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British Standard

Structural use of steelwork in building

Part 8. Code of practice for fire resistant design

Aciers de construction
Partie 8. Constructions résistant au feu — Code de bonne pratique

Stahlbau
Teil 8. Bemessung in Hinblick auf Brandsicherheit

British Standards Institution

Foreword

This Part of BS 5950 has been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee. 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 in simple and continuous construction : hot rolled sections
Part 2	Specification for materials, fabrication and erection : hot rolled sections
Part 3	Design in composite construction
Section 3.1	Code of practice for design of simple and continuous composite beams
Section 3.2*	Code of practice for design of composite columns and frames
Part 4	Code of practice for design of floors with profiled steel sheeting
Part 5	Code of practice for design in cold formed sections
Part 6*	Code of practice for design in light gauge sheeting, decking and cladding
Part 7*	Specification for materials and workmanship : cold formed sections
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 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 provision will be entrusted to appropriately qualified and experienced people; also that construction, the application of any fire protection and supervision will be carried out by capable and experienced organizations.

This code of practice represents a standard of good practice and therefore takes the form of recommendations.

The full list of organizations who have taken part in the work of the Technical Committee is given on the back cover. The Chairman of the Committee is Mr P R Brett and the following people have made a particular contribution in the drafting of the code.

Mr J T Robinson Chairman of Drafting Panel
Dr G M E Cooke
Mr J I Hardwick
Dr R M Lawson
Mr G M Newman
Dr C I Smith
Mr A D Weller

NOTE. The numbers in square brackets used throughout the text of this standard relate to the bibliographic references given in appendix G.

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

*In preparation.

Contents

Section one. General

1.0 Introduction

1.0.1 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.

This Part of BS 5950 is concerned with items (c) and (d).

1.0.2 Steel in fire

Steel progressively weakens with increasing temperature and eventually failure occurs in a member as a result of its inability to sustain the applied load, e.g. buckling in the case of a column or excessive deflection in the case of flexural members. The limiting temperature at which failure occurs varies and is dependent on the loading which the member is carrying, its support conditions, the change in its properties as the temperature rises, and the temperature gradient through the cross section.

1.1 Scope

This Part of BS 5950 gives recommendations for the two following methods of achieving the specified fire resistance for steel building members and sub-assemblies (see appendix A).

(a) fire resistance derived from tests in accordance with BS 476 : Parts 20 and 21;

(b) fire resistance derived from calculations.

NOTE 1. These methods may also be applied to members for which the required fire resistance has been derived from the consideration of natural fires.

NOTE 2. The titles of the publications referred to in this standard are listed on the inside back cover.

1.2 Definitions

For the purpose of this Part of BS 5950 the following definitions apply.

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

NOTE. The web of an I, H or channel section or the stalk of a T section, is not normally critical.

1.2.2 design temperature. The temperature that the critical element will reach at the end of the specified period of fire resistance in a test in accordance with BS 476 : Parts 20 and 21.

1.2.3 element. An element may be taken as one of the following:

(a) a 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) a leg of an angle;

(d) the flange or the stalk of a T section;

(e) a side of a rectangular hollow section.

1.2.4 fire protection material. A material, which has been shown by fire resistance tests in accordance with BS 476 : Parts 20 and 21, to be capable of remaining in position and providing adequate thermal insulation for the fire resistance period under consideration.

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

1.2.6 integrity. The 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.

1.2.7 limiting temperature. The temperature of the critical element of a member at failure under fire conditions.

1.2.8 load capacity. Limit of force or moment which may be applied without causing failure due to yielding or rupture.

1.2.9 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.

1.2.10 fire resistance. The length of time for which the member or other component is required to withstand exposure to the fire regime given in BS 476 : Part 20 without the load capacity falling below the fire limit state factored load or loss of integrity and/or insulation.

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

1.3 Major symbols

A	Gross cross-sectional area of a section
F_f	Applied axial load at the fire limit state, using the factored loads given in **3.1**
H_p	Heated perimeter (see table 3)
M_f	Applied moment at the fire limit state, using the factored loads given in **3.1**
M_{cf}	Moment capacity at the required period of fire resistance
M_c	Moment capacity at 20 °C
θ_L	Limiting temperature
θ_D	Design temperature
γ_f	Load factor
γ_m	Material strength factor

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Section two. Steel in fire

2.1 Properties at elevated temperature

The following properties apply to hot finished structural steels complying with BS 4360 at elevated temperatures and are for use in fire calculations. Properties at ambient temperature are given in BS 5950 : Part 1.

