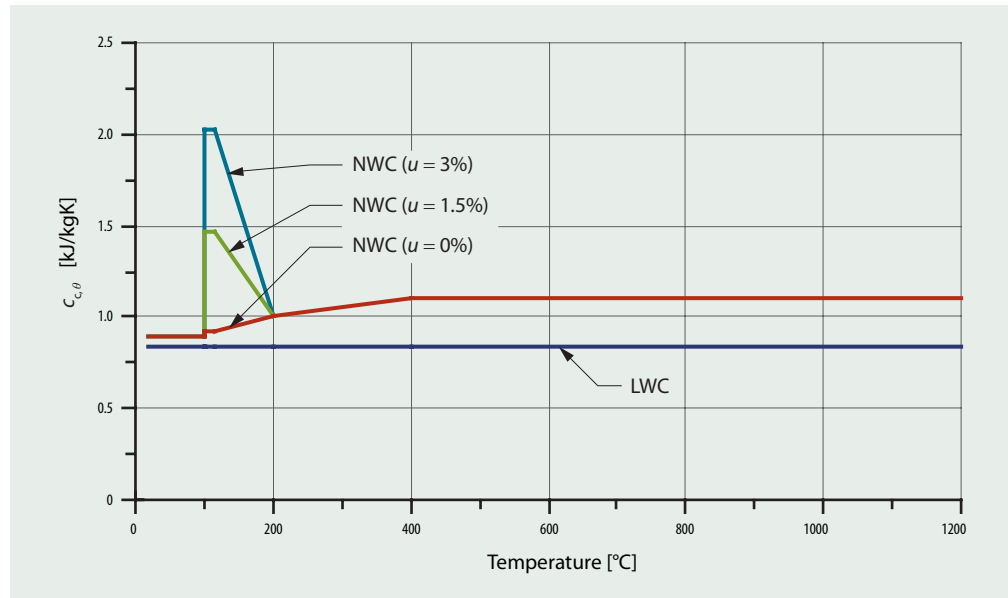


Figure 5.15
Specific heat
of concrete as
a function of
temperature

4-1-2/3.3.2 states that the thermal conductivity of concrete λ_c can be determined between lower and upper limiting values, given by:

Upper limit:

$$\lambda_c = 2 - 0.2451(\theta_c / 100) + 0.0107(\theta_c / 100)^2 \quad \text{[W/mK]}$$

Lower limit:

$$\lambda_c = 1.36 - 0.136(\theta_c / 100) + 0.0057(\theta_c / 100)^2 \quad \text{[W/mK]}$$

Both expressions are valid for θ_c between 20°C and 1200°C.

The variation of the upper and lower limits of thermal conductivity with temperature is shown in Figure 5.16. The UK NA (4-1-2/NA.2.7) recommends the use of the upper limit values in all cases.

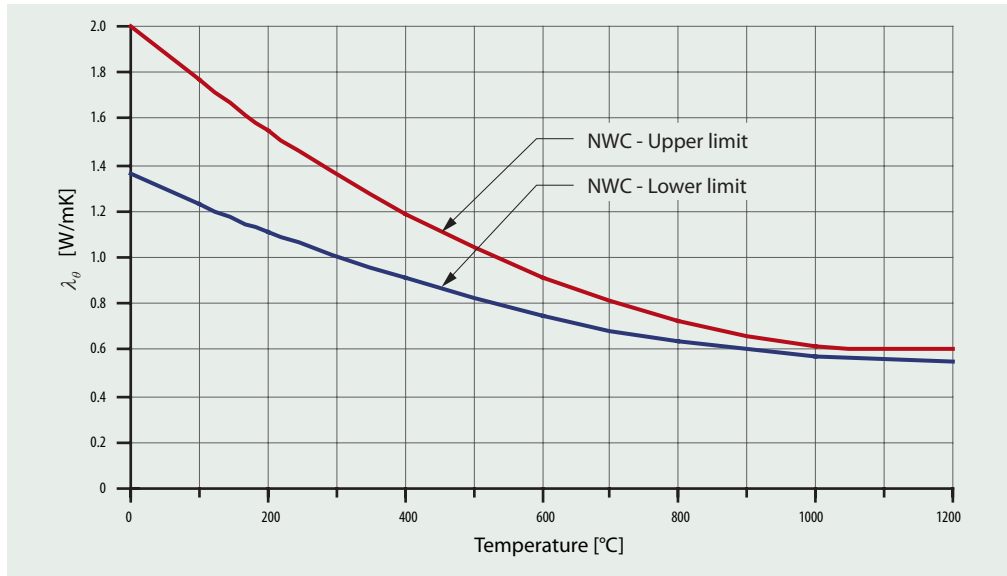


Figure 5.16
Thermal conductivity
of concrete as
a function of
temperature

The variation of density of concrete $\rho_{c,\theta}$ with temperature is influenced by free water loss. 4-1-2/3.4 states that, for static loads, the density of concrete may be considered independent of temperature. The density of unreinforced normal-weight concrete may be taken as 2300 kg/m³.

However, for the thermal analysis, the variation of density may be considered using the guidance given in 2-1-2/3.3.2(3). Alternatively, the variation of density may be approximated by:

$$\rho_{c,\theta} = 2354 - 23.47(\theta_c / 100) \quad [\text{kg/m}^3]$$

where θ_c is the concrete temperature (°C).

5.4.2 Mechanical properties of normal weight concrete

The mechanical properties given in 4-1-2/3.2.2 are valid for heating rates between 2 and 50 K/min.

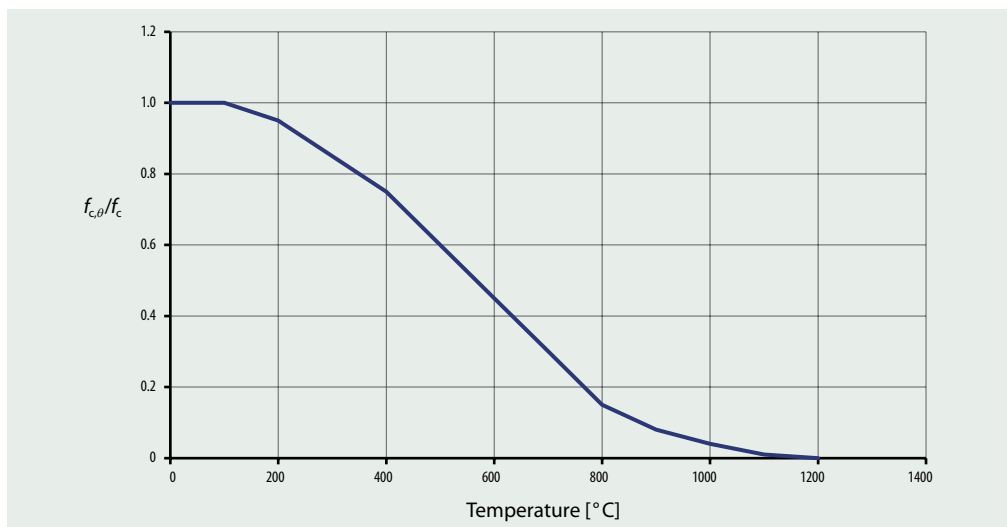


Figure 5.17
Reduction of strength
for normal-weight
concrete at elevated
temperatures

Strength reduction factors and strain at peak stress are given for siliceous aggregate concrete are given in 4-1-2/Table 3.3. The values may also be used for calcareous aggregate concrete or reference may be made to 2-1-2/Table 3.1. The values of $k_{c,\theta}$ for normal weight concrete are shown in Figure 5.17.

The stress-strain relationship for concrete under uniaxial compression at elevated temperatures is given by:

$$\sigma_{c,\theta} = \frac{3\varepsilon_{c,\theta} f_{c,\theta}}{\varepsilon_{cu,\theta} \left(2 + \left(\varepsilon_{c,\theta} / \varepsilon_{cu,\theta} \right)^3 \right)} \quad \text{For } \varepsilon \leq \varepsilon_{cu,\theta}$$

where:

- $f_{c,\theta}$ is the concrete compressive strength at temperature $\theta_c (= k_{c,\theta} f_c)$
- f_c is the concrete compressive strength at normal temperature
- ε is the concrete strain
- $\varepsilon_{cu,\theta}$ is the concrete strain corresponding to $f_{c,\theta}$
- θ_c is the concrete temperature [°C]
- $\sigma_{c,\theta}$ is the concrete stress.

For $\varepsilon > \varepsilon_{cu,\theta}$, a descending branch, extending to zero stress at a strain $\varepsilon_{ce,\theta}$ (given by 4-1-2/Annex B) may be required for numerical purposes. Either a linear or non-linear model descending branch is permitted ($\varepsilon_{ce,\theta}$ is given by 4-1-2/Annex B).

The relationship is illustrated in Figure 5.18.

Figure 5.19 shows the stress-strain curves for normal weight siliceous concrete at a range of elevated temperatures in accordance with the relationship given above. It can be seen that the peak compressive strength $f_{c,\theta}$ reduces and the corresponding strain increases with increasing temperature.

For advanced calculation models, using natural fires, the stress-strain relationship given in 4-1-2/Annex C should be used.

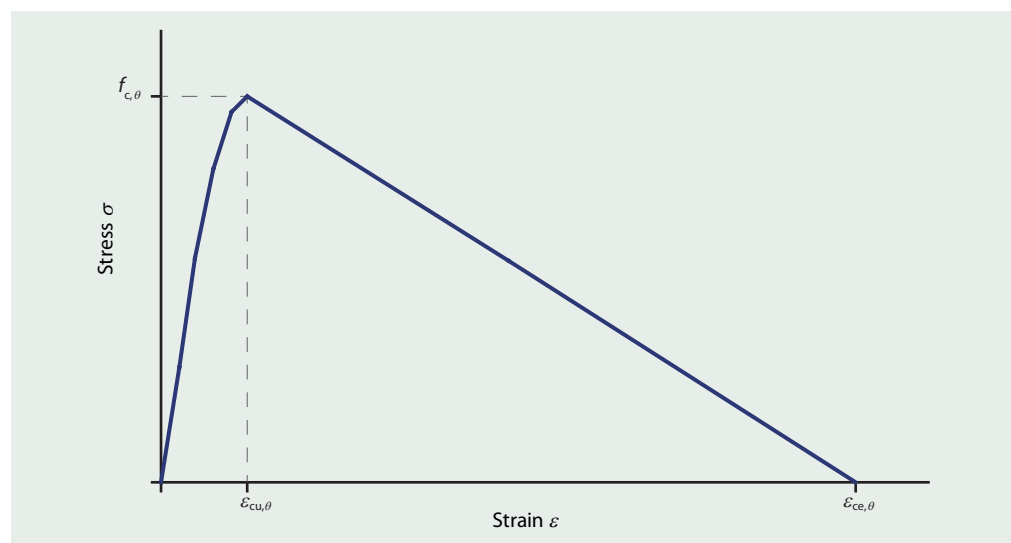


Figure 5.18
Stress-strain
relationship of
concrete under
compression at
elevated temperatures
(assuming linear
descending branch)

