Structural use of steelwork in building —

Part 8: Code of practice for fire resistant design

ICS 13.220.50; 91.080.10

British Standards

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/31, Structural use of steel, upon which the following bodies were represented.

British Constructional Steelwork Association Ltd.

Cold Rolled Sections Association

Confederation of British Metalforming

Department of the Environment, Transport and the Regions

Health and Safety Executive

Institution of Civil Engineers

Institution of Structural Engineers

Office of the Deputy Prime Minister — Building Regulations Division

Office of the Deputy Prime Minister — Represented by Building Research Establishment

Steel Construction Institute

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Welding Institute

Co-opted members

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Contents

Foreword

This part of BS 5950 has been prepared by Subcommittee B/525/31, Structural use of steel. It supersedes BS 5950-8:1990, which is withdrawn.

This new edition has been prepared following the developments in fire engineering and the issue of a number of new and related standards adopting European and international standards for materials. The principal change in this edition is the introduction of a more comprehensive set of elevated temperature properties for hot finished steel, normal weight concrete, lightweight concrete and hot rolled and cold worked reinforcing bars. These properties are in agreement with those given in the Eurocodes and have been introduced to enable the fire engineering design of steel structures.

Other changes include the renaming of “Strength reduction factors” to “Strength retention factors” and the inclusion of strength retention factors for normal and lightweight concretes, bolts and welds.

The load factor for non-permanent imposed loads in offices for general use at the fire limit state has also been changed from 0.8 to 0.5. Furthermore, the load factor of 0.0 for wind loads used in the design for boundary conditions to control external fire spread has been added.

The material strength factor of concrete has been reduced from 1.30 to 1.10 and a material strength factor of 1.0 has been added for cold-worked reinforcing steel.

The section factor was defined as H_p/A in the 1990 edition of BS 5950-8. In this edition, European terminology has been used and the section factor is now defined as A_m/V. However, the numerical values are unaltered.

The subclause on castellated sections has been edited to clarify its scope of application.

BS 5950 is a document combining codes of practice to cover the design, construction and fire resistance of steel structures and specifications for materials, workmanship and erection. It comprises the following parts:

— *Part 1: Code of practice for design — Rolled and welded sections;*

— *Part 2: Specification for materials, fabrication and erection — Rolled and welded sections;*

— *Part 3: Design in composite construction: Section 3.1: Code of practice for design of simple and continuous composite beams;*

— *Part 4: Code of practice for design of composite slabs with profiled steel sheeting;*

— *Part 5: Code of practice for design of cold formed thin gauge sections;*

— *Part 6: Code of practice for design of light gauge profiled steel sheeting;*

— *Part 7: Specification for materials, fabrication and erection: cold formed sections and sheeting;*

— *Part 8: Code of practice for fire resistant design;*

— *Part 9: Code of practice for stressed skin design.*

This part of BS 5950 gives recommendations for evaluating the fire resistance of steel and steel/concrete composite structures. Methods are given for determining the thermal response of the structure and evaluating the protection required, if any, to achieve the specified performance, although it is recognized that there are situations where other proven methods may be appropriate.

It has been assumed in the drafting of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experience people; also that construction, the application of any fire protection and supervision will be carried out by capable and experienced organizations.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages ii to iv, pages 1 to 37 and a back cover.

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1 Scope

This part of BS 5950 gives recommendations for:

— steel beams, columns and tension members designed to BS 5950-1;

— steel/concrete composite beams designed to BS 5950-3;

— concrete-filled steel hollow sections designed to BS 5400-5;

— composite floors designed to BS 5950-4.

For each type of member, recommendations are given for the load carrying capacity and, where appropriate insulation performance, for a given fire exposure assuming they act in isolation and restraint to thermal expansion is ignored.

These recommendations are based on:

a) fire resistance derived from standard fire tests;

b) fire resistance derived from calculations.

NOTE These recommendations may also be applied to members for which the fire exposure has been determined from natural fires.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 476-20, *Fire tests on building materials and structures — Part 20: Method for determination of the fire resistance of elements of construction (general principles).*

BS 4482, *Specification for cold reduced steel wire for the reinforcement of concrete.*

BS 5400-5, *Steel, concrete and composite bridges — Part 5: Code of practice for design of composite bridges.*

BS 5950-1, *Structural use of steelwork in building — Part 1: Code of practice for design — Rolled and welded sections.*

BS 5950-2, *Structural use of steelwork in building — Part 2: Specification for materials, fabrication and erection — Rolled and welded sections.*

BS 5950-3.1, *Structural use of steelwork in building — Part 3: Design in composite construction: Section 3.1: Code of practice for design of simple and continuous composite beams.*

BS 5950-4, *Structural use of steelwork in building — Part 4: Code of practice for design of composite slabs with profiled steel sheeting.*

BS 7974, *Application of fire safety engineering principles to the design of buildings — Code of practice.*

BS 8110 (all parts), *Structural use of concrete.*

BS 8290-1, *Suspended ceilings — Part 1: Code of practice for design.*

BS EN 1363-1, *Fire resistance tests — Part 1: General requirements.*

BS EN 1991-1-2, *Eurocode 1: Actions on structures — Part 1-2: General actions — Actions on structures exposed to fire.*

BS EN 10025, *Hot rolled products of non-alloy structural steels — Technical delivery conditions.*

BS EN 10143, *Continuously hot-dip metal coated steel sheet and strip — Tolerances on dimensions and shape.*

BS EN 10210-1, *Hot finished structural hollow sections of non-alloy and fine grain structural steels — Part 1: Technical delivery requirements.*

DD ENV 10080, *Steel for the reinforcement of concrete — Weldable ribbed reinforcing steel B500 — Technical delivery conditions for bars, coils and welded fabric.*

3 Terms and definitions

For the purpose of this part of BS 5950, the following terms and definitions apply.

3.1
critical element
element of a section that is fundamental to the structural response

3.2
design temperature
temperature that the critical element will reach at the end of the specified period of fire resistance in a test in accordance with the standard fire test

3.3
element
one of the following:

a) the flange of a rolled or built-up I, H or channel section;

b) the web of a rolled or built-up I, H or channel section;

c) the leg of an angle and channel;

d) the flange or the stalk of an I section;

e) the side of a hollow section.

3.4
fire exposure
time–temperature response based on either the standard fire (fire resistance) or natural fires

3.5
fire protection material
material that has been shown by a standard fire test to be capable of remaining in position and providing adequate thermal insulation for the fire resistance period under consideration

3.6
fire resistance
length of time for which the member or other component is required to withstand exposure to the fire regime given in the standard fire without the load capacity falling below the fire limit state factored load or loss of integrity and/or insulation

3.7
insulation
ability of a separating component to restrict the temperature rise of its unexposed face to below specified levels

3.8
integrity
ability of a separating component to contain a fire to specified criteria for collapse, freedom from holes, cracks and fissures and sustained flaming on its unexposed face

3.9
limiting temperature
temperature of the critical element of a member at failure under fire conditions

3.10
load capacity
limit of force or moment that may be applied without causing failure due to yielding or rupture

3.11
natural fires
fire consisting of a growth and decay phase based on the type and size of the compartment, available combustible material and air supply, including parametric fires and characteristic fires as defined in BS 7974 and BS EN 1991-1-2

3.12
standard fire
fire represented by a time–temperature relationship given in BS 476-20 and BS EN 1363-1

3.13
standard fire test
test where the standard fire is used to determine the fire resistance of the member

3.14
structural member
part of a structure designed to resist force or moment, such as a steel section formed by hot rolling, cold forming or welding sections and/or plates together

3.15
thermal expansion
increase in length, cross-sectional area or volume of a material per degree increase in temperature

4 Major symbols

The major symbols used in this standard are as follows:

A_m Exposed surface area of a member per unit length;
F_f Applied axial load at the fire limit state, using the factored loads given in **7.1**;
M Applied moment at the fire limit state, using the factored loads given in **7.1**;
M_c Moment capacity at 20 °C;
M_{cf} Moment capacity at the required period of fire resistance;
V Volume of the member per unit length;
γ_f Load factor;
γ_m Material strength factor.

5 General principles

5.1 Aims of fire precautions

The aims of fire precautions are to safeguard life and to minimize fire damage to property and financial loss. These aims are principally achieved by:

a) minimizing the risk of ignition;

b) providing a safe exit for occupants;

c) restricting the spread of fire;

d) minimizing the risk of structural collapse to ensure safe exit for occupants and safety of fire-fighters.

This part of BS 5950 is principally concerned with items c) and d).

The designer responsible for the fire strategy of the building as a whole should ensure the compatibility of the structural performance requirements with the aims of fire precautions.

5.2 Structural performance requirements

The structure should be designed and constructed such that its stability will be maintained for an appropriate period by following either a prescriptive or performance based approach to fire safety.

The deformation of the structure for a given fire exposure should be considered in the design criteria of any separating elements.

The designer responsible for the overall stability of the structure during a fire should be clearly identified. This designer should ensure the compatibility of the structural design and detailing between all those structural parts and components that are required for overall stability, even if some or all the structural design and detailing of those structural parts and components is carried out by another designer.

5.3 Methods of design

The structural performance may be assessed using results from standard fire tests or calculation models that consider the fire exposure, thermal response and structural response.

5.4 Fire exposure

The fire exposure may be determined in terms of:

— the time temperature relationship defined by the standard fire (which forms the basis of fire resistance); or

— natural fire, where the geometry and linings of the compartment, the combustible material and the possible ventilation are considered. This includes parametric fires and characteristic fires as defined in BS 7974.

5.5 Thermal response

The temperature of a structural component for a given fire exposure may be determined from:

— tabulated data given in this code for structural steel elements and composite floor slabs, based on fire resistance (time–temperature response from the standard fire); or

— any calculation model for thermal response based on acknowledged principles and assumptions of the theory of heat transfer.

5.6 Structural response

The response of the structure to a given fire exposure may be determined by considering either:

— individual structural members, where the effects of restraint to thermal expansion may be ignored;

— structural analysis of sub-frames where thermal expansion and geometric non-linearity is considered;

— global structural analysis where thermal expansion and geometric non-linearity are considered.

6 Structural members in fire

6.1 Properties at elevated temperature

6.1.1 *Hot finished steel*

6.1.1.1 *General*

The following properties apply to hot finished structural steels, which are not subsequently heat treated, complying with BS EN 10025 and BS EN 10210-1 at elevated temperatures and are for use in fire calculations. Properties at ambient temperature are given in BS 5950-1.