(a) coefficient of linear thermal expansion = 14×10^{-6} per °C above 100 °C;

(b) specific heat = 520 J/kg·°C;

(c) thermal conductivity = 37.5 W/m·°C;

(d) Poisson's ratio = 0.3.

These values may also be assumed to apply to cold finished steels complying with BS 2989.

2.2 Strength reduction factors

The strength reduction factors for grade 43 and 50 steels complying with BS 4360 are given in table 1. The appropriate value of strain should be determined from **2.3**. The factors are expressed as fractions of the room temperature design strength and may be applied to tension, compression or shear.

Strength reduction factors for cold finished steels complying with BS 2989 are given in appendix B.

Table 1. Strength reduction factors for steel complying with grades 43 to 50 of BS 4360

Temperature	Strength reduction factors at a strain (in %) of:		
	0.5	1.5	2.0
°C			
100	0.97	1.00	1.00
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 1300 °C may be assumed.

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

2.3 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, 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) above: 0.5 %

Section three. Fire limit states

3.1 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 BS 476 : Part 20 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 2.

Wind loads should only be applied to buildings where the height to eaves is greater than 8 m and only considered when checking the design of the primary elements of the framework.

Table 2. 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) all other areas (imposed snow loads on roofs may be ignored)	0.80
Wind loads	0.33

3.2 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 1.30.

3.3 Performance criteria

Members should maintain their load capacity under the factored loads derived from **3.1** for the required period of fire resistance.

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

NOTE. The appropriate statutory requirements should be satisfied.

3.4 Bracing members

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. Whenever practicable, bracing should be built into other fire resisting components of the building, such that the bracing needs no additional protection.

3.5 Re-use of steel after a fire

It may be possible to re-use steel after a fire. Guidance is given in appendix C.

Section four. Evaluation of fire resistance

4.1 General

Fire resistance may be determined by either of the following:

(a) fire tests in accordance with BS 476 : Parts 20 and 21 for all types of members (see **4.3**);

(b) calculation in the case of hot finished steel members only (see **4.4**).

NOTE. Detailed routes through these procedures are given in appendix A.

4.2 Section factor

4.2.1 General

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

H_p is the heated perimeter (in m) as given in table 3;

A is the gross cross-sectional area of the section (in m^2).

4.2.2 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.

4.2.3 Castellated sections

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

4.2.4 Tapered sections

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

4.3 Fire resistance derived from testing

4.3.1 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 which has been derived from tests in accordance with BS 476 : Parts 20 and 21.

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

The loads applied in these tests should be equal to the member capacity (determined in accordance with the recommendations of the appropriate Part of BS 5950) divided by a factor in the range 1.4 to 1.7.

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 5 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.

4.3.2 Unprotected members

A hot finished rolled or hollow section member which has a load ratio $R \leqslant 0.6$ (see **4.4.2.2** and **4.4.2.3**) may be assumed to have an inherent fire resistance of 30 minutes without any fire protection, provided that it has a section factor H_p/A not exceeding the appropriate maximum value given in table 4.

4.3.3 Protected members

4.3.3.1 *Required thickness.* The required thickness of fire protection materials for the required period of fire resistance should be determined from fire tests in accordance with BS 476 : Parts 20 and 21.

NOTE. Further information on the appraisal of fire test data may be obtained from [2] and [3].

4.3.3.2 *Junctions between fire protection materials.* Full continuity of fire protection should be maintained at junctions between different methods of fire protection.

4.3.3.3 *Castellated sections.* For castellated sections the thickness of the fire protection material should be 1.2 times the thickness determined from the section factor H_p/A of the original (uncastellated) section.

4.3.3.4 *Hollow sections.* The required thickness of fire protection for a hollow section should be determined using the values of the section factor H_p/A given **in 4.2**.

For passive spray-applied 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 H_p/A as follows:

for $H_p/A < 250$

thickness = $t[1 + (H_p/A)/1000]$

for $H_p/A \geqslant 250$

thickness = $1.25t$

In the following cases a separate appraisal of protection thickness should be made:

(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.