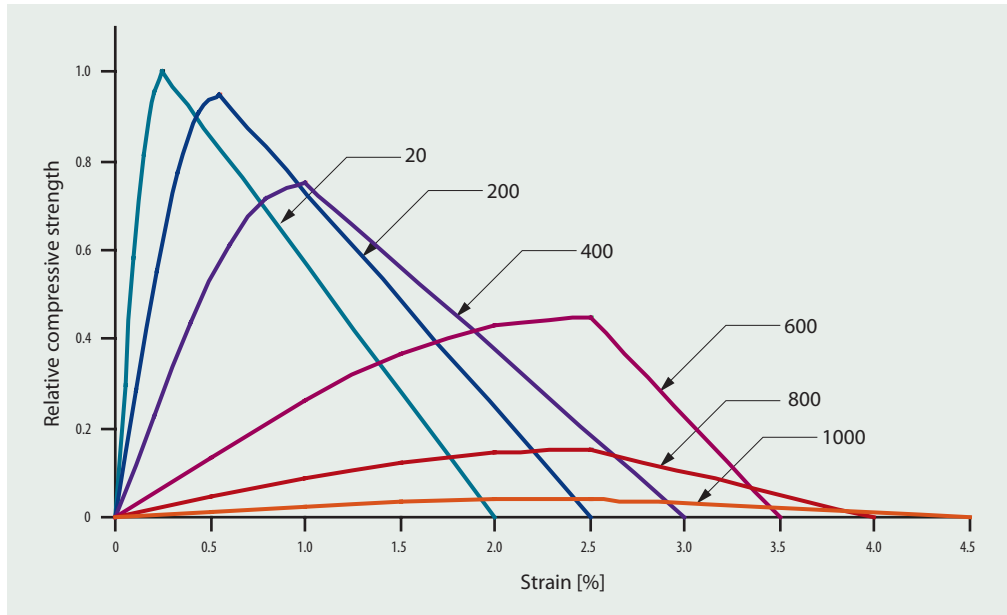


Figure 5.19
Stress-strain curves for siliceous normal weight concrete at elevated temperatures

Conservatively, the tensile strength of concrete may be ignored. However, 4-1-2/3.2.2(9) allows the tensile strength to be taken into account in an advanced calculation model, by referring to the guidance given in BS EN 1992-1-2.

5.4.3 Thermal properties of lightweight concrete

Lightweight concrete has lower thermal expansion and thermal conductivity than normal weight concrete. 4-1-2/3.3.3 provides a simple linear relationship between elongation and temperature for lightweight concrete, as shown in Figure 5.20.

The specific heat c_c of lightweight concrete may be considered to be independent of the concrete temperature and is given by $c_c = 840 \text{ J/kgK}$.

The thermal conductivity λ_c of lightweight concrete may be determined from the following:

$$\lambda_c = 1.0 - (\theta_c / 1600) \quad [\text{W/mK}] \quad \text{for } 0^\circ\text{C} \leq \theta_c \leq 800^\circ\text{C}$$

$$\lambda_c = 0.5 \quad [\text{W/mK}] \quad \text{for } \theta_c > 800^\circ\text{C}$$

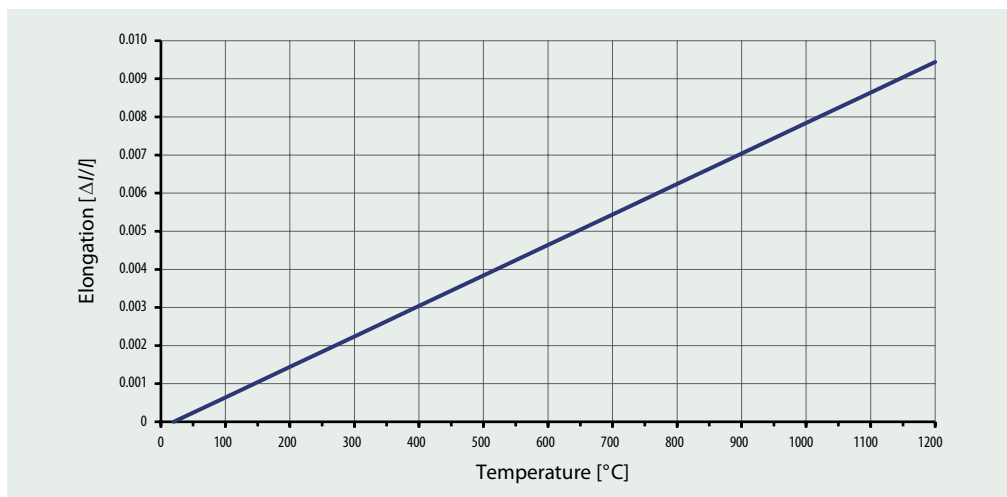


Figure 5.20
Thermal strain of lightweight concrete as a function of temperature

5.4.4 Mechanical properties of lightweight concrete

For lightweight concrete, BS EN 1994-1-2 adopts the same stress-strain model used for normal-weight concrete, although the strength reduction factor is different; values of $k_{c,\theta}$ for lightweight concrete are shown in Figure 5.21. Lightweight concrete has a better strength reduction factor (i.e. greater) than normal-weight concrete.

No values of the strain $\varepsilon_{cu,\theta}$ corresponding to $f_{c,\theta}$ are given in 4-1-2/3.2.2: it recommends that values be obtained from tests.

For advanced calculation models, using natural fires, the stress-strain relationship for lightweight concrete given in 4-1-2/Annex C should be used.

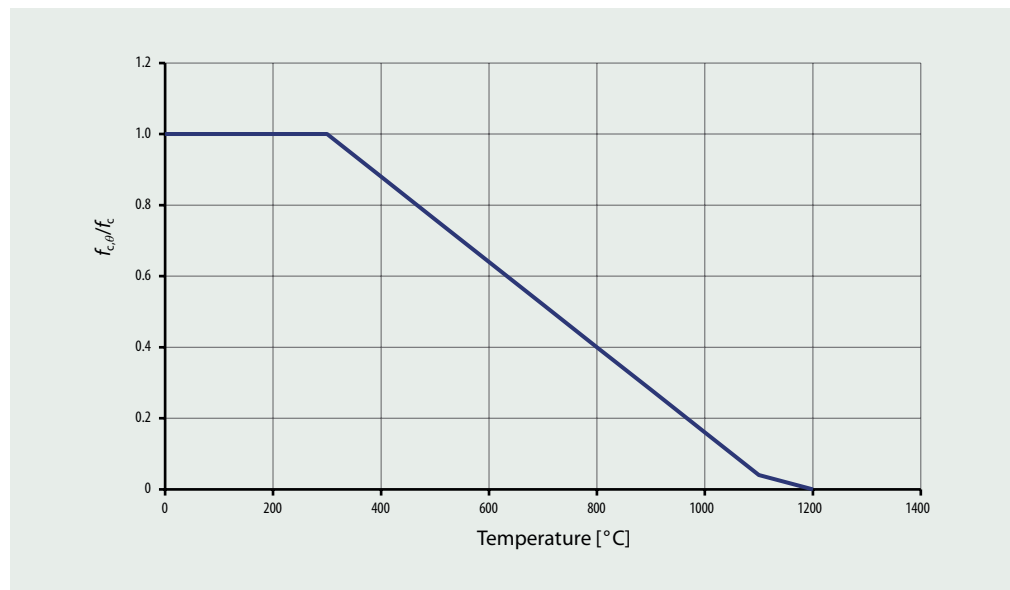


Figure 5.21
Reduction of strength
for lightweight
concrete at elevated
temperatures

5.5 Reinforcing steels

BS EN 1994-1-2 covers hot-rolled and cold worked reinforcing steels. No rules are given for prestressed parts of composite structures.

The thermal and mechanical properties of hot-rolled reinforcing steel are assumed to be the same as for structural steel, except that the strain limit for the yield 'plateau' is lower (see 4-1-2/3.2.3(2)). For cold worked reinforcing steel, the thermal properties are assumed to be the same as hot-rolled steel but the mechanical properties are different – see 4-1-2/Table 3.4. The variation of the stiffness, extent of the proportional limit, and the yield strength, with temperature, are shown in Figure 5.22.

5.6 Bolts and welds

Annex D of BS EN 1993-1-2 is rejected by the UK NA; instead it recommends alternative NCCI on www.steel-ncci.co.uk (see document PN003, which provides limited information on the fire performance of bolts and welds, comprising mechanical properties and design temperatures for the connection components).

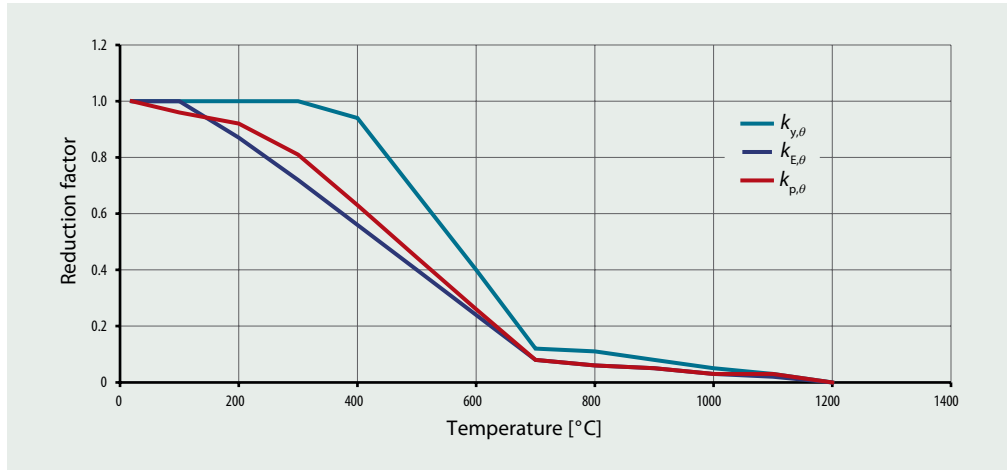


Figure 5.22
Reduction factors for reinforcing steels

Based on limited amount of tests, the same strength reduction factor $k_{b,\theta}$ is given for bolts in shear and tension, regardless of bolt type. For preloaded bolts, it is assumed that the bolts slip in fire and the fire resistance of an individual bolt is determined as for bearing type bolts. The strength reduction factor is given by 3-1-2/Table D.1 and the variation of the strength reduction factor with temperature is shown in Figure 5.23.

The reduction factor $k_{w,\theta}$ for fillet welds is given by 3-1-2/Table D.1 and the variation of strength reduction factor with temperature is illustrated in Figure 5.23. Fillet welds are considered to have better fire performance than bolts, but have a lower strength reduction compared to butt welds or the parent metal.

The strength of a full penetration butt weld, for temperatures up to 700°C, should be taken as equal to the strength of the connecting members, using the appropriate reduction factors for structural steel. For temperatures greater than 700°C, the reduction factors given for fillet welds can also be applied to butt welds.