6.1.1.2 *Thermal elongation*

The thermal elongation of steel $\Delta l/l$ may be determined from the following:

for 20 °C $\leq \theta_a <$ 750 °C:

$$\Delta l/l = 1.2 \times 10^{-5}\,\theta_a + 0.4 \times 10^{-8}\,\theta_a^{2} - 2.416 \times 10^{-4}$$

for 750 °C $\leq \theta_a \leq$ 860 °C:

$$\Delta l/l = 1.1 \times 10^{-2}$$

for 860 °C $< \theta_a \leq$ 1 200 °C:

$$\Delta l/l = 2 \times 10^{-5}\,\theta_a - 6.2 \times 10^{-3}$$

where

l is the length at 20 °C;

Δl is the temperature induced expansion;

θ_a is the steel temperature (°C).

The relation of thermal elongation with temperature is shown in Figure 1.

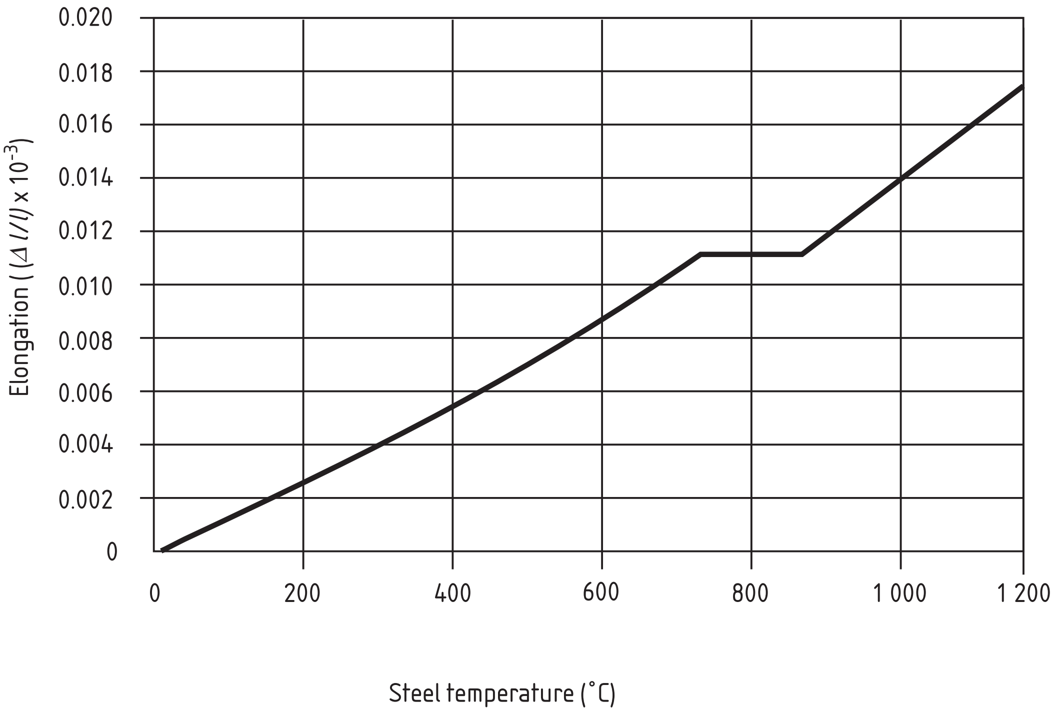


Figure 1 — Thermal elongation of steel

For simple calculation models, the thermal elongation of steel $\Delta l/l$ may be determined from:

$$\Delta l/l = 14 \times 10^{-6} (\theta_a - 20)$$

6.1.1.3 *Specific heat*

The specific heat of steel c_a may be determined from the following:

for 20 °C ≤ θ_a < 600 °C:

$$c_a = 425 + 7.73 \times 10^{-1}\theta_a - 1.69 \times 10^{-3}\theta_a^{\ 2} + 2.22 \times 10^{-6}\theta_a^{\ 3} \text{ J/kg·K}$$

for 600 °C ≤ θ_a < 735 °C:

$$c_a = 721 + \frac{5\ 371}{738 - \theta_a} \text{ J/kg·K}$$

for 735 °C ≤ θ_a < 900 °C:

$$c_a = 605 + \frac{7\ 624}{\theta_a - 731} \text{ J/kg·K}$$

for 900 °C ≤ θ_a ≤ 1 200 °C:

$$c_a = 650 \text{ J/kg·K}$$

where

θ_a is the steel temperature (°C).

The variation of specific heat with temperature is shown in Figure 2.

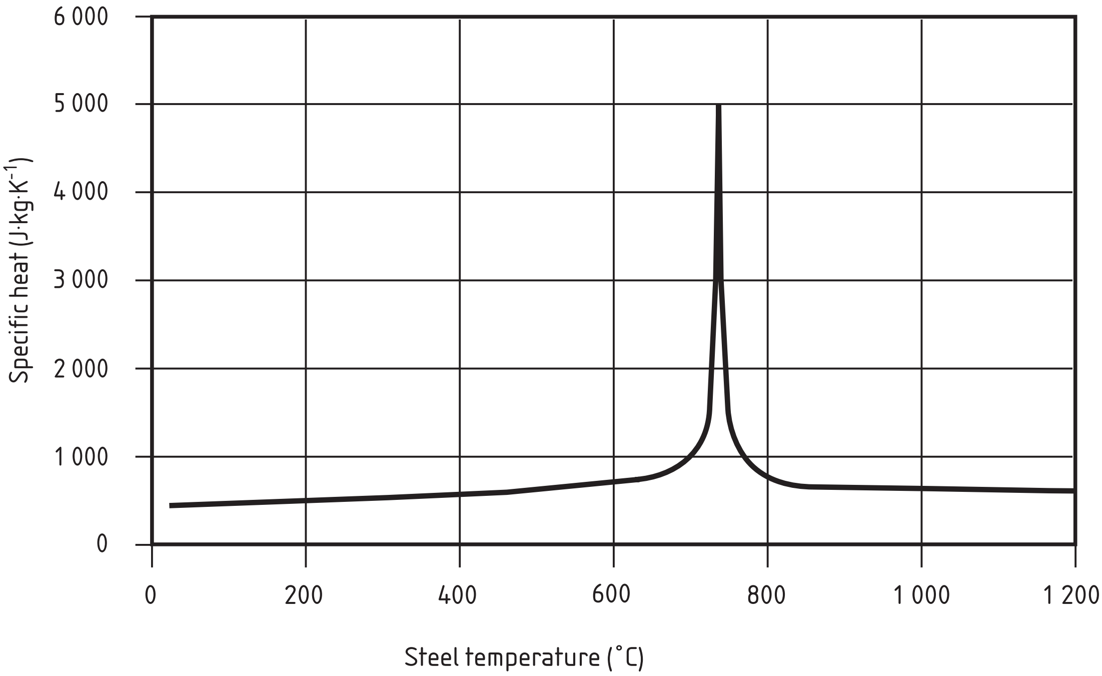


Figure 2 — Variation of specific heat with temperature for steel

6.1.1.4 *Thermal conductivity*

The thermal conductivity λ_a of steel may be determined from the following:

for 20 °C ≤ θ_a < 800 °C:

$$\lambda_a = 54 - 3.33 \times 10^{-2}\ \theta_a \text{ W/m·K}$$

for 800 °C ≤ θ_a ≤ 1 200 °C:

$$\lambda_a = 27.3 \text{ W/m·K}$$

where

θ_a is the steel temperature (°C).

The variation of thermal conductivity with temperature is shown in Figure 3.

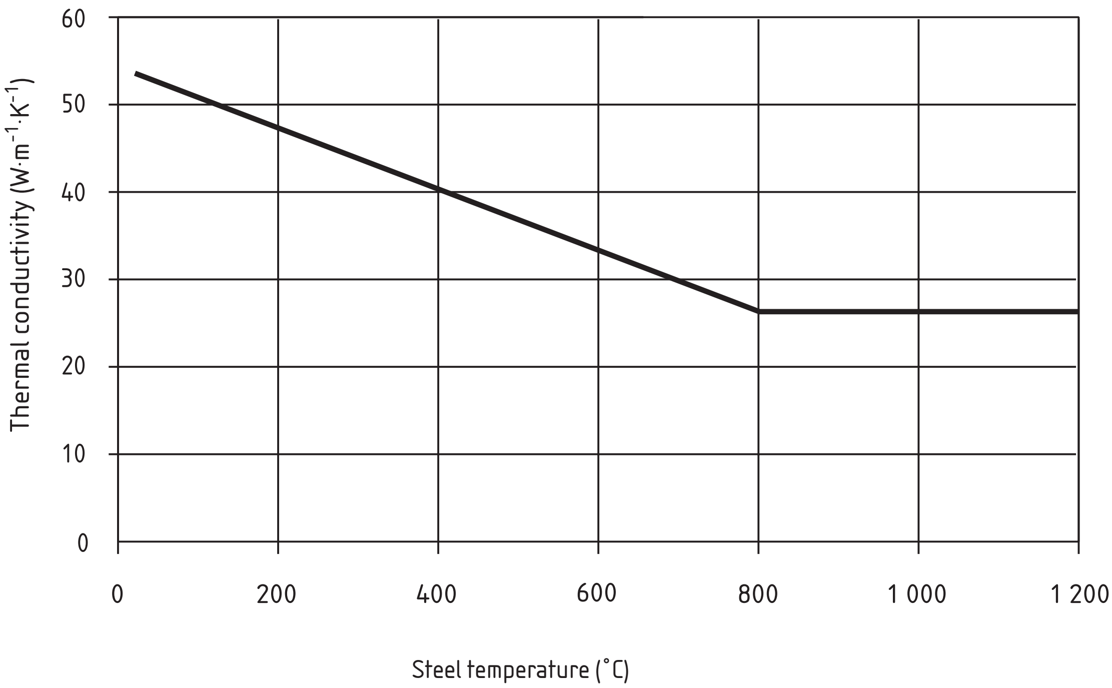


Figure 3 — Variation of thermal conductivity with temperature for steel

6.1.2 *Normal weight concrete*

6.1.2.1 *General*

The following properties can be used for normal weight concrete with a cube strength less than or equal to 60 N/mm^2, composed of either siliceous aggregate or calcareous aggregate.