4.3.3.5 *Structural connections.* When fire protection materials are applied to a structure, the thickness of protection applied to a bolted or welded connection should be based on the thickness required for whichever of the members jointed by the connection has the highest section factor H_p/A.

4.3.3.6 *Tension members.* Where thermal expansion may cause gaps in the fire protection materials, special consideration should be given to the penetration of heat.

4.4 Fire resistance derived from calculation

4.4.1 General

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

(a) the limiting temperature method (see **4.4.2**);

(b) the moment capacity method (see **4.4.4**).

4.4.2 Limiting temperature method

4.4.2.1 *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 : Part 1.

Where the limiting temperature, as given in table 5 for the applicable load ratio, is not less than the design temperature given by **4.4.3** 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 **4.4.3** the protection thickness necessary to provide adequate fire resistance may be derived either from **4.3** or else from the calculation given in appendix D.

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 **4.4.2.2**, **4.4.2.3** or **4.4.2.4**, as applicable;

(b) the temperature gradient within the member;

(c) the stress profile through the cross section;

(d) the dimensions of the section.

4.4.2.2 *Load ratio for beams.* For beams designed in accordance with BS 5950 : Part 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 section about the major and minor axes in the absence of axial load;

m is the equivalent uniform moment factor.

4.4.2.3 *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 : Part 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 **4.4.2.2**;

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 : Part 1.

Table 3. Calculation of H_p/A 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) D, t, B, H_p	$4B+2D-2t$	$3B+2D-2t$	$2B+2d-t$	$2B+D-t$	B	$2B+2D$	$B+2D$	$B+2d$	$B+D$	B
B, D, t, H_p	$2B+2D$	$B+2D$	$2B+2D-t$			$2B+2D$	$B+2D$	$B+2D$		
Angles B, D, t, H_p	$2B+2D$	$B+2D$	$2B+2D-t$			$2B+2D$	$B+2D$	$B+2D$		
Channels B, D, t, H_p	$4B+2D-2t$	$4B+D-2t$	$3B+2D-2t$			$2B+2D$	$2B+D$	$B+2D$		
Hollow sections, square or rectangular B, D, H_p	$2B+2D$	$B+2D$				$2B+2D$	$B+2D$			
Hollow sections, circular D, H_p	πD					πD	(see note 2)			

NOTE 1. The general principle applied in calculating H_p/A 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 H_p/A, and therefore H_p, higher than that for profile protection would be anomalous. Hence H_p is taken as the circumference of the tube and not $4D$.

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:

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

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

where

A_g, p_c, p_y, Z_y F_f M_x and M_y are as defined in **4.4.2.3** (a);

M_b, M_{cx}, M_{cy} and m are as defined in **4.4.2.2**.

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

4.4.2.4 *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_x, M_y are as defined in **4.4.2.3** (a);

M_{cx} and M_{cy} are as defined in **4.4.2.2**.

4.4.3 Design temperature

4.4.3.1 *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 6 for columns and tension members or table 7 for beams.

Table 4. Maximum section factor for unprotected members

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

Table 5. Limiting temperatures for 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
	°C	°C	°C	°C	°C	°C
Members in compression, for a slenderness λ (see note)						
⩽ 70	510	540	580	615	655	710
> 70 but ⩽ 180	460	510	545	590	635	635
Members in bending supporting concrete slabs or composite slabs:						
unprotected members, or protected members complying with item (a) or (b) of **2.3**	590	620	650	680	725	780
other protected members	540	585	625	655	700	745
Members in bending not supporting concrete slabs:						
unprotected members, or protected members complying with item (a) or (b) of **2.3**	520	555	585	620	660	715
other protected members	460	510	545	590	635	690
Members in tension:						
all cases	460	510	545	590	635	690

NOTE. λ is the slenderness, i.e. the effective length divided by the radius of gyration.