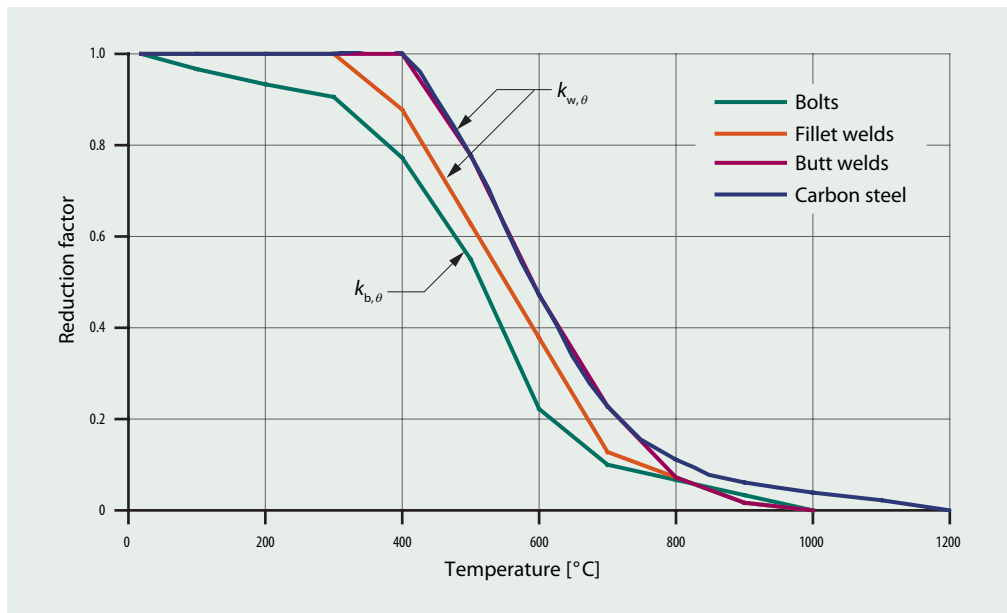


Figure 5.23
Strength reduction factors for bolts and welds at elevated temperatures

STEEL MEMBERS – SIMPLE CALCULATION MODELS

BS EN 1993-1-2 permits the fire resistance to be determined either by simple calculation models or by advanced calculation models that are based on fundamental physical behaviour. Detailed rules are provided for the simplified approach and these rules are discussed below.

With a simple calculation model, verification of load-bearing resistance may be carried out either in the time domain (such that $E_{fi,d} \leq R_{fi,d,t}$ is satisfied for the situation at time t) or in the temperature domain (such that $E_{fi,d} \leq R_{fi,d,\theta}$ is satisfied for a uniform temperature θ).

6.1 Partial factor on material properties

The partial factor for mechanical material properties and thermal material properties is designated as $\lambda_{M,fi}$ and a value of $\lambda_{M,fi} = 1.0$ is recommended in 3-1-2/2.3. The UK NA adopts that value.

6.2 Design resistance of members at elevated temperature

6.2.1 Section classification

As for normal temperature design, all cross sections that act wholly or partly in compression are classified in order to establish the appropriate design resistance of the cross section. As the strength and the elastic modulus of steel reduce at different rates in fire conditions, the section classifications at elevated temperature may differ from those for normal temperature design. However, rather than determine classification at each elevated temperature, a single classification is made, based on normal temperature behaviour. The classification is carried out using the rules in 3-1-1/5.5 except that the value of ε for fire conditions is given by 3-1-2/4.2.2 as:

$$\varepsilon = 0.85 \sqrt{\frac{235}{f_y}}$$

where f_y is the yield strength at 20°C.

The coefficient 0.85 takes account of the variation of material properties at elevated temperatures and is an approximation for $\sqrt{k_{E,\theta}/k_{y,\theta}}$.

As for normal temperature design there are four possible classes of cross section:

- Class 1** cross sections, which can form plastic hinges.
- Class 2** cross sections, which can develop plastic moment resistance but cannot form plastic hinges.
- Class 3** cross sections, which can reach yield strength at an extreme fibre but cannot develop plastic resistance.
- Class 4** cross sections, in which local buckling will occur before yield strength is reached at an extreme fibre.

6.2.2 Resistance of tension members

For a tension member with a uniform temperature θ_a the design resistance in fire is calculated simply by applying to the normal temperature value the reduction factor $k_{y,\theta}$ for yield strength at elevated temperature, with an adjustment for the different material safety factors in normal design and fire design. In practice, since the UK NAs set both γ_{M0} and $\gamma_{M,fi}$ to unity, this adjustment is not necessary. The design resistance of tension members is given by 3-1-2/4.2.3.1 as:

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} \left(\frac{\gamma_{M0}}{\gamma_{M,fi}} \right)$$

where:

$k_{y,\theta}$ is the strength reduction factor for the yield strength of steel at temperature θ_a (see Section 5.1.2)

N_{Rd} is the design resistance of the cross section for normal temperature design.

6.2.3 Resistance of compression members

Members with uniform temperature

The design buckling resistance of compression members of Class 1, 2 or 3 with a uniform temperature θ_a at time t is determined in a similar manner as for normal temperature design but with adjustments for reduced properties at elevated temperatures. The design resistance is given by 3-1-2/4.2.3.2 as:

$$N_{b,fi,t,Rd} = \chi_{fi} A k_{y,\theta,max} \frac{f_y}{\gamma_{M,fi}}$$

The reduction factor for flexural buckling χ_{fi} is the lower of the values about the y-y and z-z axes, determined as follows:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}}$$

With

$$\varphi_{\theta} = \frac{1}{2} \left(1 + \alpha \bar{\lambda}_{\theta} + \bar{\lambda}_{\theta}^2 \right) \quad \text{and}$$

$$\alpha = 0.65 \sqrt{\frac{235}{f_y}}$$

The non-dimensional slenderness at a uniform temperature θ_a is given by:

$$\bar{\lambda}_{\theta} = \bar{\lambda} \sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}}$$

where:

$\bar{\lambda}$ is the non-dimensional slenderness at normal temperature

$k_{E,\theta}$ is the stiffness reduction factor for steel at temperature θ_a (see Section 5.1.2).

Clause 3-1-2/2.3.2(3) recommends that the non-dimensional slenderness $\bar{\lambda}$ is determined as for normal temperature design except that, for braced frames, the buckling length l_{fi} may take account of end restraint, as shown in Figure 6.1, provided that each storey of the building comprises a separate fire compartment, and that the fire resistance of the compartment boundaries is not less than that of the column. Because the continuing columns are much stiffer than the column in the fire compartment, it is assumed that they cause the end(s) of the heated column to be restrained in direction.

To achieve effective continuity of the columns, column splices may be designed in accordance with SCI P358^[26].

For members with Class 4 cross sections, see Section 6.2.7.

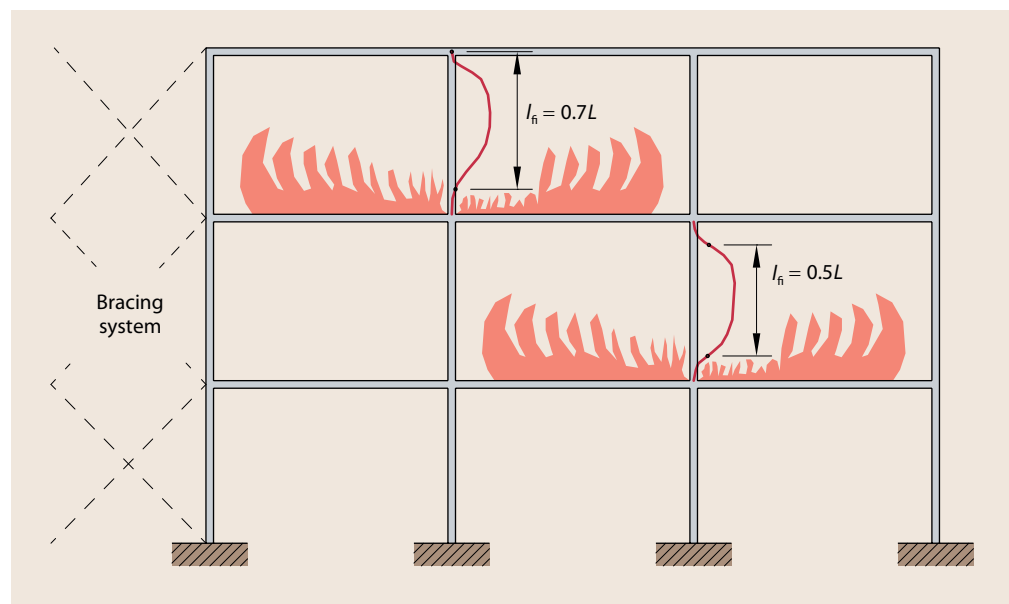


Figure 6.1
Buckling lengths of
columns in fire

Members with non-uniform temperature

For a member with a non-uniform temperature distribution, the resistance may be taken as that for a uniform temperature equal to the maximum temperature in the member at the time considered.

6.2.4 Resistance of restrained beams in bending

For restrained beams, the member resistance in bending is that of the cross section.

Beams with uniform temperature

The design moment resistance is determined in a similar manner as for normal temperature design but with adjustments for reduced properties at elevated temperatures. For Class 1 and 2 cross sections, the design resistance with a uniform temperature θ_a is given by 3-1-2/4.2.3.3 as:

$$M_{fi,\theta,Rd} = k_{y,\theta} M_{Rd} \left(\frac{\gamma_{M0}}{\gamma_{M,fi}} \right)$$

where:

M_{Rd} is the plastic moment resistance for normal temperature design, reduced if necessary to allow for the effects of shear.

As an example, the variation of the moment resistance with temperature, $M_{fi,\theta,Rd}$, for an uniformly heated $610 \times 229 \times 125$ UKB section, is shown in Figure 6.2. The section is assumed to be subject to a design moment, $M_{fi,Ed}$, of 549 kNm. The Figure shows that the resistance of the section will reduce to a value of 549 kNm when the steel temperature reaches 558°C. This value may be taken as the critical temperature, θ_{cr} (see discussion of critical temperature in Section 6.3).

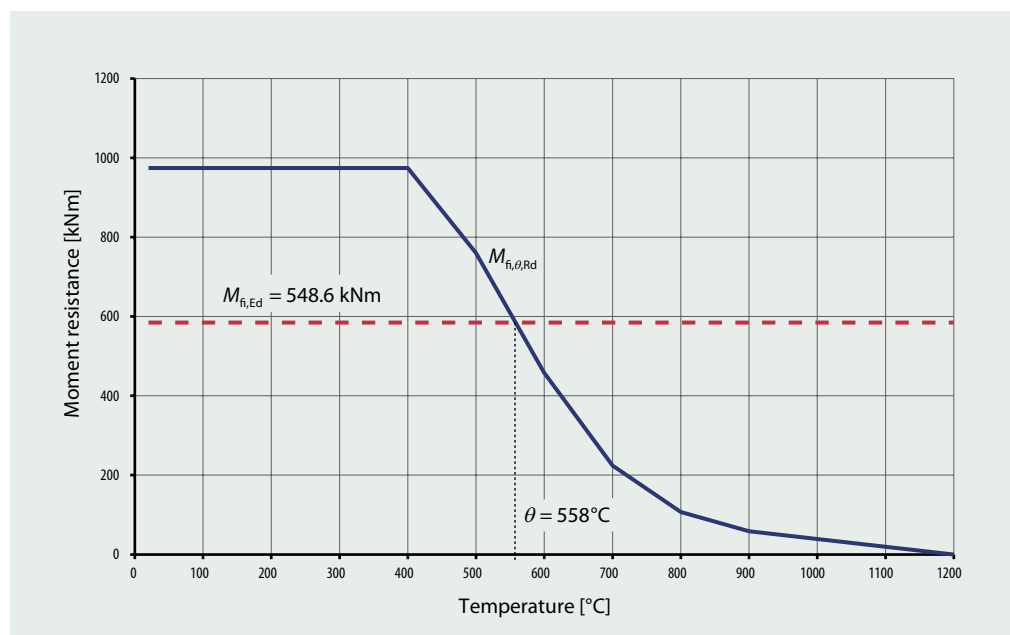


Figure 6.2
Example of the variation of beam resistance with temperature for a uniformly heated $610 \times 229 \times 125$ UKB

For a Class 3 cross section, 3-1-2/4.2.3.4 gives a similar expression:

$$M_{fi,t,Rd} = k_{y,\theta} M_{Rd} \left(\frac{\gamma_{M0}}{\gamma_{M,fi}} \right)$$

where:

M_{Rd} is the elastic moment resistance for normal temperature design, reduced if necessary to allow for the effects of shear.