6.1.2.2 *Thermal elongation*

The thermal elongation $\Delta l/l$ of normal weight concrete may be determined from the following:

for 20 °C ≤ θ_c < 700 °C:

$$\Delta l/l = -1.8 \times 10^{-4} + 9 \times 10^{-6}\theta_c + 2.3 \times 10^{-11}\theta_c^{\ 3}$$

for 700 °C ≤ θ_c ≤ 1 200 °C:

$$\Delta l/l = 14 \times 10^{-3}$$

where

l is the length of the concrete member at 20 °C;

Δl is the temperature induced expansion;

θ_c is the concrete temperature (°C).

For simple calculation models, the thermal elongation of normal weight concrete $\Delta l/l$ may be determined from:

$$\Delta l/l = 18 \times 10^{-6}(\theta_a - 20)$$

The variation of thermal elongation with temperature is shown in Figure 4.

6.1.2.3 *Specific heat*

The specific heat of normal weight concrete c_c may be determined from:

for 20 °C ≤ θ_c < 1 200 °C:

$$c_c = 900 + 80(\theta_c/120) - 4(\theta_c/120)^2 \text{ J/kg·K}$$

where

θ_c is the concrete temperature (°C).

The variation of specific heat with temperature is shown in Figure 5.

6.1.2.4 *Thermal conductivity*

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

for 20 °C ≤ θ_c < 1 200 °C:

$$\lambda_c = 2 - 0.24(\theta_c/120) + 0.012(\theta_c/120)^2 \text{ W/m·K}$$

The variation of thermal conductivity with temperature is shown in Figure 6.

6.1.2.5 *Moisture content*

In the absence of specific data, the moisture content should be assumed to be 4 % of the concrete weight.

6.1.3 ***Lightweight concrete***

6.1.3.1 *General*

The following properties can be used for lightweight concrete with a cube strength less than or equal to 60 N/mm^2.

6.1.3.2 *Thermal elongation*

The thermal elongation $\Delta l/l$ of lightweight concrete may be determined from:

$$\Delta l/l = 8 \times 10^{-6}(\theta_c - 20)$$

where

l is the length of the concrete member at 20 °C;

Δl is the temperature induced expansion;

θ_c is the concrete temperature (°C).

The variation of thermal elongation with temperature is shown in Figure 4.

6.1.3.3 *Specific heat*

The specific heat of lightweight concrete c_a may be considered to be independent of temperature and a value of 840 J/kg·K may be taken.

The variation of specific heat with temperature is shown in Figure 5.

6.1.3.4 *Thermal conductivity*

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

for 20 °C ≤ θ_c ≤ 800 °C:

$$\lambda_c = 1.0 - (\theta_c/1\ 600) \text{ W/m·K}$$

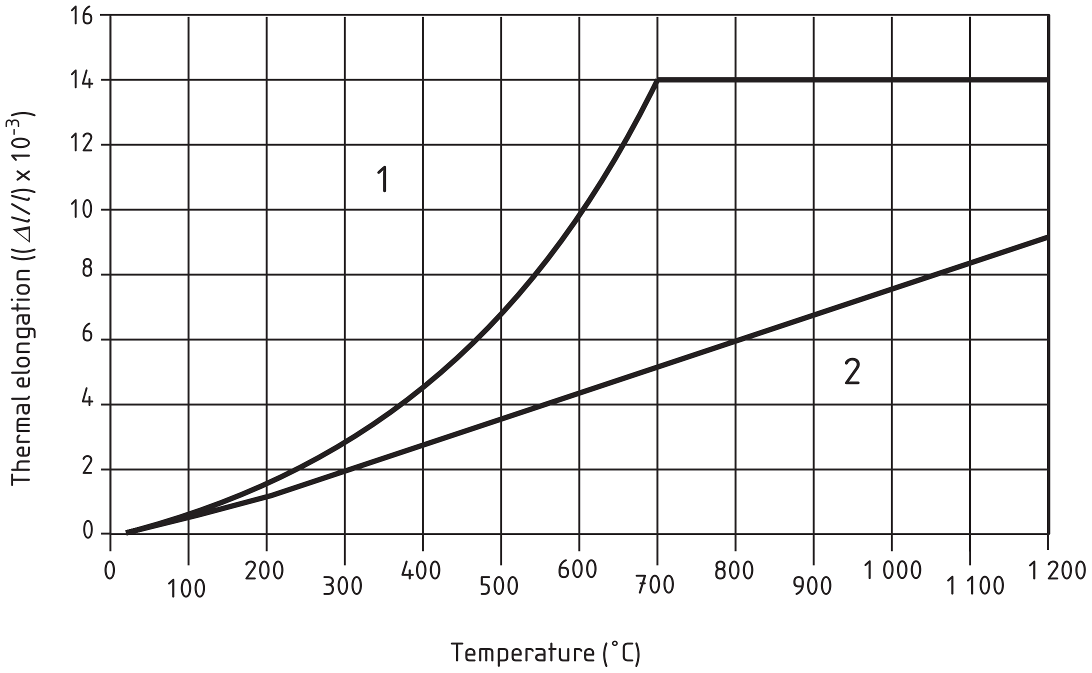
for θ_c > 800 °C:

$$\lambda_c = 0.5 \text{ W/m·K}$$

The variation of thermal conductivity with temperature is shown in Figure 6.

6.1.3.5 *Moisture content*

In the absence of specific data, the moisture content should be assumed to be 5 % of the concrete weight.

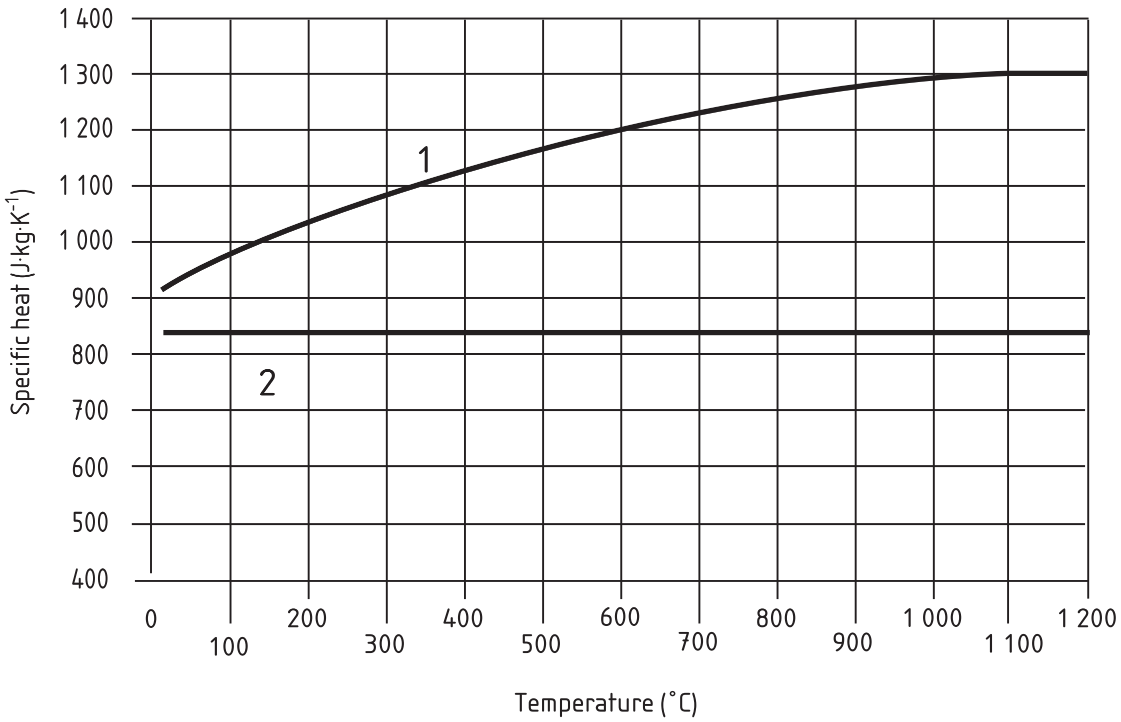


Key

1 Normal weight concrete

2 Lightweight concrete

Figure 4 — Variation of thermal elongation with temperature for normal weight and lightweight concrete

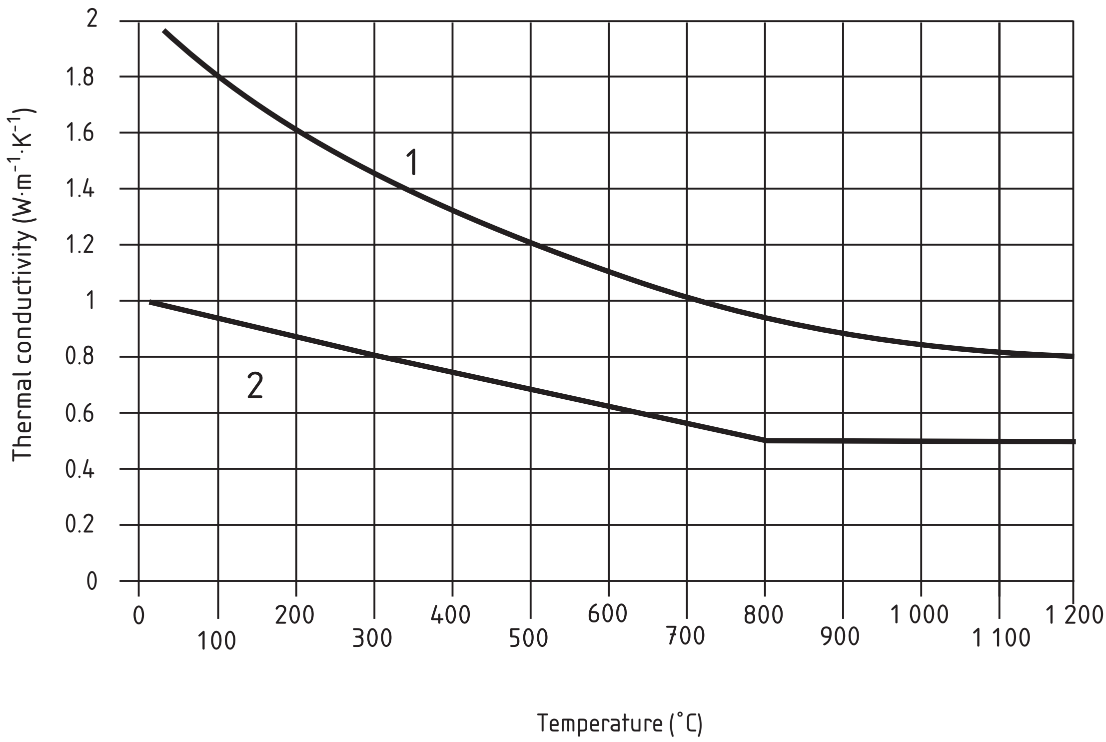


Key

1 Normal weight concrete

2 Lightweight concrete

Figure 5 — Variation of specific heat with temperature for normal weight and lightweight concrete



Key

1 Normal weight concrete

2 Lightweight concrete

Figure 6 — Variation of thermal conductivity with temperature for normal weight and lightweight concrete

6.1.4 *Hot rolled and cold worked reinforcing bars*

The properties for thermal expansion, specific heat and thermal conductivity used for hot rolled steels can be used for hot rolled and cold worked reinforcing bars.