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Table 6. Design temperature for columns and tension members

Flange thickness	Design temperature for fire resistance period of:			
	30 min	60 min	90 min	120 min
mm	°C	°C	°C	°C
⩽6.8	841	945	1006	1049
9.4	801	911	950	1020
11.0	771	900	950	1020
12.5	747	891	950	1020
14.2	724	882	950	1020
15.4	709	877	950	1020
17.3	689	869	950	1020
18.7	676	864	950	1020
20.5	661	858	950	1020
21.7	652	854	950	1020
23.8	637	848	950	1020
25.0	630	844	950	1020
27.0	618	839	950	1020
30.2	601	832	950	1020
31.4	595	829	950	1020
36.5	574	820	950	1020
37.7	569	818	950	1020
42.9	552	810	950	1020
44.1	548	808	950	1020
49.2	533	801	950	1020
58.0	512	791	950	1020
67.5	494	782	950	1020
77.0	479	774	950	1020

NOTE. The values given in table 6 assume heating from four sides.

4.4.3.2 *Beams of low aspect ratio.* A shielding effect occurs in I or H section beams of low aspect ratio, which reduces the heating rate of the web and inside faces of the flange, so the design temperature values given in table 7 should be reduced by the values given in table 8.

The aspect ratio should be taken as D_e/B_e

where

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

B_e is the width of its exposed bottom flange.

Table 7. Design temperature for beams

Flange thickness	Design temperature for fire resistance period of:			
	30 min	60 min	90 min	120 min
mm	°C	°C	°C	°C
⩽6.8	810	940	1000	1045
8.6	790	939	1000	1045
9.7	776	938	1000	1045
10.9	767	938	1000	1045
11.8	755	936	1000	1045
12.7	750	936	1000	1045
13.2	746	936	1000	1045
14.8	741	936	1000	1045
17.0	739	935	1000	1045
17.7	736	933	1000	1045
18.8	730	931	1000	1045
19.7	722	929	1000	1045
20.2	719	929	1000	1045
22.1	716	928	1000	1045
23.6	694	920	1000	1045
25.4	688	919	1000	1045
26.8	676	914	1000	1045
27.9	665	908	1000	1045
32.0	625	885	1000	1045
36.6	586	849	1000	1045

NOTE. The values in table 7 assume heating from three sides.

Table 8. Design temperature reductions

Aspect ratio D_e/B_e	Design temperature reduction for fire resistance period of:			
	30 min	60 min	90 min	> 90 min
	°C	°C	°C	°C
⩽0.6	80	40	20	0
>0.6 ⩽0.8	40	20	0	0
>0.8 ⩽1.1	20	0	0	0
>1.1 ⩽1.5	10	0	0	0
>1.5	0	0	0	0

NOTE. This table does not apply to channels or hollow sections.

4.4.4 Moment capacity method

This method is applicable only to beams which have webs which satisfy the requirements for a plastic or compact section as defined in BS 5950 : Part 1.

A beam whose temperature profile can be defined, may have its fire resistance assessed by calculating its moment capacity M_{cf} using the elevated temperature profile for the required period of fire resistance and the appropriate values of the strength reduction factor, given in **2.2**. 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 fire resistance without protection.

When the applied moment M_f at the fire limit state exceeds M_{cf} the protection thickness necessary to provide adequate fire resistance may be derived either from **4.3** or else from the calculation given in appendix D.

A simplified calculation method for beams with shelf angles is given in appendix E.

4.5 Portal frames

In buildings with fire resistant external walls which rely for their stability on the columns of portal frames which 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, and where appropriate, wind loading.

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.

A simple method of calculation for portal frame buildings is given in appendix F.

NOTE. For further information see [4].

4.6 Concrete-filled hollow section columns

4.6.1 General

The fire resistance of structural hollow sections manufactured in accordance with BS 4848 : Part 2, filled with ordinary dense structural concrete, with or without reinforcement, used as columns in simple construction in accordance with BS 5950 : Part 1 may be determined as follows.

When reinforcement is necessary, it may be either conventional high yield steel bar reinforcement in accordance with BS 4449 or else drawn steel fibre reinforcement spread uniformly throughout the concrete and forming approximately 5 % by dry mass of the constituents, before addition of water. The fibre shape and dimensions should be such as to provide an adequate pull-out strength. Typically, fibres should be 0.5 mm diameter, not longer than 38 mm, and have crimped flats or hooked ends to ensure adequate pull-out resistance.

Two vent holes, 12 mm diameter, should be provided in opposite faces of the column at the head and foot of every storey height or at a spacing of not more than 4 m centre-to-centre, whichever is smaller, to ensure adequate venting of any steam generated in the event of fire.