Beams with non-uniform temperature

In the case of steel beams supporting concrete slabs on the top flange, the non-uniform temperature distributions may be accounted for analytically in calculating the design moment resistance (either plastic or elastic), by dividing the cross section into uniform-temperature elements, reducing the strength of each according to its temperature, and finding the resistance moment by summation across the section.

Alternatively, the resistance may be calculated conservatively by the use of two empirical adaptation factors κ_1 and κ_2 as follows:

$$M_{fi,t,Rd} = \frac{M_{fi,\theta,Rd}}{\kappa_1 \kappa_2}$$

where:

$M_{fi,\theta,Rd}$ is the design moment resistance with a uniform temperature θ_a

κ_1 is an adaptation factor for non-uniform cross-sectional temperature

κ_2 is an adaptation factor for non-uniform temperature along the beam.

The values of κ_1 and κ_2 given in 3-1-2/4.2.3.3 are given in Table 6.1.

SITUATION	FACTOR
For a beam exposed on 4 sides	$\kappa_1 = 1.0$
For an unprotected beam exposed on three sides and a composite or concrete slab on side four	$\kappa_1 = 0.70$
For an protected beam exposed on three sides and a composite or concrete slab on side four	$\kappa_1 = 0.85$

For a non-uniform temperature distribution along a beam:

At the supports of a statically indeterminate beam $\kappa_2 = 0.85$

Table 6.1
Adaptation factors

6.2.5 Buckling resistance of unrestrained beams in bending

The design buckling resistance moment is determined in a similar manner as for normal temperature design but with adjustments for reduced properties at elevated temperatures. For Class 1 or 2 cross sections, 3-1-2/4.2.3.3 gives:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{pl,y} k_{y,\theta,com} \frac{f_y}{\gamma_{M,fi}}$$

For Class 3 cross sections, 3-1-2/4.2.3.4 gives:

$$M_{b,fi,t,Rd} = \chi_{LT,fi} W_{el,y} k_{y,\theta,com} \frac{f_y}{\gamma_{M,fi}}$$

where:

$\chi_{LT,fi}$ is the lateral-torsional buckling reduction factor in a fire design situation
 $k_{y,\theta,com}$ is the reduction factor for yield strength at the maximum temperature in the compression flange $\theta_{a,com}$ at time t .

The lateral-torsional buckling reduction factor in the fire design situation is given by:

$$\chi_{LT,fi} = \frac{1}{\varphi_{LT,\theta,com} + \sqrt{[\varphi_{LT,\theta,com}]^2 - [\bar{\lambda}_{LT,\theta,com}]^2}}$$

with

$$\varphi_{LT,\theta,com} = \frac{1}{2} \left(1 + \alpha \bar{\lambda}_{LT,\theta,com} + [\bar{\lambda}_{LT,\theta,com}]^2 \right) \quad \text{and}$$

$$\alpha = 0.65 \sqrt{\frac{235}{f_y}} \quad \text{and}$$

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT} \sqrt{\frac{k_{y,\theta,com}}{k_{E,\theta,com}}}$$

in which $\bar{\lambda}_{LT}$ is the slenderness at normal temperature.

6.2.6 Members subject to combined bending and axial compression

The verification of the buckling resistance of Classes 1, 2 or 3 members subjected to combined uniaxial or biaxial bending and axial compression is very similar to that used in normal temperature design.

Two cases need to be considered:

- Members that can fail by flexural buckling, but which are restrained against lateral torsional buckling.
- Members that can fail by combined flexural buckling and lateral torsional buckling.

In 3-1-2/4.2.3.5, separate interaction criteria are given for each case, for Class 1 / Class 2 cross sections and for Class 3 cross sections, but these may be generalised to two expressions.

Members restrained against LTB

In the first case, the member should satisfy the criterion:

$$\frac{N_{fi,Ed}}{\chi_{min,fi} A k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} + \frac{k_y M_{y,fi,Ed}}{W_y k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} + \frac{k_z M_{z,fi,Ed}}{W_z k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} \leq 1.0$$

For Class 1 or 2 cross sections, the section moduli W_y and W_z are replaced by the plastic moduli $W_{pl,y}$ and $W_{pl,z}$ respectively. For Class 3 cross sections, W_y and W_z are replaced by the elastic moduli $W_{el,y}$ and $W_{el,z}$.

The minimum flexural buckling reduction factor $\chi_{min,fi}$ is the lower of the values about the y-y and z-z axes, as discussed in Section 6.2.3.

The amplification factors k_y and k_z express the increase of internal bending moments due to compression force and flexural buckling, and are calculated as for the normal temperature process except for some small changes.

Unrestrained members

In the second case, the member should satisfy the interaction criterion:

$$\frac{N_{fi,Ed}}{\chi_{z,fi} A k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} + \frac{k_{LT} M_{y,fi,Ed}}{\chi_{LT} W_y k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} + \frac{k_z M_{z,fi,Ed}}{W_z k_{y,\theta} \frac{f_y}{\gamma_{M,fi}}} \leq 1.0$$

This expression assumes that flexural buckling occurs in the same direction as the lateral-torsional buckling, so the reduction factor for buckling about the z-z axis, $\chi_{z,fi}$ is used.

The values of the coefficients k_y , k_z , and k_{LT} are given by 3-1-2/4.2.3.5. The values depend on the equivalent uniform moment factors $\beta_{M,y}$, $\beta_{M,z}$ and $\beta_{M,LT}$, which are determined from 3-1-2/Figure 4.2.

6.2.7 Class 4 members

For Class 4 members, where local buckling will occur, the members should be verified by the critical temperature method, for a critical temperature of $\theta_{crit} = 350^\circ\text{C}$.

6.2.8 Shear resistance

Shear resistance is also determined by reference to the resistance at normal temperature. The design shear resistance is given by 3-1-2/4.2.3.3(6) as:

$$V_{fi,t,Rd} = k_{y,\theta,web} V_{Rd} \left(\frac{\gamma_{M1}}{\gamma_{M,fi}} \right)$$

where:

$k_{y,\theta,web}$ is the strength reduction factor for the steel based on the average temperature in the web.

V_{Rd} is the shear resistance of the gross cross section for normal temperature design, according to BS EN 1993-1-1.

6.3 The critical temperature method

As an alternative to calculating resistances at the maximum temperature, 3-1-2/4.2.4 provides a method of determining a limiting temperature that depends on the utilization of the member in the fire situation. This is the simplest method of determining the fire resistance of an isolated loaded member in fire conditions.

According to 3-1-2/4.2.4, the method can be used only for member types for which deformation criteria or stability considerations do not have to be taken into account. This allows its use for tension members and restrained beams, but precludes its use for both columns and unrestrained beams, where stability phenomena have to be considered.

The critical temperature θ_{cr} is the temperature at which failure is expected to occur in a structural steel element with a uniform temperature distribution. Its value is determined from:

$$\theta_{cr} = 39.19 \ln \left[\frac{1}{0.9674 \mu_0^{3.833}} - 1 \right] + 482 \quad [^\circ\text{C}]$$

where:

μ_0 is the degree of utilization, given by $\mu_0 = E_{fi} / R_{fi,d,0}$ (but not less than 0.013)

E_{fi} is the design effect in the fire situation

$R_{fi,d,t}$ is the design resistance for the fire situation at time t , calculated with $\gamma_{M,fi}$.

This expression is valid for all except Class 4 sections, where, as noted in Section 6.2.7, $\theta_{cr} = 350^\circ\text{C}$ (see 3-1-2/4.2.3.6).

Values of critical temperatures, related to the degree of utilisation are shown in Figure 6.3.

An alternative simple conservative value, which presumes that the member is fully utilised under fundamental ULS load combinations is given by:

$$\mu_0 = \eta_{fi} \left(\frac{\gamma_{M,fi}}{\gamma_{M1}} \right)$$

where η_{fi} is a reduction factor defined in 3-1-2/2.4.2(3) (see Section 2.2).

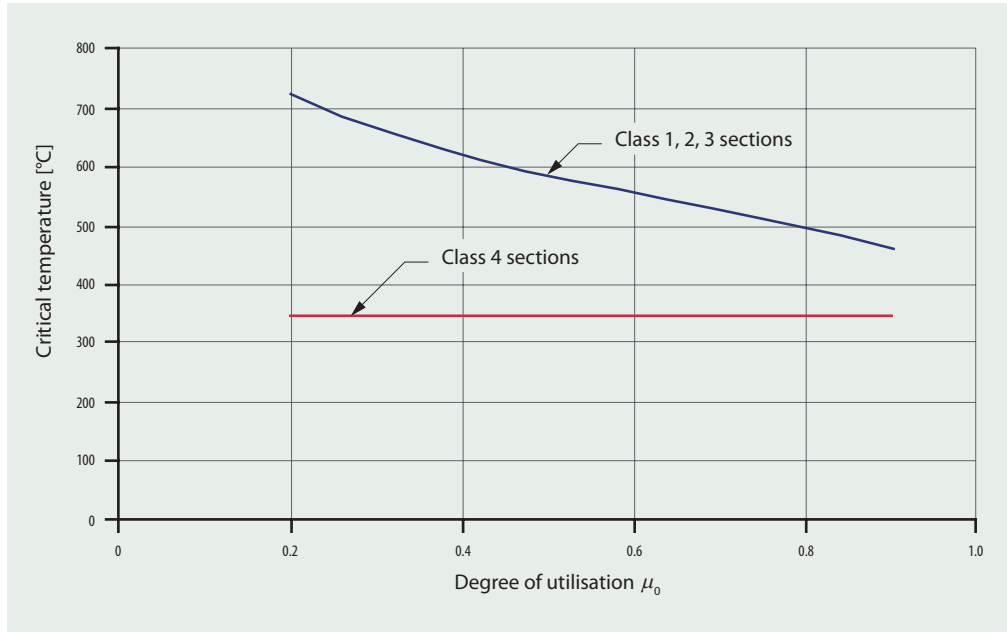


Figure 6.3
Critical temperature,
related to degree
of utilisation

6.3.1 Critical temperatures for members in compression and unrestrained beams

It is also possible to determine critical temperatures for other member types, including compression members, by utilising the expressions for resistance described in Section 6.2. This is achieved by calculating the variation of member resistance with temperature, as illustrated in Figure 6.4 for compression members.