6.2 Strength retention factors

The strength retention factors for grade S275 and S355 steels complying with BS EN 10025 and BS EN 10210-1 are given in Table 1. The appropriate value of strain should be determined from **6.3**.

The strength retention factors for normal weight and lightweight concrete are given in Table 2.

The strength retention factors for cold worked reinforcing steel are given in Table 3. The retention factors for hot rolled reinforcing steel may be taken from Table 1 corresponding to 2 % strain.

The strength retention factors for bolts and welds are given in Table 4.

The factors in Table 1, Table 2, Table 3 and Table 4 are expressed as fractions of the room temperature design strength.

Strength retention factors for cold finished steels complying with BS EN 10143 are given in Annex A.

Strength retention factors for other grades of steel should be established on the basis of elevated temperature tensile tests.

Table 1 — Strength retention factors for steel complying with grades S275 to S355 of BS EN 10025 and BS EN 10210-1

Temperature °C	Strength retention factors at a strain (in %) of:		
	0.5	1.5	2.0
100	0.97	1.000	1.000
150	0.959	1.000	1.000
200	0.946	1.000	1.000
250	0.884	1.000	1.000
300	0.854	1.000	1.000
350	0.826	0.968	1.000
400	0.798	0.956	0.971
450	0.721	0.898	0.934
500	0.622	0.756	0.776
550	0.492	0.612	0.627
600	0.378	0.460	0.474
650	0.269	0.326	0.337
700	0.186	0.223	0.232
750	0.127	0.152	0.158
800	0.071	0.108	0.115
850	0.045	0.073	0.079
900	0.030	0.059	0.062
950	0.024	0.046	0.052

NOTE 1 Intermediate values may be obtained by linear interpolation.

NOTE 2 For temperatures higher than the values given, a linear reduction in strength to zero at 1 300 °C may be assumed.

Table 2 — Strength retention factors for concrete complying with BS 8110

Temperature °C	Strength retention factors	
	Normal weight concrete	Lightweight concrete
20	1.00	1.00
100	0.95	1.00
200	0.90	1.00
300	0.85	1.00
400	0.75	0.88
500	0.60	0.76
600	0.45	0.64
700	0.30	0.52
800	0.15	0.40
900	0.08	0.28
1 000	0.04	0.16
1 100	0.01	0.04
1 200	0.00	0.00

Table 3 — Strength retention factors for cold worked reinforcing steel complying with DD ENV 10080

Temperature °C	Strength retention factor
20	1.00
100	1.00
200	1.00
300	1.00
400	0.94
500	0.67
600	0.40
700	0.12
800	0.11
900	0.08
1 000	0.05
1 100	0.03
1 200	0.00

Table 4 — Strength retention factors for grade 4.6 and 8.8 bolt assemblies and welds complying with BS 5950-2

Temperature °C	Strength retention factor for bolts (Tension and shear)	Strength retention factor for welds
20	1.00	1.00
100	0.97	1.00
150	0.95	1.00
200	0.94	1.00
300	0.90	1.00
400	0.78	0.88
500	0.55	0.63
600	0.22	0.38
700	0.10	0.13
800	0.07	0.07
900	0.03	0.02
1 000	0.00	0.00

6.3 Strain levels

When calculating the structural performance in fire, consideration should be given to both the limiting strain in the steel and the corresponding strain in any fire protection material. The following strains should not be exceeded, unless it has been demonstrated in fire resistance tests that a higher level of strain may be satisfactorily developed in the steel and that the fire protection material has the ability to remain intact:

a) composite members in bending which are unprotected or protected with fire protection materials which have demonstrated their ability to remain intact at this level of strain: 2.0 %;

b) non-composite members in bending which are unprotected or protected with fire protection materials which have demonstrated their ability to remain intact at this level of strain: 1.5 %;

c) members not covered in a) or b): 0.5 %.

7 Fire limit states

7.1 General

The structural effects of a fire in a building, or part of a building, should be considered as a fire limit state. A fire limit state should be treated as an accidental limit state.

At the fire limit state members or sub-assemblies should be assumed to be subject to the heating conditions specified in the standard fire test for the required period of fire resistance, except when analysis is based on the consideration of natural fires.

In checking the strength and stability of the structure at the fire limit state, the loads should be multiplied by the relevant load factor γ_f given in Table 5.

Wind loads should only be applied to buildings where the height to eaves is greater than eight metres and only considered when checking the design of the primary elements of the framework. The effect of wind loads can be ignored when designing for boundary conditions to control external fire spread.

Table 5 — Load factors for fire limit state

Load	γ_f
Dead load	1.00
Imposed loads:	
a) permanent:	
1) those specifically allowed for in design, e.g. plant, machinery and fixed partitions	1.00
2) in storage buildings or areas used for storage in other buildings (including libraries and designated filing areas)	1.00
b) non-permanent:	
1) in escape stairs and lobbies	1.00
2) in offices for general use	0.50
3) all other areas (imposed snow loads on roofs may be ignored)	0.80
Wind loads:	
a) design for boundary conditions to control external fire spread	0.00
b) all other cases	0.33

7.2 Material strength factors

At the fire limit state, the capacities of the members may be calculated using the following material strength factors (γ_m):

a) steel 1.00;
b) concrete 1.10;
c) cold worked reinforcing steel 1.00.

7.3 Performance criteria

In the case of a fire, the mechanical resistance of the entire structure or individual structural members should be designed and constructed in such a way that their load bearing function is maintained under the factored loads derived from **7.1** during the relevant fire exposure.

Any specified requirements for the insulation and integrity of compartment walls and floors, including any incorporated members, should also be satisfied.

7.4 Bracing members

Steel bracing members required to provide stability to the structure at the fire limit state should have adequate fire resistance, unless alternative load paths can be identified.

Where fire protection to bracing members is necessary, the protection thickness should be based on the section factor of the member or a value of 200 m^{-1}, whichever is the smaller value.

In some cases, it might not be necessary to apply fire protection to bracing members and consideration should be given to:

— shielding bracing members from fire by placing them in vertical shafts or within walls;

— the use of infill masonry walls which, although they are typically ignored in terms of overall stability at ambient temperature, can provide the sufficient shear capacity during a fire instead of relying on the steel bracing systems;

— the possibility that only bracing systems within a fire compartment might be subjected to elevated temperatures and the other unaffected bracing systems might be sufficient to provide the required stability at the fire limit state;

— the possibility that the steel beam to column connections might have sufficient stiffness to ensure stability at the fire limit state.

7.5 Re-use of steel after a fire

It might be possible to re-use steel after a fire. Guidance is given in Annex B.

8 Evaluation of fire performance

8.1 General

Fire resistance may be determined by either of the following methods:

a) standard fire tests for all types of members (see **8.3**);

b) calculation using the design methods given in this Code.

8.2 Section factor

8.2.1 *General*

The rate of temperature increase of a steel member in a fire may be assumed to be proportional to its section factor A_m/V (in m^{-1}):

where

A_m is the exposed surface area (in m) per unit length of member, as given in Table 6;

V is the volume of the member per unit length;

NOTE The section factor was defined as H_p/A in the 1990 edition. The symbols have been changed in line with European terminology. Numerical values are unaltered.

8.2.2 *Rolled, fabricated and hollow sections excluding castellated sections*

When calculating the section factor for rolled, fabricated and hollow sections, the gross cross-sectional area should be used. The effect of small holes may be ignored.

8.2.3 *Castellated sections*

For castellated sections, the section factor should be taken as that of the uncut parent section.

8.2.4 *Tapered sections*

For tapered sections, the maximum section factor should be used.

8.3 Fire performance derived from testing

8.3.1 *General*

Members designed in accordance with the appropriate part of BS 5950 may be given the required fire resistance by applying, when necessary, a fire protection material at a thickness that has been derived from the standard fire test.

Data for determining the required thickness of a given fire protection material for a member with a given section factor A_m/V for a given period of fire resistance should be derived from appraisal of a series of such tests.

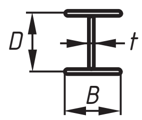
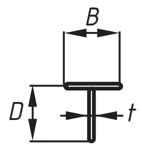
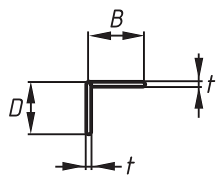
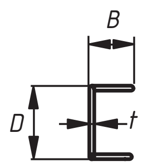
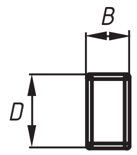
Where the factored loads for the fire limit state differ from those applied in the tests, the test results should be adjusted, either by using Table 8 or else by means of fire engineering calculation, as appropriate.

These tests should be carried out at an approved testing station and the recommendations derived from them should be prepared by a suitably qualified person.

8.3.2 *Unprotected members*

A hot finished rolled or hollow section member which has a load ratio $R \leq 0.6$ (see **8.4.2.2** and **8.4.2.3**) may be assumed to have an inherent fire resistance of 30 min without any fire protection, provided that it has a section factor A_m/V not exceeding the appropriate maximum value given in Table 7.