The UK NA provides an additional table of critical temperatures, based on this method of calculation (see 3-1-2/NA.2.6). The critical temperatures for members in compression are tabulated with respect to non-dimensional slenderness $\bar{\lambda}$, calculated as for normal temperature design, but using the effective length for fire l_{fi} in place of L_{cr} .

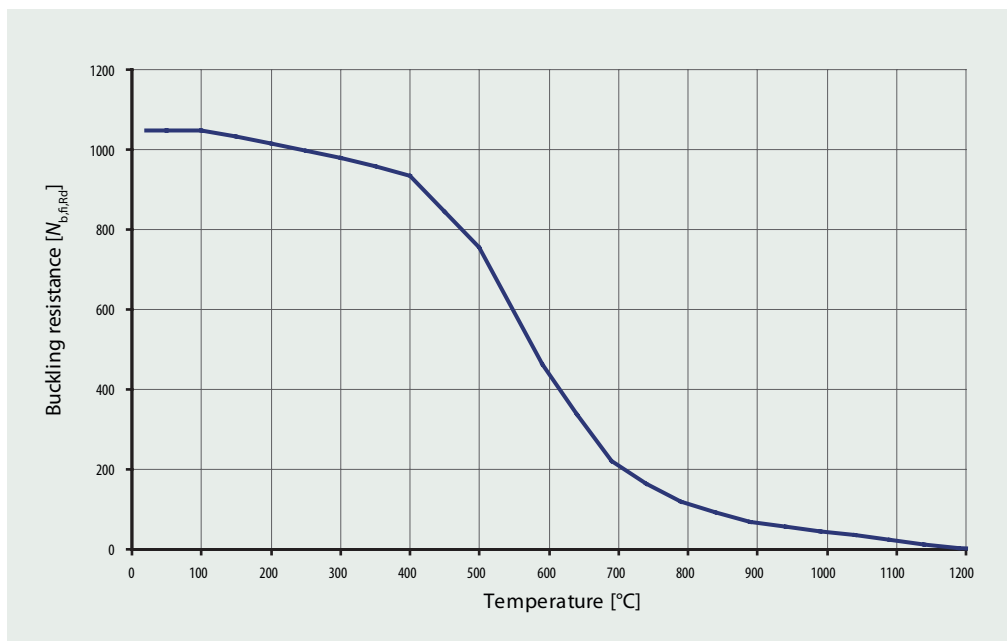


Figure 6.4
Variation of flexural
buckling resistance
with temperature

DESCRIPTION OF MEMBER	CRITICAL TEMPERATURE (°C) FOR UTILIZATION FACTOR μ_0						
	0.7	0.6	0.5	0.4	0.3	0.2	
a) Compression member: Non-dimensional slenderness $\bar{\lambda}$	0.4	485	526	562	598	646	694
	0.6	470	518	554	590	637	686
	0.8	451	510	546	583	627	678
	1.0	434	505	541	577	619	672
	1.2	422	502	538	573	614	668
	1.4	415	500	536	572	611	666
	1.6	411	500	535	571	610	665
b) Protected beam supporting a concrete slab or composite slab	558	587	619	654	690	750	
c) Unprotected beam supporting a concrete slab or composite slab	594	621	650	670	717	775	
d) Beam not supporting concrete slab; member in tension	526	558	590	629	671	725	

Table 6.2
Critical temperatures for steel compression members and unrestrained beams in bending (from 3-1-2/NA.2.6)

Notes:

1. Critical temperatures for beams supporting a concrete slab or composite slab are bottom flange temperatures.
2. For protected beams, the NA values are based on $\kappa_1 = 0.85$. For unprotected beams $\kappa_1 = 0.70$.
3. The NA values are based on $\kappa_2 = 1.0$.

6.4 Design considerations for joints

The details of joints used in steel-framed and composite construction vary widely, and depend on a variety of factors such as the basic design philosophy and assumptions about the framing system.

In the UK, it is common to use 'simple construction' for multi-storey buildings, where the joints are assumed to be nominally pinned and resistance to horizontal forces is provided by a bracing system or structural core. In this case, the principal role of beam-to-column connections is to carry the vertical end reactions of the beams.

In a frame without a bracing system, the beam-to-column connections must be designed to resist combined bending, shear and axial forces.

The fire resistance of the joints must be at least the same as for the connected members. This means that beam-to-column connections should be able to transmit the internal forces during the period when the member is required to support the design loading. When passive fire protection is used on the members, this requirement is generally considered to be fulfilled if the same thickness of fire protection is applied to the joints. In consequence, it is usually said that beam-to-column joints do not present a major problem because, due to the concentration of material, the actual temperature

of the joint tends to be lower than that of the connected members and therefore the resistance is greater than a design value based on the temperature of the beams.

6.4.1 Typical joints at normal temperature and in fire

Simple joints

Simple joints are assumed to transfer beam-end vertical shear forces into the columns. Such joints possess very low rotational stiffness (see 3-1-8/5.2 for classification of joint stiffness) and the moments induced in the columns are caused only by the eccentricity from the column centre-lines of the reaction forces. Design procedures for simple joints are given in publication P358^[26].

The flexibility of such joints is purely rotational. Horizontally, they may be required to carry tying forces in order to fulfil the robustness requirement of avoiding progressive collapse as a result of a local structural failure. Guidance on design for robustness is given in SCI P391^[27].

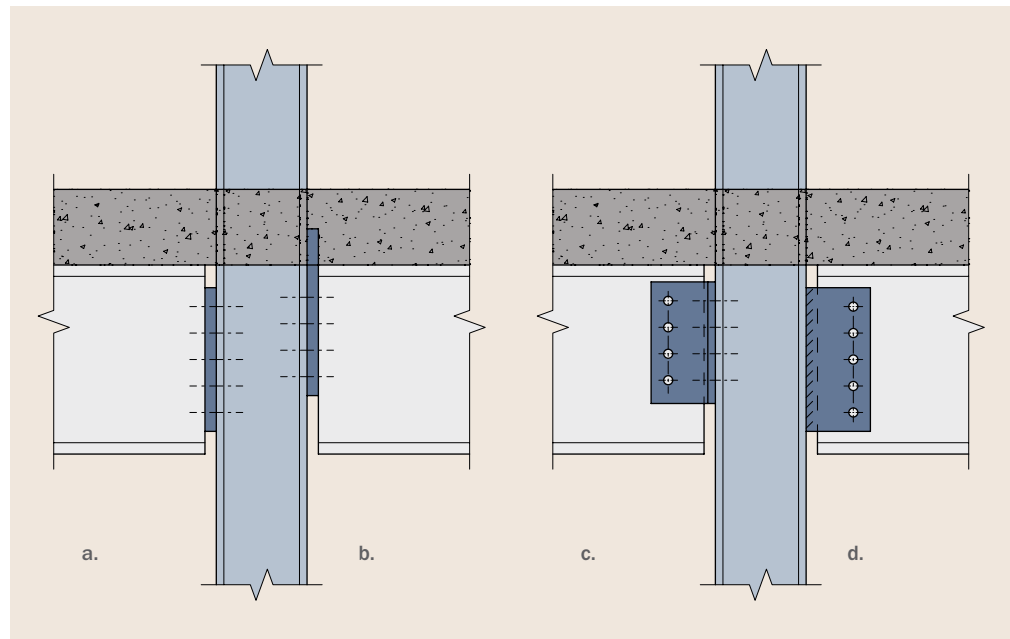


Figure 6.5
Simple joint types:
(a), (b) partial-depth
end-plates, (c) web
cleat, (d) fin-plate

6.4.2 Design recommendations for joints

BS EN 1993-1-2 has relatively little to say about joints, in contrast to the highly detailed treatment for joints at normal temperature according to BS EN 1993-1-8. There is no provision given explicitly for evaluating their moment-rotation behaviour, although the relatively cool temperatures in joint components compared with those in the members they connect make the rotational stiffness of simple joints much more significant in fire than at normal temperature.

Generally the joint will be fire protected in the same way as the members which it connects. 3-1-2/4.2.1(6) advises that the thermal resistance of the fire protection at the joint and the joint utilisation should be checked to ensure adequate performance.

The thermal resistance of the fire protection is defined in 3-1-2/4.2.1(6) as d_f/λ_f , where:

d_f is the thickness of the fire protection material ($d_f = 0$ for unprotected members.)
 λ_f is the effective thermal conductivity of the fire protection material.

For adequate performance, the thermal resistance $(d_f/\lambda_f)_c$ of the joint's fire protection should be equal or greater than the minimum value of thermal resistance $(d_f/\lambda_f)_m$ of fire protection applied to any of the jointed members.

Secondly, the utilization of the joint should be equal to or less than the maximum value of utilization of any of the connected members. As a simplification the comparison of the level of utilization within the joints and the jointed members may be performed for normal temperature. As an alternative the resistance of the joint at elevated temperature may be determined using the strength reductions for bolts and welds given in PN004^[28] at www.steel-ncci.co.uk.

Finally, the resistance of the joint at normal temperature should satisfy the recommendations given in BS EN 1993-1-8.

6.4.3 Design temperature of joints in fire

The design temperature of joints may be determined from finite element analysis. This requires some skill in the use of finite elements and some prior knowledge of the thermal performance of joints in fire. Once the temperature of the key joint components is known the resistance of the connection may be determined using the strength reduction factors given in PN004.

6.4.4 Bolted connections

Resistance of sections with holes

In terms of the member resistance at the connection, there is no requirement to calculate the net section resistance in fire, because the temperature at the joint is always lower than that of the member away from the joint. This means that the joint loses strength less quickly than the member in fire, for any of the normal loading conditions for which it is designed at normal temperature.

Bolts in shear connections

Bolts in shear may either be of the bearing type, in which the connected parts are assumed to be able to slip, one relative to the other, until the bolts are in bearing, or of the slip resistant type, which use a specified minimum preload in the bolts to generate a frictional resistance.

In the fire situation it is assumed that the heating of preloaded bolts will effectively relieve the preload, so that the bolts slip into bearing; they are then treated in the same way as bearing bolts.

The design shear resistance of a fastener in fire is given by:

$$F_{v,t,Rd} = F_{v,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

where:

$F_{v,Rd}$ is the design shear resistance of the bolt per shear plane, in accordance with BS EN 1993-1-8

$k_{b,\theta}$ is the strength reduction factor determined for the appropriate bolt temperature (see Section 5.6)

γ_{M2} is the partial safety factor for resistance of bolts at normal temperature ($\gamma_{M2} = 1.25$ according to 3-1-8/Table NA.1)

$\gamma_{M,fi}$ is the partial safety factor for fire conditions, ($\gamma_{M,fi} = 1.0$ according to 3-1-2/NA.2.2).

The design bearing resistance of bolts in fire is given by:

$$F_{b,t,Rd} = F_{b,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

in which, apart from the factors defined above,

$F_{b,Rd}$ is the design bearing resistance of the bolt in accordance with BS EN 1993-1-8.

Bolts in tension connections

The tensile strength of bolts at elevated temperatures is not usually important in calculating the reduced strength of a beam-column or beam-beam joint because bolts are rarely used in direct tension in such joints. However, when beams reach high deflections in fire they lose most of their bending stiffness and strength, and hang in catenary between the joints at their ends. At this stage the tying resistance of the joint is probably the key structural property that prevents the floor slabs from collapsing and thus allowing fire to spread vertically to higher storeys.