Table 6 — Calculation of A_m/V values

Steel section	Profile protection 4 sides	Profile protection 3 sides	Profile protection 3 sides	Profile protection 2 sides	Profile protection 1 side	Box and solid protection 4 sides	Box and solid protection 3 sides	Box and solid protection 3 sides	Box and solid protection 2 sides	Box and solid protection 1 side
Universal beams, universal columns and joists (plain and castellated)  A_m	$4B + 2D - 2t$	$3B + 2D - 2t$	$2B + 2d - t$	$3B + D - t$	B	$2B + 2D$	$B + 2D$	$B + 2d$	$B + D$	B
 A_m	$2B + 2D$	$B + 2D$	$2B + 2D - t$			$2B + 2D$	$B + 2D$	$B + 2D$		
Angles  A_m	$2B + 2D$	$B + 2D$	$2B + 2D - t$			$2B + 2D$	$B + 2D$	$B + 2D$		
Channels  A_m	$4B + 2D - 2t$	$4B + D - 2t$	$3B + 2D - 2t$			$2B + 2D$	$2B + D$	$B + 2D$		
Hollow sections, square or rectangular  A_m	$2B + 2D$	$B + 2D$				$2B + 2D$	$B + 2D$			
Hollow sections, circular  A_m	πD					πD See note 2				

NOTE 1 The general principle applied in calculating A_m/V for unprotected or profile-protected sections is to use the actual profile of the steel section; fillet radii may be taken into account and are normally included in published tables. For box protection, the smallest enclosing rectangle of the steel section is used.

NOTE 2 The air space created in boxing a section improves the insulation and a value of A_m/V, and therefore A_m higher than that for profile protection would be anomalous. Hence A_m is taken as the circumference of the tube and not 4D.

Table 7 — Maximum section factor for unprotected members

Description	A_m/V m^{-1}
Members in bending, directly supporting concrete slabs or composite slabs	90
Columns in simple construction (as described in BS 5950-1)	50
Columns comprising rolled sections filled with aerated concrete blockwork between the flanges in accordance with [3]	69

8.3.3 *Protected members*

8.3.3.1 *Required thickness*

The required thickness of fire protection materials for the required period of fire resistance should be determined from standard fire tests.

NOTE Further information on the appraisal of fire test data may be obtained from [1] and [2].

8.3.3.2 *Junctions between fire protection materials*

Full continuity of fire protection should be maintained at junctions between different methods of fire protection.

8.3.3.3 *Castellated sections*

For castellated sections complying with **4.15.5** of BS 5950-1:2000, the thickness of passive fire protection material should be 1.2 times the thickness determined from the section factor A_m/V of the original (uncastellated) section.

8.3.3.4 *Hollow sections*

The required thickness of fire protection for a hollow section should be determined using the values of the section factor A_m/V given in **8.2**.

For spray-applied lightweight cementitious based fire protection materials, the thickness required for a hollow section may be derived from the thickness t required for an I or H section with the same section factor A_m/V as follows:

for $A_m/V < 250$

thickness = $t\{1 + (A_m/V)/1\,000\}$

for $A_m/V \geq 250$

thickness = $1.25t$

In the following cases, these equations should not be used and a separate appraisal of protection thickness should be made instead:

a) where intumescent fire protection materials are used;

b) where the test data has been derived from I or H sections filled between the flanges.

8.3.3.5 *Structural connections*

Provided the load ratio of the connection defined as the load carried during the fire to the load capacity at 20 °C is lower or equal to the load ratio (see **8.4.2.2** and **8.4.2.3**) of the connecting members the thickness of protection applied to various parts of the connection should be the same as the thickness of the protection applied to the connected members.

If the load ratio of the connection is higher than the load ratio of the connecting members the protection to the connection should be increased from that applied to the connecting members to ensure that the connection does not fail first. This can be achieved by calculating the required protection thickness assuming the connecting members have a load ratio equal to the connection and applying this thickness over the connection area.

Where test evidence is available, it might be possible to reduce the protection to the bolts of the connection.

8.3.3.6 *Tension members*

Where thermal expansion can cause gaps in the fire protection materials, consideration should be given to the penetration of heat.

8.4 Fire performance derived from calculation

8.4.1 *General*

The fire behaviour of hot finished steel members may be determined using either:

a) the limiting temperature method (see **8.4.2**);

b) the moment capacity method (see **8.4.4**).

8.4.2 *Limiting temperature method*

8.4.2.1 *General*

The limiting temperature method may be used to determine the behaviour in fire of columns, tension members and beams with low shear load, designed in accordance with BS 5950-1.

Where the limiting temperature, as given in Table 8 for the applicable load ratio, is not less than the design temperature given by **8.4.3** for the required period of fire resistance, the member may be considered to have adequate fire resistance without protection.

When the limiting temperature is less than the design temperature given in **8.4.3**, the protection thickness necessary to provide adequate fire resistance may be derived either from **8.3** or else from validated calculation methods.

The limiting temperature, which should not be exceeded during the required period, depends upon the following:

a) the ratio of the load carried during the fire to the load capacity at 20 °C given in **8.4.2.2**, **8.4.2.3** or **8.4.2.4**, as applicable;

b) the temperature gradient within the member;

c) the stress profile through the cross-section;

d) the dimensions of the section.

8.4.2.2 *Load ratio for beams*

For beams designed in accordance with BS 5950-1 and having three or four sides fully exposed, the load ratio R should be taken as the greater of:

$$R = \frac{M_f}{M_c} \quad \text{or} \quad R = \frac{mM_f}{M_b}$$

where

M_f is the applied moment at the fire limit state;

M_b is the buckling resistance moment (lateral torsional);

M_c is M_{cx} or M_{cy} as appropriate to the axis of bending, where they are the moment capacity of the section about the major and minor axes in the absence of axial load;

m is the equivalent uniform moment factor.

8.4.2.3 *Load ratio for columns*

The load ratio for columns exposed on up to four sides should be determined from the following:

a) for columns in simple construction designed in accordance with the recommendations of BS 5950-1:

$$R = \frac{F_f}{A_g p_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{p_y Z_y}$$

where

A_g is the gross area;
p_c is the compressive strength;
p_y is the design strength of steel;
Z_y is the elastic modulus about the minor axis;
M_b is as defined in **8.4.2.2**;
F_f is the axial load at the fire limit state;
M_{fx} is the maximum moment about the major axis at the fire limit state;
M_{fy} is the maximum moment about the minor axis at the fire limit state.

b) for columns in continuous construction designed in accordance with BS 5950-1.

For sway or non-sway frames a load ratio of 0.67 may be used or, alternatively, the load ratio R may be taken as the greater of:

$$R = \frac{F}{A_g p_y} + \frac{M_{fx}}{M_{cx}} + \frac{M_{fy}}{M_{cy}} \quad \text{or}$$

$$R = \frac{F}{A_g p_c} + \frac{mM_{fx}}{M_b} + \frac{mM_{fy}}{p_y Z_y}$$

where

A_g, p_c, p_y, Z_y, F_f, M_{fx} and M_{fy} are as defined in **8.4.2.3**a);
M_b, M_{cx}, M_{cy} and m are as defined in **8.4.2.2**.

F_f, M_{fx}, M_{fy} should be determined taking account of any notional horizontal forces.

8.4.2.4 *Load ratio for tension members*

For tension members exposed on up to four sides, the load ratio R should be determined from:

$$R = \frac{F_f}{A_g p_y} + \frac{M_{fx}}{M_{cx}} + \frac{M_{fy}}{M_{cy}}$$

where

A_g, p_y, F_f, M_{fx}, M_{fy} are as defined in **8.4.2.3**a);
M_{cx} and M_{cy} are as defined in **8.4.2.2**.

8.4.3 *Design temperature*

8.4.3.1 *General*

The design temperature depends on the section configuration and dimensions. For unprotected rolled I or H sections, it may be determined from tests or, for common periods of fire resistance, from Table 9 for columns and tension members or Table 10 for beams.

Table 8 — Limiting temperatures for the design of protected and unprotected hot finished members

Description of member	Limiting temperature at a load ratio of:						
	0.7	0.6	0.5	0.4	0.3	0.2	0.1
	°C	°C	°C	°C	°C	°C	°C
Members in compression, for a slenderness λ[a]:							
≤ 70	510	540	580	615	655	710	800
> 70 but ≤ 180	460	510	545	590	635	635	635
Non-composite members in bending supporting concrete slabs or composite slabs:							
unprotected members, or protected members complying with item a) or b) of **6.3**	590	620	650	680	725	780	880
other protected members	540	585	625	655	700	745	800
Composite members in bending supporting concrete slabs or composite slabs:							
unprotected members, or protected members complying with item a) or b) of **6.3**							
i) 100 % degree of shear connection	550	580	610	645	685	740	840
ii) 40 % degree of shear connection	575	600	635	665	700	760	865
other protected members							
i) 100 % degree of shear connection	495	530	570	610	650	705	785
ii) 40 % degree of shear connection	530	560	595	630	675	725	795
Members in bending not supporting concrete slabs:							
unprotected members, or protected members complying with item a) or b) of **6.3**	520	555	585	620	660	715	810
other protected members	460	510	545	590	635	690	770
Members in tension: all cases	460	510	545	590	635	690	770

NOTE For beams supporting a composite slab the limiting temperatures only apply when the voids between the top of the beam and underside of the steel deck are filled with non-combustible void fillers. For guidance on limiting temperatures when void fillers are not used, see [2].

[a] λ is the slenderness, i.e. the effective length divided by the radius of gyration.

Table 9 — Design temperatures for columns and tension members. (Temperatures apply to I and H sections heated on three and four sides and angle and channel sections heated on four sides)

Flange thickness	Design temperature for fire resistance period of:			
	15 min	30 min	45 min	60 min
mm	°C	°C	°C	°C
6	672	821	895	941
7	654	811	894	940
8	635	799	893	939
9	615	786	891	938
10	596	772	889	938
11	577	760	886	937
12	558	750	883	936
13	540	743	879	935
14	523	738	874	934
15	507	735	868	933
16	491	732	861	931
17	476	728	853	930
18	462	722	844	928
19	449	716	835	926
20	436	708	825	923
22	413	692	804	917
24	391	674	784	909
26	372	657	766	899
28	355	639	752	887
30	339	622	743	874
32	325	605	737	859
34	312	589	734	844
36	300	574	729	828
38	289	559	723	812
40	279	545	714	797
45	256	512	690	764
50	237	482	663	744
55	221	456	637	735
60	208	432	612	728
65	196	410	589	714
70	185	390	567	697
75	176	373	547	679
80	168	356	527	661

NOTE 1 For fire resistance periods greater than 60 min, the steel temperature can conservatively be assumed to be the same as the furnace temperature used in the standard fire test.

NOTE 2 For square, round and rectangular steel hollow sections used in bending, compression or tension the above temperatures corresponding to a flange thickness 2.05 times the section wall thickness may be taken.

NOTE 3 The values in this table can be used to conservatively determine the web temperature of a steel section by replacing the flange thickness by the web thickness.

Table 10 — Design temperature for members in bending. (Temperatures apply to I and H sections heated on three and four sides and channel sections heated on three sides)

Flange thickness	Design temperature for fire resistance period of:			
	15 min	30 min	45 min	60 min
mm	°C	°C	°C	°C
6	672	820	895	940
7	654	810	894	940
8	634	798	887	939
9	615	785	892	938
10	596	772	891	938
11	577	760	889	937
12	560	750	887	936
13	542	743	884	935
14	526	739	879	934
15	511	736	875	933
16	496	733	869	932
17	482	730	863	930
18	468	725	856	928
19	457	720	847	927
20	445	714	840	925
22	423	699	831	920
24	401	683	812	913
26	385	668	793	905
28	370	654	778	897
30	354	638	764	886
32	342	624	752	875
34	328	609	744	862
36	318	596	738	850
38	307	583	736	837
40	300	574	730	827

NOTE 1 For fire resistance periods greater than 60 min, the steel temperature can conservatively be assumed to be the same as the furnace temperature used in the standard fire test.

NOTE 2 For hollow sections, refer to Table 9.

NOTE 3 The values in this table can be used to conservatively determine the web temperature of a steel section by replacing the flange thickness by the web thickness.

8.4.4 *Moment capacity method*

8.4.4.1 *Non-composite beams*

This method is applicable only to laterally restrained beams which have webs which satisfy the requirements for a Class 1 plastic or Class 2 compact section as defined in BS 5950-1.

A beam whose temperature profile can be defined, for a given fire exposure, may be assessed by calculating its moment capacity M_{cf} using the elevated temperature profile for the required fire exposure and the appropriate values of the strength retention factor, given in **6.2**. Provided that the applied moment M_f at the fire limit state does not exceed M_{cf}, the member may be considered to have adequate load carrying capacity for the defined fire exposure without protection.

For a cross-section that is uniformly heated to a temperature θ_D the moment capacity M_{cf} can be determined from:

$$M_{cf} = k_r M_c$$

where

k_r is the strength retention factor from Table 1 for 2 % strain relating to the steel temperature θ_D;

M_c is M_{cx} or M_{cy} as appropriate to the axis of bending, where they are the moment capacity of the section about the major and minor axes in the absence of axial load.

For a cross-section that is non-uniformly heated the moment capacity M_{cf} can be determined from:

$$M_{cf} = \sum_{i=1}^{n} A_i z_i k_r p_y$$

where

A_i is the area of an element of the cross-section with an uniform temperature θ_i;

z_i is the distance from the plastic neutral axis to the centroid of element area A_i;

k_r is the strength retention factor for a temperature θ_i from Table 1 for 2 % strain;

p_y is the design strength of steel.

When the applied moment M_f at the fire limit state exceeds M_{cf}, the protection thickness necessary to provide adequate load carrying capacity may be derived either from **8.3** or else from validated calculation methods.

A simplified calculation method for beams with shelf angles is given in Annex C.

8.4.4.2 *Composite beams*

This method is applicable only to beams which act compositely with the supporting concrete slab and have webs which satisfy the requirements for a Class 1 plastic or Class 2 compact section as defined in BS 5950-1.

The following equation can be used to determine the load-carrying capacity of the composite beam provided:

— any gaps between the underside of a steel trapezoidal or re-entrant deck of the composite floor and the top flange of the steel beam are filled with insulation;

— only the sagging moment capacity is considered;

— the depth of the steel section is less than 500 mm and the overall depth of the composite slab or concrete slab is greater than 120 mm.

The moment capacity M_{cf} for a composite beam can be determined from:

$$M_{cf} = \frac{M_c k_r}{0.7 + 0.2\left(N/N_f\right)}$$

where

M_{cf} is the moment capacity at the ultimate limit state (using partial material factors given in BS 5950-3.1 for the ultimate limit state);

k_r is the strength retention factor from Table 1 for 2 % strain based on the steel temperature of the bottom flange;

N/N_f is the degree of shear connection in the cold design.

Guidance on the design of composite beams with no insulation between the underside of the composite slab and top of the steel beam can be obtained from [2].

8.5 Portal frames

In buildings with fire resistant external walls that rely for their stability on the columns of portal frames that have rafters with no fire protection, the portal frames should be so constructed that the fire resistance of the external walls will be maintained in the event of rafter collapse in a fire.

This may be achieved by designing the bases and foundations of the portal frame columns supporting the external walls, to resist the forces and moments generated by the collapse of the portal frame rafter, taking account of the amount of roof cladding in place at the time of rafter collapse.

The columns supporting the wall should have the same fire resistance as the wall. Any fire protection to the column should extend at least to the top of the fire-resistant part of the wall, although the method of analysis used may require such protection to extend beyond that point.

NOTE For further information and design methods, see [4].

8.6 Concrete-filled hollow section columns

8.6.1 *General*

Square, rectangular and circular hollow sections, filled with unreinforced or reinforced concrete can obtain a good level of fire resistance when used as columns, without the need to apply external fire protection to the steel shell.

For greatest efficiency at the fire limit state, the section should be designed such that the load bearing capacity of the steel shell is low compared to the load carrying capacity of the concrete core.

Guidance on the design of concrete-filled hollow sections can be obtained from references [5] and [6].

8.6.2 *Externally applied fire protection to concrete-filled circular or rectangular hollow sections*

Concrete-filled structural hollow sections may be protected against fire with externally applied insulating materials. The thickness of fire protection material required for a concrete-filled structural hollow section may be determined by multiplying the thickness of the same fire protection material required for a hollow structural section of the same section factor A_m/V without concrete filling, by the modification factor c obtained from Table 11.

This method should only be used for passive insulating materials and is not applicable to active materials such as intumescent coatings.

Table 11 — Fire protection thickness modification factor

A_m/V	c
50 to 75	1.00
75	1.00
100	0.92
125	0.88
150	0.81
175	0.75
200	0.69
260 to 300	0.55

NOTE The minimum practical thickness of a protective system might be limited by the fixing system used or by the stability of the fire protection material.

8.7 Water-filled structures

Where the columns and/or beams of a structure are filled with water (or any other liquid and additive mixture suitable for use as a cooling agent), the rate at which they will heat up in a fire may be sufficiently low for any other form of fire protection to be unnecessary. If it can be shown that, in the event of a fire, any such steelwork would not be heated to a temperature that would render it unable to maintain its function, then the water-filling may be considered to give adequate fire resistance.

Methods of assessment are beyond the scope of this document and specialist literature should be consulted. The engineer should, however, be satisfied that the procedure and the assumptions made are applicable to the structure in question.

NOTE For further information, see [7].

8.8 External bare steel

Steelwork positioned outside the envelope of a building may, in certain circumstances, be adequately safe in a fire without the need for protection. If it can be shown that, in the event of a fire, any external steelwork will not be heated to such a temperature as to render it unable to maintain its function, then it may be left without any protection. Use may be made of heat shields and wired glass in adjacent windows.

In assessing the effects of fire on an external steel member, the possibility of flames being deflected by the wind and causing forced ventilation should be considered.

Methods of assessment are beyond the scope of this document and specialist literature should be consulted. The engineer should, however, be satisfied that the procedure and assumptions made are applicable to the structure in question.

NOTE For further information, see [8].

8.9 Floor and roof slabs

8.9.1 *General*

The fire resistance of a concrete floor or roof slab may be determined as follows:

a) a composite slab in accordance with BS 5950-4 should comply with **8.9.2** or **8.9.3**;

b) all other concrete slabs should comply with BS 8110-2.

8.9.2 *Unprotected composite slabs with profiled steel sheeting*

8.9.2.1 *Simplified design method*

Where a slab is designed in accordance with BS 5950-4, it may be considered to have at least a fire resistance of 30 min in its simply-supported form. Where such a slab is continuous over a number of supports, account may be taken of the enhanced fire resistance produced by such continuity.

NOTE Design data for continuous composite slabs with mesh reinforcement may be obtained from [9] and [10].

8.9.2.2 *Analytical design method*

The capacity of the composite floor for a given fire exposure can be determined subject to the following provisions.

a) The temperature within the concrete slab should be determined from Table 12, or from test results corrected for the effects of moisture.

b) The strength of any reinforcement within the slab for a given temperature should be determined using the strength retention factors given in Table 3.

c) The strength of the steel deck should be ignored at the fire limit state.

d) At the fire limit state, the plastic moment capacity of the slab based on the concrete and embedded reinforcement may be used.

e) At the fire limit state unlimited redistribution of moments may be assumed.

f) It may be assumed that, provided the slab is designed in accordance with BS 5950-4, shear failure need not be considered at the fire limit state.

NOTE For further information, see [10].

Wherever account is taken of continuity over a support, to ensure adequate ductility, the steel fabric or reinforcing bars used as support reinforcement should satisfy the minimum elongation requirement specified in BS 4482 and DD ENV 10080.

8.9.2.3 *Thermal insulation requirement*

The minimum slab depth for thermal insulation (see Figure 8 and Figure 9) in fire is met if:

a) for open trapezoidal profiles the depth of concrete above the deck is not less than that given in Table 13; or

b) for re-entrant profiles (in which the opening in the soffit does not exceed 10 % of the soffit area, and the re-entrant gap does not exceed 20 mm), the overall slab depth is not less than that given in Table 14.

NOTE BS 5950-4 recommends that the minimum depth of structural concrete over the profiled steel sheet should be 50 mm.

8.9.2.4 *Integrity*

The integrity of a composite slab with profiled steel sheeting should be maintained by forming a continuous membrane with the side seams being locked into and sealed by the concrete.

8.9.3 *Protected composite slabs with profiled steel sheeting*

The fire resistance of protected composite slabs with profiled steel sheeting may be assessed by the standard fire test.