Non-preloaded and preloaded bolts are treated in the same way for tension in the fire situation. The design tension strength of a single bolt at elevated temperature is given by:

$$F_{ten,t,Rd} = F_{t,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

where:

$F_{t,Rd}$ is the tension resistance at normal temperature

$k_{b,\theta}$ is the strength reduction factor for bolt strength.

6.4.5 Welded connections

Butt welds

As noted in Section 5.6, the strength of a full-penetration butt weld, for temperatures up to 700°C, is taken as equal to the strength of the weaker part joined, using the appropriate strength reduction factors for structural steel. For temperatures above 700°C the reduction factors given for fillet welds can also be applied to butt welds.

Fillet welds

The design strength per unit length of a fillet weld at elevated temperature may be determined as follows using the strength reduction factors given in PN004.

$$F_{w,t,Rd} = F_{w,Rd} k_{w,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

where:

$k_{w,\theta}$ is the strength reduction factor for fillet welds

$F_{w,Rd}$ the strength per unit length at normal temperature.

COMPOSITE BEAMS AND SLABS – SIMPLE CALCULATION MODELS

BS EN 1994-1-2 permits the fire resistance of composite beams and slabs to be determined either by using a simple calculation model, by using tabulated data or by using an advanced calculation model that is based on fundamental physical behaviour. Where no data are available and no simple model is applicable, either an advanced model or a method based on test results may be used.

Detailed rules are provided in BS EN 1994-1-2 for a simple calculation model and these rules are discussed in this Section. They are applicable to members that have construction details conforming to Section 5 of BS EN 1994-1-2 and for assessment for the standard fire exposure (see Section 3.3.1 above). Only uncased sections are considered here.

Simple calculation models and tabulated data should give conservative results compared to tests or advanced calculation models.

7.1 Partial factor on material properties

The partial factor for mechanical material properties and thermal material properties in BS EN 1994-1-2 is designated as $\gamma_{M,fi}$. The values for steel, reinforcing steel, concrete and stud connectors are all recommended as 1.0 in 4-1-2/2.3. The UK NA adopts those values.

7.2 Construction details

For uncased composite beams, 4-1-2/5.1 requires that the constructional detailing shall ensure adequate shear connection between the steel and the concrete, for normal temperature and fire design. If the required shear connection cannot be maintained in the fire design situation, the beam cannot be assumed to be composite and either the steel part or the concrete part must fulfil the fire design requirements independently.

For concrete surfaces exposed to fire, the cover to reinforcement must be between 20 and 50 mm.

7.3 Composite slabs

7.3.1 Failure criteria

Composite slabs need to comply with all three of the criteria defined in 4-1-2/2.1.2(1)P:

- integrity (criterion E)
- load bearing (criterion R)
- insulation (criterion I).

The integrity of a structural element can only be demonstrated by testing but, for composite slabs designed in accordance with BS EN 1994-1-1, 4-1-2/4.3.2(6) notes that the integrity criterion is assumed to be satisfied.

For the standard fire exposure, slabs that have been designed to comply with the recommendations of BS EN 1994-1-1 (i.e. at normal temperature) are deemed to have at least 30 minutes fire resistance when assessed against load bearing criterion R.

All slabs should be checked for compliance with the insulation criteria I.

7.3.2 Unprotected slabs

A typical configuration of an unprotected composite slab is shown in Figure 7.1.

The mesh reinforcement in the top of the slab has the primary function of controlling cracking of the concrete but can also be utilised for the distribution of concentrated loads or to enhance the longitudinal shear resistance of the slab at composite beam positions. The reinforcement bar shown in the rib of the composite slab provides the main tensile reinforcement at normal temperature and in fire conditions. In the UK, the profiled sheeting often provides the main tensile reinforcement, in which cases the bar in the rib of the slab is omitted. This design approach is supported by test evidence, both at room temperature and in fire conditions.

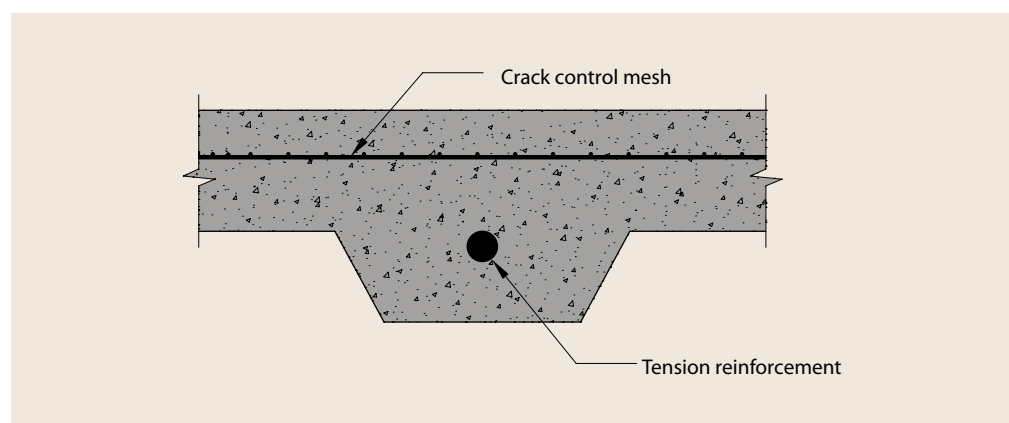


Figure 7.1
Unprotected
composite slab

Tabulated data for slabs

In the UK, most manufacturers of profiled steel sheeting for composite floors have conducted fire tests on composite floor slabs constructed using their profiles. The manufacturers are therefore able to offer specific fire design information on the fire

performance of composite floor slabs constructed using their products. Such design tables result in less conservative designs than the simple calculation model in BS EN 1994-1-2.

Sheeting manufacturers in the UK have also tested their profiles in accordance with BS EN 1363-1, so Eurocode-compliant fire test data is also available. Re testing of composite slabs to the Eurocode has resulted in an NCCI document, PN005^[21], to replace the Informative Annex D in BS EN 1994-1-2, which has been rejected in the UK National Annex.

Simple calculation model

The simple calculation model in 4-1-2/4.3.2 applies to the design of composite slabs constructed using profiled steel sheeting where the slab is either simply supported or acts as a continuous member. It applies only to sheets not protected by insulation and with no insulation between slab and screed. (Application of the rules in situations where there is insulation would give very conservative results and is not recommended.) Either normal weight or lightweight concrete may be used.

Rules for determining the temperature in slabs is given in 4-1-2/Annex D but, as noted in Section 4.3.2, that informative Annex is rejected in the UK NA (because its field of application excluded a number of UK trapezoidal profiles) and instead the UK NA refers to guidance available on www.steel-ncci.co.uk. For this purpose, the guidance in document PN005^[21] may be used.

PN005 gives rules for the verification of the load bearing criterion and insulation criterion. The rules allow the temperature profile through the thickness of the slab to be estimated, which can then be used with the strength reduction factors in Section 5.4 and Section 5.5 to calculate the strength of the components of the cross section and hence the moment resistance.

The design moment resistance of the slab is calculated using the principles of plastic resistance set out in 4-1-2/4.3.1, based on reduction factors appropriate to the calculated temperatures and material properties at normal temperature. In contrast to the guidance given in BS EN 1994-1-2, PN005 provides rules to allow the tensile resistance of the sheeting to contribute to the sagging resistance of the composite slab. No contribution from the sheeting is permitted in the calculation of the hogging resistance. PN005 also provides guidance on how the hogging resistance may be modified to allow for the rotation capacity of the slab. These recommendations follow the guidance given in BS EN 1992-1-1.

PN005 provides guidance on the minimum thickness of slab required to fulfil the insulation criterion.

Standard fire resistance tests

A more accurate assessment of the performance of a composite floor may be obtained by conducting a series of fire resistance^[29] tests in accordance with BS EN 1365-2^[29],

together with a formal assessment of the extended application of the fire resistance test results. Most UK sheeting manufacturers use this method to develop design data for composite slabs constructed using their sheeting products.

7.4 Protected composite slabs

An improvement in the fire resistance of a composite slab may be obtained by applying a fire protection system to the underside of the slab, as indicated in Figure 7.2. The performance of the protection system should be assessed in accordance with the recommendations of DD ENV 13381-5^[30].

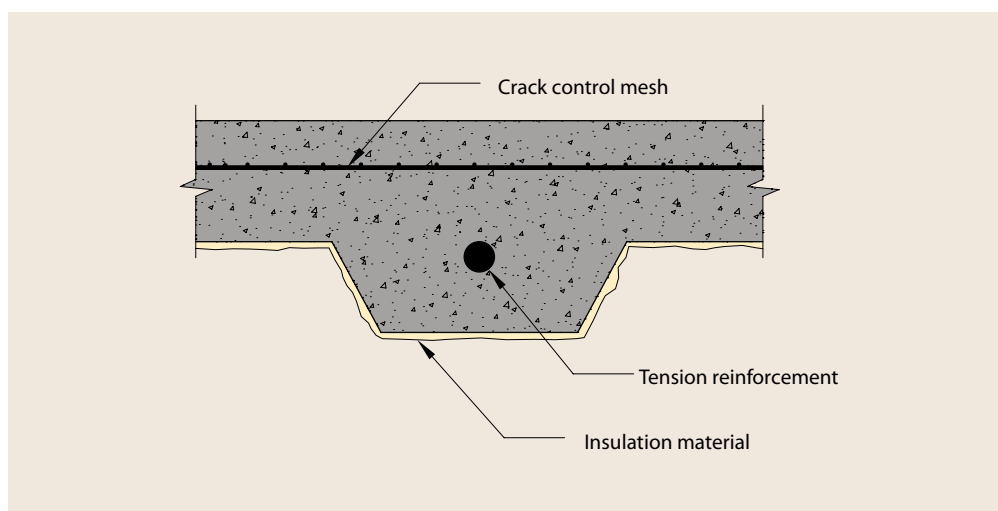


Figure 7.2
Protected
composite slab

For protected composite slabs, the thermal insulation criterion may be achieved with a lower insulation thickness, subject to the minimum thickness requirements given in BS EN 1994-1-1 for load bearing resistance.

For composite slabs designed to BS EN 1994-1-1 (at normal temperature), the load bearing criterion will be fulfilled as long as the temperature of the steel sheeting is at or below 350°C, when heated from below by the standard fire. As noted earlier, even unprotected composite slabs will achieve at least 30 minutes fire resistance when assessed for load bearing resistance. An assessment of the load bearing resistance may be made in the same way as for unprotected slabs but using design temperatures derived from a thermal analysis of the protected slab. This will require the use of thermal analysis software.

7.5 Design of composite beams without encasement

Clause 4.3.4 of BS EN 1994-1-2 contains a simple design method for down-stand beams constructed compositely, of the form shown in Figure 7.3. The method is applicable to simply supported beams and to continuous members.