Table 12 — Temperature distribution through a composite floor with profiled steel sheeting

Depth into slab[a]	Temperature distribution for a fire resistance period of:											
	30 min		60 min		90 min		120 min		180 min		240 min	
	NW	LW	NW	LW	NW	LW	NW	LW	NW	LW	NW	LW
mm	°C	°C	°C	°C	°C	°C	°C	°C	°C	°C	°C	°C
10	470	460	650	620	790	720	*	770	*	*	*	*
20	340	330	530	480	650	580	720	640	*	740	*	*
30	250	260	420	380	540	460	610	530	700	630	770	700
40	180	200	330	290	430	360	510	430	600	520	670	600
50	140	160	250	220	370	280	440	340	520	430	600	510
60	110	130	200	170	310	230	370	280	460	380	540	440
70	90	80	170	130	260	170	320	220	410	320	480	380
80	80	60	140	80	220	130	270	180	360	270	430	320
90	70	40	120	70	180	100	240	150	320	230	380	280
100	60	40	100	60	160	80	210	140	280	190	360	270

Key

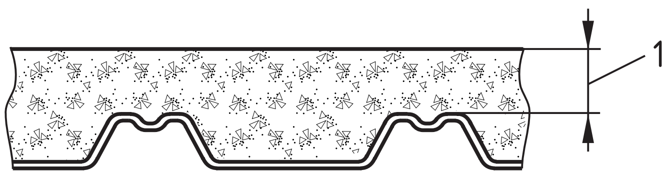
NW is ordinary dense structural concrete

LW is lightweight concrete

* indicates a temperature greater than 800 °C

[a] For any profile shape, the depth into the concrete is measured normal to the surface of the profiled steel sheet (see Figure 7).

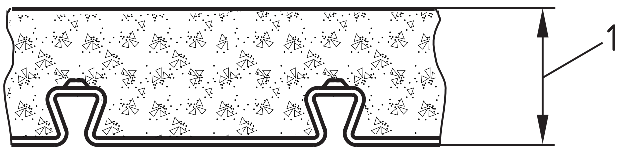
Figure 7 — Measurement of depth into concrete slab



Key

1 Insulation thickness (includes screed)

Figure 8 — Insulation thickness for trapezoidal profiled steel sheets



Key

1 Insulation thickness (includes screed)

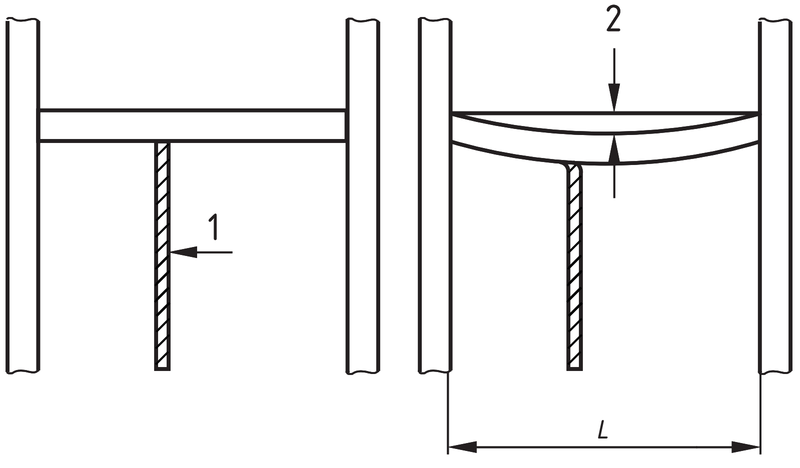
Figure 9 — Insulation thickness for re-entrant profiled steel sheets

Table 13 — Minimum thickness of concrete for trapezoidal profiled steel sheets (see Figure 8)

Concrete type	Minimum thickness of concrete for a fire resistance period of:					
	30 min	60 min	90 min	120 min	180 min	240 min
	mm	mm	mm	mm	mm	mm
Ordinary dense structural concrete	60	70	80	90	115	130
Lightweight concrete	50	60	70	80	100	115

Table 14 — Minimum thickness of concrete for re-entrant profiled steel sheets (see Figure 9)

Concrete type	Minimum thickness of concrete for a fire resistance period of:					
	30 min	60 min	90 min	120 min	180 min	240 min
	mm	mm	mm	mm	mm	mm
Ordinary dense structural concrete	90	90	110	125	150	170
Lightweight concrete	90	90	105	115	135	150



a) Before fire

b) After fire

Key

1 Internal fire wall

2 *L*/100 (at mid-span)

Figure 10 — Effect of beam deflection on a fire-resisting wall

8.10 Walls

8.10.1 *General*

The appropriate thickness of fire protection to be applied to steel members incorporated into fire-resisting walls should be determined in accordance with **8.3** or by using validated calculation methods. If the wall itself provides protection to the steel member, this may be taken into account in assessing the section factor for the member.

NOTE To comply with statutory requirements, walls very close to a site boundary may also need to be checked for resistance to an external fire.

8.10.2 *Walls connected to steel members*

Properly designed fire-resisting walls may be assumed to have sufficient inherent robustness to accommodate thermally induced differential movements between the wall and steel members incorporated into it or directly connected to it, except for walls directly under beams which support significant vertical loads, see **8.10.3**.

8.10.3 *Walls under beams*

Where a fire-resisting wall is liable to be subjected to significant additional vertical load due to the increased vertical deflection of a steel beam in a fire, see Figure 10, either:

a) provision should be made to accommodate the anticipated vertical movement of the beam; or

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

For the purpose of this clause, the anticipated vertical movement at midspan of a vertically loaded steel beam in a fire should be taken as 1/100 of its span, unless a smaller value can be justified by an analytical assessment.

8.10.4 *Independent fire-resisting walls*

Where a steel member is very close to, or touching, a fire-resisting wall which obtains its resistance to horizontal forces independently of that steel member, the effects of horizontal thermal bowing of the wall and the steel member on the stability and integrity of the fire-resisting wall should be directly assessed. Any fire protection applied to the steel member may be taken into account when determining its thermal bowing.

NOTE Further guidance is given in [11].

8.11 Roofs

Where a roof spans across a fire resisting compartment wall and it is required that strips of the roof should be fire protected on the underside either side of the compartment wall, care should be taken to fire stop any gaps between the top of the wall and the underside of the roof cladding to allow for differential thermal movement in fire.

Where practicable, combustible insulation should also be fire stopped along the line of the wall.

8.12 Ceilings

8.12.1 *General*

The contribution of the protection provided by a ceiling may be considered as supplying all or part of the fire protection required by a floor or roof, subject to the requirements of **8.12.2**.

8.12.2 *Dry suspended ceiling systems*

8.12.2.1 *General*

For structural fire protection, the complete ceiling and floor or roof construction should be considered. Ceilings should be constructed in accordance with BS 8290-1.

8.12.2.2 *Suspension systems*

The grid with its appropriate expansion cut-outs should be supported and restrained so as to ensure that the tiles or boards will remain in place and will not be dislodged in fire conditions.

8.12.2.3 *Fittings*

All fittings which penetrate the ceiling should have the same fire resistance as the ceiling, or be enclosed in a recess in the ceiling which is designed to provide the same level of fire protection as the ceiling. Ventilation ducts and similar openings should be given special consideration to ensure that the integrity of the ceiling is not broken.

8.12.2.4 *Junctions*

Junctions with other elements of the building should be checked to ensure that there will be no breakdown in the integrity of the fire resistant barrier. Care should be exercised, in particular, with the connection of internal partitions to ensure that they will not disrupt the ceiling in the event of a fire. Fire barriers in the ceiling void should be so detailed and constructed as to ensure full continuity of protection.

8.12.2.5 *Installation and maintenance*

Particular care should be taken over the installation and maintenance of suspended ceilings to ensure that long-term protection is given.

Annex A (normative)
Strength retention factors for cold formed steels complying with BS EN 10143

The strength retention factors for cold formed members made from steels complying with BS EN 10143 may be taken from Table A.1. The appropriate value of strain should be determined from **6.3**.

Table A.1 — Strength retention factors for cold formed steels complying with BS EN 10143

Strain %	Strength retention factors at a temperature (in °C) of:								
	200	250	300	350	400	450	500	550	600
0.5	0.945	0.890	0.834	0.758	0.680	0.575	0.471	0.370	0.269
1.5	1.000	0.985	0.949	0.883	0.815	0.685	0.556	0.453	0.349
2.0	1.000	1.000	1.000	0.935	0.867	0.730	0.590	0.490	0.390

NOTE Intermediate values may be obtained by linear interpolation.

Further guidance on the fire protection of cold formed steel members used as primary structural members is given in [12].

Annex B (normative)
Re-use of steel after a fire

B.1 General

Structural steel may be reused after a fire provided that its mechanical properties have not been severely impaired and that the members have not been distorted or damaged beyond acceptable engineering criteria. It is important to recognize that structural steels are produced by different manufacturing routes and these will affect the extent of distortion and deterioration in mechanical properties. Reference should be made to BS 5950-2 on the acceptability of dimensional changes that might have occurred.

In practice, for the vast majority of cases distortion of steel members due to thermal effects e.g. restriction on expansion, relaxation of locked in residual stresses generated in manufacturing cold formed shapes, will usually render a steel member unsuitable for re-use before the material properties have been unduly permanently affected. However it is important that members outside the fire affected zone should be checked for distortion or damage particularly at the main connections.

B.2 Temperature effects on strength

B.2.1 *Hot finished mild and micro-alloyed steels*

Structural steels are generally produced with mechanical properties in excess of the minimum required to satisfy the relevant standard and therefore some deterioration in properties can usually be tolerated whilst still maintaining compliance with the design strengths.

The mechanical properties of hot finished structural steels to BS EN 10025 and BS EN 10210-1 are not significantly affected until they have achieved temperatures well in excess of 600 °C. Without any confirmatory testing for mild steels (S235 and S275), it may be assumed that at least 90 % of the mechanical properties are regained irrespective of the temperatures attained (even after heating to above 1 000 °C). For micro-alloyed steels (S355), it can be assumed that at least 75 % of the strength is regained on cooling from temperatures above 600 °C.

Non-destructive test methods can be employed to ascertain whether the mechanical strength of members have been unduly affected. However, this should be carried out under the advice of a competent metallurgist with an understanding of the effect of temperature on micro-structural properties. In cases where the strength properties have deteriorated, it may be assumed for notch tough grades C and above the Charpy impact transition temperature may have also been adversely affected.

B.2.2 *"Old" hot finished structural steels to BS 15*

Hot finished structural steels produced to BS 15 are the forerunner to present day mild steels. The guidance presented for Grades S235 and S275 in **B.2.1** may be adopted.

B.2.3 ***Cold finished steel***

Cold finished steel will distort at low temperatures (less than 600 °C) due to relieving of residual stresses locked in from the manufacturing processes and will render a member unsuitable for re-use. It may be assumed that for Grades up to Z35 (BS EN 10143) 90 % of the original design strength may be retained. After a fire, the surface condition (corrosion protection system) must be checked for integrity. Most paint systems will be damaged by hot smoke and galvanized coatings will be affected by a temperature rise of only 200 °C. For light steel sections such as purlins and rails, replacement rather than reinstatement is usually the most economic proposition.