4-1-2/4.3.4.1.1 requires composite beams to be checked for:

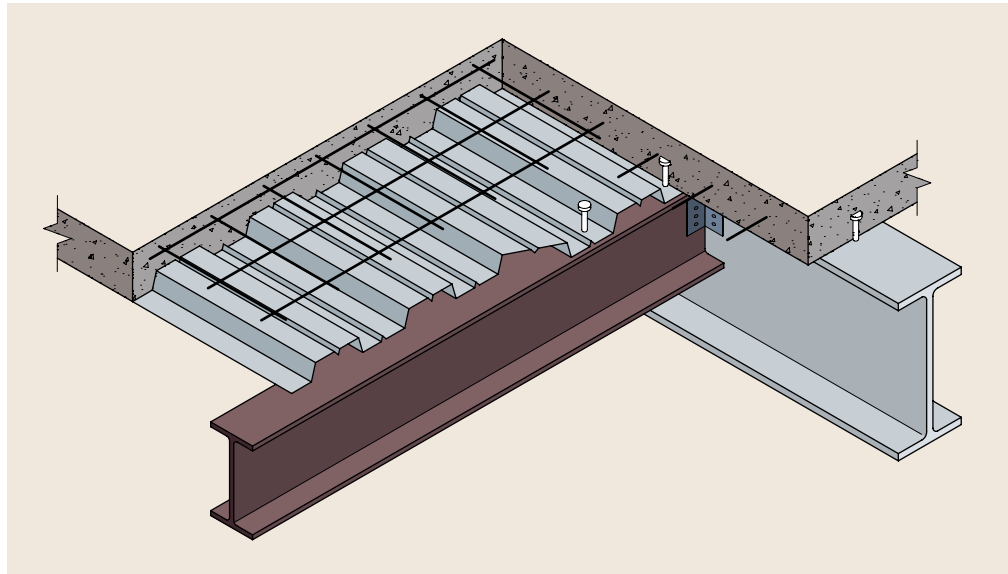


Figure 7.3
Composite beams
without encasement

- Vertical shear
- Resistance of critical cross sections to bending
- Resistance to longitudinal shear.

The temperature distribution over the cross section for the standard fire should be determined, as discussed in Section 4 for unprotected and protected steel sections.

The bending resistance may be determined either by the critical temperature model or by the bending moment resistance model.

7.5.1 Critical temperature model

When using the critical temperature model given in 4-1-2/4.3.4.2.3 to a composite beam, the temperature of the steel section is assumed to be uniform. The method is applicable to symmetric steel sections up to 500 mm deep supporting a composite slab that is not less than 120 mm deep. This method is only appropriate for simply supported members subject to sagging moments. The design method uses the concept of load level* $\eta_{fi,t}$ to define the magnitude of the load applied to the beam at the fire limit state relative to that at normal temperature. The load level is given in clause 4.3.4.2.3 by:

$$\eta_{fi,t} = \frac{E_{fi,d,t}}{R_d}$$

where:

$E_{fi,d,t}$ is the design effect in the fire situation at time t

R_d is the design resistance at normal temperature.

(But note that $E_{fi,d,t}$ is not normally time-dependent and thus neither is $\eta_{fi,t}$.)

* Note that load level $\eta_{fi,t}$ is effectively the same as the utilization factor μ_0 in 3-1-2/4.2.4 and should not be confused with the reduction factor η_{fi} ($= E_{fi,d}/E_d$) in 4-1-2/2.4.2.

The resistance of the beam is defined by the ratio between the yield strength at the critical temperature and the yield strength at normal temperature, as defined by the following relationships.

For R30
$$0.9\eta_{fi,t} = \frac{f_{ay,\theta_{cr}}}{f_{ay}}$$

For other periods of fire resistance
$$1.0\eta_{fi,t} = \frac{f_{ay,\theta_{cr}}}{f_{ay}}$$

The temperature rise in the steel section can be calculated based on the section factor of the lower flange of the steel section, using the method given in 4-1-2/4.3.4.2.2 and described in Section 3.4.3 of this guide.

For composite beams that are to be fire protected, a critical temperature is required in order to allow the required thickness of fire protection to be determined. BS EN 1994-1-2 does not provide critical temperatures for composite beams but, Table 7.1 provides an initial estimate of the critical temperature for composite beams subject to bending, based on the degree of utilization.

DEGREE OF SHEAR CONNECTION	CRITICAL TEMPERATURE (°C) FOR A UTILIZATION FACTOR μ_0					
	0.7	0.6	0.5	0.4	0.3	0.2
40%	558	588	628	666	698	763
60%	556	586	619	655	691	752
80%	545	577	608	647	684	744
100%	536	567	598	639	679	738

Table 7.1
Critical temperatures for composite down-stand beams

Note: The term utilization factor is not used in BS EN 1994-1-2 but it is taken here to mean the same as in 3-1-2/4.2.4, that is: $\mu_0 = E_{fi} / R_{fi,d,0}$.

Table 7.1 has been calculated on the basis of a uniform temperature for the steel section, which is conservative for both unprotected and protected composite beams. There is no limitation on the depth of the section that may be designed using the temperatures from the Table and the temperatures are appropriate for all values of shear connection. The slab depth should meet the requirements for minimum insulation depth given in PN005 and the minimum structural depth, which is specified in BS EN 1994-1-1 as 50 mm cover to the profiled steel sheeting.

If this Table is used for initial design, it is recommended that the resistance of the composite beam is verified by calculation of the moment resistance $M_{fi,Rd}$ using the procedure given in 4-1-2/4.3.4.2 and described below. The temperature of the steel for this calculation should be based on the value of critical temperature obtained from Table 7.1.

7.5.2 Bending resistance model

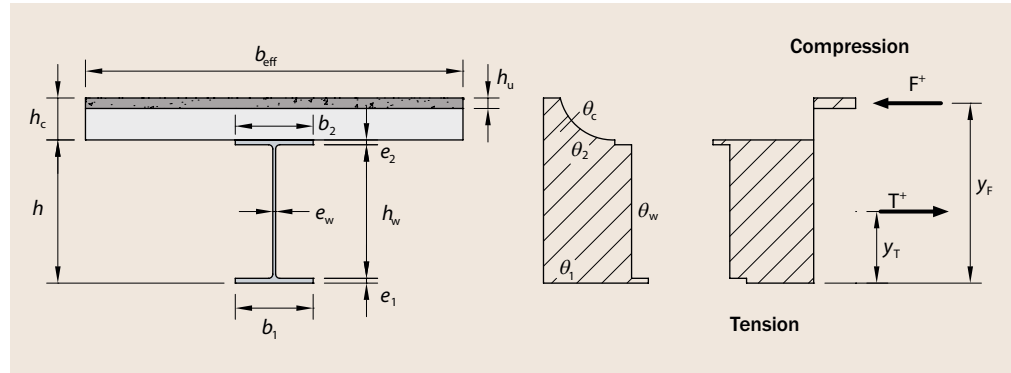
According to 4-1-2/4.3.4.1.2(1), the bending resistance of composite beams, may be determined using plastic theory for any Class of cross section except for Class 4. For simply supported beams, the steel flange in compression may be treated, independent of its class, as Class 2, provided that it is connected to the concrete slab by shear connectors (see 4-1-1/6.6.5.5).

Sagging bending resistance

For composite beams in sagging bending at elevated temperature, 4-1-2/4.3.4.2.4 requires the degree of shear connection to be taken into account and reference to 4-1-2/Annex E for a method of determining the design resistance.

To determine sagging resistance, it is necessary to calculate the overall tensile force T^+ , and its location y_T , as shown in 4-1-2/Figure E.1 and reproduced here as Figure 7.4.

Figure 7.4
Calculation of
sagging moment
resistance



The tensile force is calculated on the basis of the properties of the steel section at a given time and temperature profile through the section, as follows:

$$T^+ = b_2 e_2 k_{y,\theta_2} \frac{f_{ay}}{\gamma_{M,fi,a}} + h_w e_w k_{y,\theta_w} \frac{f_{ay}}{\gamma_{M,fi,a}} + b_1 e_1 k_{y,\theta_1} \frac{f_{ay}}{\gamma_{M,fi,a}}$$

$$y_T = \frac{b_2 e_2 k_{y,\theta_2} \frac{f_{ay}}{\gamma_{M,fi,a}} \left(h - \frac{e_2}{2} \right) + h_w e_w k_{y,\theta_w} \frac{f_{ay}}{\gamma_{M,fi,a}} \left(e_1 + \frac{h_w}{2} \right) + b_1 e_1 k_{y,\theta_1} \frac{f_{ay}}{\gamma_{M,fi,a}} \left(\frac{e_{bf}}{2} \right)}{T^+}$$

where:

$b_1, e_1, h_w, e_w, b_2, e_2$ and h are all as defined in Figure 7.4

θ_2 is the temperature of the top flange

θ_w is the temperature of the web

θ_1 is the temperature of the bottom flange

$k_{y,\theta_2}, k_{y,\theta_w}$, and k_{y,θ_1} are the strength reduction factors, dependent on the temperatures of the top flange, web and bottom flange

$\gamma_{M,fi,a}$ is the partial factor on the strength of steel in the fire situation

(= 1.0 according to the UK NA).

The tensile force in the steel beam is limited by the ability of the shear connectors between the support and the beam cross section being considered to transfer force to the concrete flange, expressed as:

$$T^+ \leq NP_{fi,Rd}$$

where:

N is the number of shear connectors between the support and the position of maximum moment

$P_{fi,Rd}$ is the design resistance of the shear connectors in fire (see Sections 4.3.3 and 7.5.3).

As a first estimate in an iterative process, the depth of the effective compressive zone h_u , may be based on the compressive strength of concrete at normal temperature and is given by:

$$h_u = T^+ / (b_{eff} \alpha_{slab} f_c / \gamma_{M,f,c})$$

where:

b_{eff} is the effective breadth of the slab

f_c is the compressive strength of the concrete at normal temperature (the characteristic value of the cylinder strength)

$$\alpha_{slab} = 0.85.$$

Note that the factor α_{slab} is included in this expression. This is consistent with 4-1-2/4.3.1. The factor is not included in Annex E expressions but the reason for its omission there is not known. It is conservative to apply it.

The effective width of slab acting compositely with the beam is calculated from 4-1-1/5.4.1.2 as:

$$b_{eff} = b_0 + \sum b_{ei}$$

with

$$b_{ei} = L_e / 8$$

where:

b_0 is the distance between centres of outstand stud connectors (= 0 for single studs)

L_e is the equivalent span (distance between points of contraflexure).

The temperature of the concrete in the compressive block may be determined using the method given in Section 4.3.2, which evaluates temperature for a series of horizontal strips, the temperature varying with depth. If the temperature over the depth h_u given by the above initial estimate does not exceed 250°C, no reduction in concrete

strength is needed. If the temperature does exceed 250°C, then the compression zone must be considered as a series of strips: the first strip is to the depth at which 250°C is attained; subsequent strips may be of uniform thickness (not greater than 10 mm), each with a reduced strength according to its temperature (see Section 5.4.2 for NC or Section 5.4.4 for LC); and a final (usually thinner) strip such that the total compression resistance equals the tension resistance.

Thus, the depth of the first slice is $(h_s - h_{cr})$

where:

h_{cr} is determined for a limiting concrete temperature of 250°C using the guidance in PN005

h_s is the depth of the composite slab.

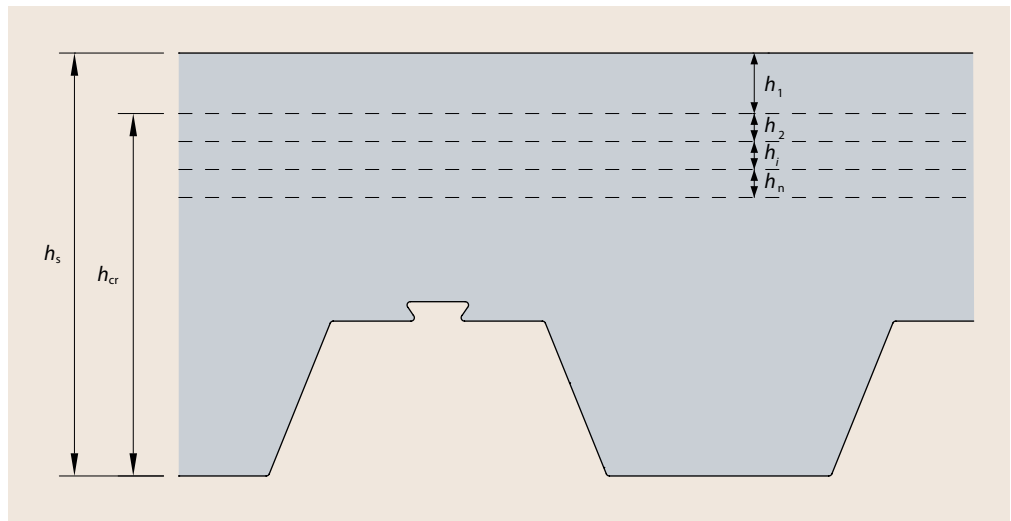


Figure 7.5
Concrete
compression zone

Equilibrium is attained when:

$$F^+ = \left[\begin{aligned} & b_{\text{eff}} (h_s - h_{cr}) \left(\frac{\alpha_{\text{slab}} f_c}{\gamma_{M,\text{fi,c}}} \right) + \sum_{i=2}^{n-1} b_{\text{eff}} h_i k_{c,\theta,i} \left(\frac{\alpha_{\text{slab}} f_c}{\gamma_{M,\text{fi,c}}} \right) \\ & + b_{\text{eff}} h_n k_{c,\theta,n} \left(\frac{\alpha_{\text{slab}} f_c}{\gamma_{M,\text{fi,c}}} \right) \end{aligned} \right] = T^+$$

Where n is the number of strips.

(Again, α_{slab} has been included, even though it is not in the Annex E expressions.)

The depth of concrete in compression is then given by:

$$h_u = (h_s - h_{cr}) + \sum_{i=2}^{n-1} h_i + h_n$$

The level of the compressive force is given by:

$$y_F \approx h + h_s - h_u / 2$$

and the sagging moment resistance is given by:

$$M_{fi,Rd^+} = T^+ (y_F - y_T)$$

The temperature of the composite slab may be determined from PN005.

7.5.3 Longitudinal shear resistance

Where beams have been designed with partial shear connection, the variation of longitudinal shear with bending resistance should be checked in the fire condition. The design longitudinal shear in the area of sagging moment is calculated from the lesser of the compression force in the slab and the tension force in the steel section, as follows:

Compression Force

$$F_C = \alpha_{slab} \sum_{j=1}^n A_j k_{c,\theta_j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right)$$

Tension Force

$$F_a = \sum_{i=1}^n A_i k_{y,\theta_i} \left(\frac{f_{y,i}}{\gamma_{M,fi,a}} \right)$$

Guidance on the provision of adequate transverse reinforcement is given in 4-1-1/6.6.6.2.

Resistance of shear connectors

The shear resistance of individual stud connectors in the fire condition is given in 4-1-2/4.3.4.2.5 as the smaller of:

$$P_{fi,Rd} = 0.8 k_{u,\theta} P_{Rd} \quad \text{and}$$

$$P_{fi,Rd} = k_{c,\theta} P_{Rd}$$

where:

$k_{u,\theta}$ is the strength reduction factor, taken as for structural steels (see Section 5.1), for the temperature of the stud θ_v (see Section 4.3.3)

$k_{c,\theta}$ is the strength reduction factor for concrete (see Section 5.4), for the temperature of the concrete (see Section 4.3.3).

This value of shear resistance is applicable to automatically welded headed studs and is based on the shear resistance of the stud at ultimate limit state in normal conditions, which should be determined from 4-1-1/6.6.3.1 except that the partial

safety factor $\gamma_{M,fi,v}$ should be used (with $\gamma_{M,fi,v} = 1.0$). Further information on the calculation of P_{Rd} at normal temperature and UK specific NCCI guidance can be obtained from P359^[31].

7.5.4 Vertical shear resistance

The resistance of the cross section is taken as the resistance of the structural steel section unless a contribution from the concrete slab has been established by testing. The design check for vertical shear resistance should be carried out in accordance with 4-1-2/E.4, which states that the rules in 4-1-1/6.2.2 should be used, except that the values of E_a , $f_{a,y}$ and γ_a are replaced by $E_{a,\theta}$, $f_{a,y,\theta}$ and $\gamma_{M,fi,a}$ respectively.

At supports of continuous beams, combined bending and shear should be checked.

REFERENCES

- [1] BRETTLER, M.E.
Steel building design: Introduction to the Eurocodes (P361).
SCI, 2009
- [2] BS EN 1991-1-2:2002,
Eurocode 1. Actions on structures. General actions - Actions on structures exposed to fire (incorporating corrigendum March 2009).
BSI, 2002
and
NA to BS EN 1991-1-2:2002,
UK National Annex to Eurocode 1: Actions on structures. General actions - Actions on structures exposed to fire.
BSI, 2007
- [3] BS EN 1993-1-2:2005,
Eurocode 3. Design of steel structures. General rules. Structural fire design (incorporating corrigenda December 2005, September 2006 and March 2009).
BSI, 2005
and
NA to BS EN 1993-1-2:2005,
UK National Annex to Eurocode 3: Design of steel structures. General rules - Structural fire design.
BSI, 2008
- [4] BS EN 1994-1-2:2005,
Eurocode 4 Design of composite steel and concrete structures. General rules - Structural fire design (incorporating corrigendum July 2008).
BSI, 2005
and
NA to BS EN 1994-1-2:2005,
UK National Annex to Eurocode 4: Design of composite steel and concrete structures. General rules - structural fire design.
BSI, 2008
- [5] PD 6688-1-2:2007,
Background paper to the UK National Annex to BS EN 1991-1.2 (incorporating corrigendum June 2007).
BSI, 2007
- [6] *The Building Regulations 2010* (Statutory Instruments 2010 No. 2214).
The Stationery Office, 2010
and
The Building (Amendment) Regulations 2011 (Statutory Instruments 2011 No. 1515).
The Stationery Office, 2011
- [7] *The Building (Scotland) Regulations 2004* (Scottish Statutory Instruments 2004 No. 406).
The Stationery Office, 2004
and Amendments up to and including: *The Building (Scotland) (Amendment) Regulations 2011* (Scottish Statutory Instruments 2011 No. 120).
The Stationery Office, 2011
- [8] *The Building Regulations (Northern Ireland) 2000* (Statutory Rules of Northern Ireland 2000 No. 389).
The Stationery Office (Belfast), 2000
and Amendments up to and including *The Building (Amendment No. 2) Regulations (Northern Ireland) 2010* (Statutory Rules of Northern Ireland 2010 No. 382).
The Stationery Office (Belfast), 2010
- [9] *The Building Regulations 2000: Approved Document B - Fire safety* (2006 Edition).
The Stationery Office, 2006
- [10] BS EN 1363-1:1999,
Fire resistance tests. Part 1 - General requirements.
BSI, 1999
- [11] Technical Handbooks 2011
Domestic Handbook, Section 2 - Fire. Non-domestic Handbook, Section 2 - Fire.
The Stationery Office, 2011
- [12] *DFP Technical Booklet E: 2005 - Fire Safety*.
Department of Finance and Personnel /
The Stationery Office, 2005
- [13] BS EN 1990:2002+A1:2005,
Eurocode. Basis of structural design (incorporating corrigenda December 2008 and April 2010).
BSI, 2010

REFERENCES

- [14] *Guide to the advanced fire safety engineering of structures.*
The Institution of Structural Engineers (IStructE), 2007
- [15] BS 5950-8:1990,
Structural use of steelwork in building. Code of practice for fire resistant design.
BSI, 1990 (Withdrawn)
- [16] DD ENV 13381-4:2002,
Test methods for determining the contribution to the fire resistance of structural members. Applied protection to steel members.
BSI, 2002
- [17] BS EN 13381-8:2010,
Test methods for determining the contribution to the fire resistance of structural members. Applied reactive protection to steel members.
BSI, 2010
- [18] KIRBY, B.R. and WAINMAN, D.E.
The behaviour of structural steelwork in natural fires.
British Steel Swinden Technology Centre, 1997
- [19] PD 7974-1:2003,
Application of fire safety engineering principles to the design of buildings. Initiation and development of fire within the enclosure of origin (Sub-system 1).
BSI, 2003
- [20] *The SFPE Handbook of Fire Protection Engineering, Fourth Edition.*
Society of Fire Protection Engineers, 2008
- [21] PN005a-GB,
NCCI: Fire resistance design of composite slabs.
Available at www.steel-ncci.co.uk
- [22] BS EN 10025,
Hot rolled products of structural steels.
(In six Parts.)
BSI, 2004
- [23] BS EN 10210,
Hot finished structural hollow sections of non-alloy and fine grain steels. (In two Parts.)
BSI, 2006
- [24] BS EN 10219,
Cold formed welded structural hollow sections of non-alloy and fine grain steels. (In two Parts.)
BSI, 2006
- [25] BS EN 10088,
Stainless steels. (In five Parts.)
BSI, 2005/2009
- [26] *Joints in steel construction, Simple joints to Eurocode 3 (P358).*
SCI, 2011
- [27] WAY, A.G.J.
Structural robustness of steel framed buildings (P391).
SCI, 2011
- [28] PN004a-GB,
NCCI: Strength of bolts and welds in fire situations.
Available at www.steel-ncci.co.uk
- [29] BS EN 1365-2:2000,
Fire resistance tests for load bearing elements. Floors and roofs.
BSI, 2000
- [30] DD ENV 13381-5:2002,
Test methods for determining the contribution to the fire resistance of structural members. Applied protection to concrete/profiled sheet composite members.
BSI, 2002
- [31] SIMMS, W.I. and HUGHES, A.F.
Composite design of steel framed buildings - In accordance with Eurocodes and the UK National Annexes (P359).
SCI, 2011

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FIRE RESISTANCE DESIGN OF STEEL FRAMED BUILDINGS

Design for fire resistance is a key requirement for most steel framed buildings. Beams and slabs are required to provide a certain level of load bearing resistance in defined fire situations; slabs are also required to provide insulation and integrity in order to limit the spread of fire. This design guide provides a general overview of the fire design of steel and composite structures in accordance with the Eurocodes, introduces the basis of design for fire situations and the criteria that need to be met. It explains how to determine the heat flux transferred to the members and how to determine their resistance at elevated temperatures. It notes that the Eurocodes cover both simple and advanced calculation models and gives guidance on the use of the simple model.

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