B.2.4 ***Reinforcing and prestressing steels***

The strength properties of reinforcing and prestressing steels are affected by fire to a greater extent than hot rolled structural steels although for hot and cold formed reinforcing steels in particular, the temperatures they would have to be subjected to would in any event result in a permanent serious loss in strength of the surrounding concrete.

Hot finished reinforcing steels regain all their strength after being heated up to 500 °C and at least 90 % after being heated to 600 °C.

Cold worked reinforcing steels regain all their strength after being heated to 350 °C and at least 90 % after being heated to 550 °C.

The strength properties of prestressing wires are permanently affected when they are heated to temperatures above 250 °C. In addition, pretension can be permanently affected by a rise in temperature of only 100 °C.

B.2.5 ***Cast steel***

The strength properties of cast steels may be treated in a similar manner to hot rolled structural steels of similar grade. Members may be re-used providing they remain within specified tolerances for straightness shape and area.

B.3 Connections

B.3.1 ***General***

Connections in the fire affected zone and adjoining areas should be examined to ensure that there has been no distortion or damage due to heat or thermal expansion. During heating and cooling high stress levels may be generated at the connections with the result that deformation and failure of bolts and welds may occur in either shear or tension. For bolts in particular, this might sometimes not be obvious until they are removed for inspection, in which case it is recommended new bolts are inserted.

B.3.2 ***Bolts***

Where structural members have been heated to the extent that paintwork has been burnt off (not just covered in smoke residues), it is recommended bolts be replaced at the main connections as a matter of course.

The properties of Grade 4.6 bolts behave in a similar manner to hot rolled structural mild steels and may adopt the same guidance given in **B.2.1**.

The strength properties of high strength heat treated bolts, Grades 8.8, 10.9 and above are significantly reduced after heating to temperatures in excess of 500 °C and should be replaced.

Friction grip fasteners may also loose their friction grip when heated at only moderate temperatures and should be replaced.

B.3.3 ***Welds***

Welded connections should be examined for visible cracks.

In general, the effect of fire on the mechanical properties of welds may be assumed to be the same as the parent steel member.

B.4 Fire protection materials

Many fire protection materials are rendered unsuitable for future use by fire. All fire-protection materials should be examined and replaced as necessary.

B.5 On-site checks

Inspection (and, where necessary, tests) should be carried out on site to verify the continued suitability of the structural members. Members should be assessed on the basis of their compliance with the appropriate British Standard. See [13].

Annex C (normative) Simplified method of calculation for beams with shelf angles

C.1 General

The moment capacity of a beam with shelf angles supporting in situ or precast concrete slabs may be calculated using the method described in **C.2** from the temperature profile in **C.3**, subject to the following conditions.

a) Precast concrete slabs should be made of normal weight concrete and should not have any deliberately designed voids in the end 75 mm of their length.

b) The void between the precast slab and the beam should be filled with grout.

c) Precast floor slabs should have at least 75 mm of bearing on the angles.

d) The steel angles should be of grade S355 steel, not less than 125 mm × 75 mm × 12 mm, fixed with the longer legs supporting the concrete slabs, and the vertical leg upwards, as shown in Figure C.1, Figure C.2 and Figure C.3.

e) The connections at either end of the beam should either be contained wholly within the depth of the floor slab or else fire protected to the same degree as the supporting member.

f) The moments due to the loads transmitted via the slab at the fire limit state, should not exceed the transverse moment capacity of the angles at the required period of fire resistance (M_{cf}) given by:

$$M_{cf} = 1.2\, p_y\, Z\, k_R$$

where

p_y is the design strength of steel;

Z is the elastic modulus of angle leg, equal to $t^2/6$ per unit length;

t is the thickness of angle leg;

k_R is the strength retention factor from Table 1 for 1.5 % strain, for the temperature of the angle at the fire limit state.

g) The angles may be welded or bolted to the beam. In addition to resisting the applied vertical loads at the fire limit state, the connection of the angles to the beam should be capable of transmitting the longitudinal shear force necessary to develop the required axial forces in the angles at the point of maximum moment. Any weld below the angle should be ignored. In these calculations the strengths of welds and bolts should be taken as 80 % of the relevant design strength at elevated temperature derived using the appropriate strength retention factor from Table 1 for 0.5 % strain.

C.2 Calculation method

In the calculations, a constant strain across the section as given in **6.3** should be assumed. The deflection may be ignored. The following procedure may be used.

a) Determine the temperature distribution across the section, at the fire limit state. For beams with shelf angles the temperature distribution may be assumed to be as given in **C.3**.

b) Divide the section into an appropriate number of blocks of constant width.

c) Calculate the elevated temperature load capacity of each block, assuming it to be entirely in tension or entirely in compression, as appropriate.

d) Determine the position of the horizontal plastic neutral axis of the section, which divides the total cross-section into tension and compression zones subject to equal and opposite forces.

e) Take the moment capacity of the section at the fire limit state as the algebraic sum of the positive or negative moment contribution of each block, about any convenient horizontal axis.

C.3 Temperature profile

C.3.1 *General*

The dimensions and temperature blocks shown in Figure C.1 should be used to determine the moment capacity of a beam with shelf angles.

The temperature for each block should be taken at its mid-height position.

C.3.2 *Exposed steelwork*

The temperature θ_1 of the lower flange (block 1) should be determined from **8.4.3**. The temperatures of blocks 2 and 3, and of the angle root (θ_2, θ_3 and θ_R respectively) should be determined from Table C.1, in which D_e/B_e is the aspect ratio (Figure C.1):

where

D_e is the overall exposed depth of the steel section; and

B_e is the width of the exposed bottom flange.

C.3.3 *Embedded steelwork*

The full design strength of steel is maintained at temperatures below 300 °C. Thus the temperatures of blocks 4, 5 and 6 should be calculated using:

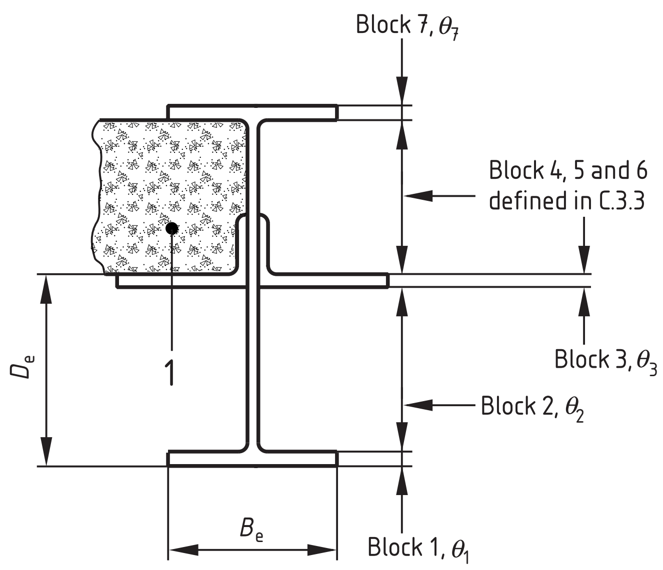
$\theta_x = \theta_R - Gx$ but $\theta_x \geq 300$ °C

where

θ_x is the temperature (in °C) at location x;

x is the distance (in mm) from angle root, measured as shown in Figure C.2;

G is the temperature gradient (in °C/mm), given in Table C.2.



Key

1 Concrete slab

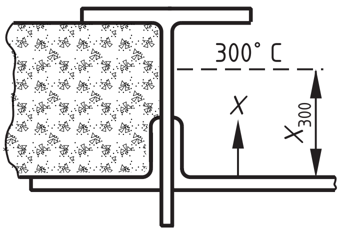
Figure C.1 — Temperature blocks for beams with shelf angles

Table C.1 — Block temperature

Aspect ratio	Block temperature for a fire resistance period of:								
	30 min			60 min			90 min		
	θ_2 °C	θ_3 °C	θ_R °C	θ_2 °C	θ_3 °C	θ_R °C	θ_2 °C	θ_3 °C	θ_R °C
$D_e/B \leq 0.6$	$\theta_1 - 140$	475	350	$\theta_1 - 90$	725	600	$\theta_1 - 60$	900	775
$0.6 < D_e/B \leq 0.8$	$\theta_1 - 90$	510	385	$\theta_1 - 60$	745	620	$\theta_1 - 30$	910	785
$0.8 < D_e/B \leq 1.1$	$\theta_1 - 45$	550	425	$\theta_1 - 30$	765	640	θ_1	925	800
$1.1 < D_e/B \leq 1.5$	$\theta_1 - 25$	550	425	θ_1	765	640	θ_1	925	800
$1.5 < D_e/B$	θ_1	550	425	θ_1	765	640	θ_1	925	800

Table C.2 — Temperature gradient

Period of fire resistance min	G °C/mm
30	2.3
60	3.8
90	4.3

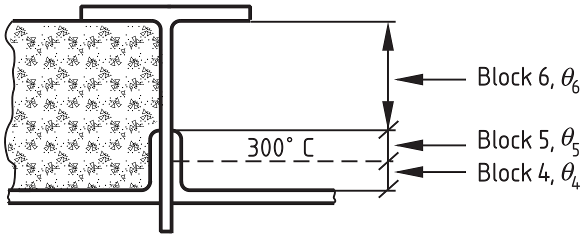


NOTE 1 The definitions of blocks 4, 5 and 6 vary depending on whether or not the 300 °C line lies within the depth of the shelf angles. See Figure C.3.

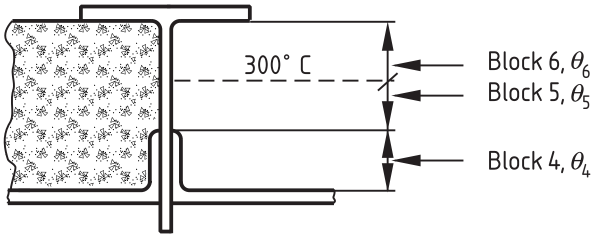
NOTE 2 The location of x_{300} is given by the following expression:

$$x_{300} = (\theta_R - 300)/G$$

Figure C.2 — Definition of dimension x



a) In angle leg



b) Above angle

Figure C.3 — Definition of blocks 4, 5 and 6

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