NOTE. For further information see [5].

4.6.2 Concrete-filled rectangular hollow sections

4.6.2.1 *Plain or fibre reinforcement.* Plain or fibre reinforced concrete-filled, hollow section columns not less than 140 mm square or 100 mm x 200 mm rectangular, should satisfy the following relationships at the fire limit state:

$$\frac{F_f + 6\left(\dfrac{M_{fx}}{d_x} + \dfrac{M_{fy}}{d_y}\right)}{0.83 K f_{cu} A_c} \leqslant \eta$$

and

$$6\left(\frac{M_{fx}}{d_x} + \frac{M_{fy}}{d_y}\right) \leqslant F_f$$

where

η is the time dependent load ratio obtained from table 9 for the relevant period of fire resistance;

f_{cu} is the characteristic concrete cube strength;

K is the concrete core buckling factor obtained from table 10;

F_f is the axial load at the fire limit state;

M_{fx} is the moment about major axis at fire limit state (always taken as positive);

M_{fy} is the moment about minor axis at fire limit state (always taken as positive);

d_x is the depth of concrete measured normal to major axis;

d_y is the depth of concrete measured normal to minor axis;

A_c is the area of concrete core.

Table 9. Time dependent load ratio η

Type of concrete	Load ratio for fire resistance period of:			
	30 min	60 min	90 min	120 min
Plain or bar reinforced	1.000	0.509	0.397	0.359
Fibre reinforced	1.000	0.678	0.534	0.473

4.6.2.2 *Bar reinforced concrete-filled sections.* Bar reinforced concrete-filled hollow section columns not less than 200 mm square or 150 mm x 250 mm rectangular, should satisfy the following relationship at the fire limit state:

$$\frac{F_f}{K(0.83 f_{cu} A_c + f_y A_r)\left(1 - \dfrac{M_{fx}}{M_{px}} - \dfrac{M_{fy}}{M_{py}}\right)} \leqslant \eta$$

where

M_{px} is the plastic moment capacity of the reinforcement about the major axis;

M_{py} is the plastic moment capacity of the reinforcement about the minor axis;

K, f_{cu}, A_c, F_f, M_{fx}, M_{fy} and η are as defined in **4.6.2.1**;

f_y and A_r are as defined in table 10.

The area of reinforcement should be not greater than 4 % of the core area and cover to the reinforcement should be in accordance with BS 8110 : Part 2. Fibre reinforced concrete should not be used in conjunction with bar reinforcement to increase fire resistance unless tests are used to determine the behaviour of such sections.

Table 10. Concrete core buckling factor K

L_E/r	K	L_E/r	K	L_E/r	K
14	1.000	70	0.703	130	0.283
20	0.984	80	0.610	140	0.247
30	0.953	90	0.521	150	0.217
40	0.905	100	0.444	160	0.193
50	0.852	110	0.379	170	0.171
60	0.786	120	0.326	180	0.154

NOTE.

L_E is the effective length of column;

r is the radius of gyration of the concrete core in the plane of buckling given by the following:

for bar reinforced columns:

$$r = 0.043 \times \sqrt{\left(\frac{450 f_{cu} I_c + E_r I_r}{0.83 f_{cu} A_c + f_y A_r}\right)}$$

for plain or fibre reinforced columns:

$$r = \sqrt{(I_c/A_c)}$$

where

I_c is the second moment of area of concrete core in the plane of buckling;

I_r is the second moment of area of the reinforcement in the plane of buckling;

E_r is the modulus of elasticity of reinforcement;

f_y is the characteristic yield strength of the reinforcement;

A_r is the area of reinforcement.

4.6.3 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 H_p/A 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

H_p/A	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 may be limited by the fixing system used or by the stability of the fire protection material.

4.7 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 [6].

4.8 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 [7].

4.9 Floor and roof slabs

4.9.1 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 : Part 4 should comply with **4.9.2** or **4.9.3**;

(b) all other concrete slabs should comply with BS 8110 : Part 2.

4.9.2 Unprotected composite slabs with profiled steel sheeting

4.9.2.1 *Design.* Where a slab is designed in accordance with BS 5950 : Part 4 it may be considered to have a fire resistance of 30 minutes 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 1. Design data for continuous composite slabs with mesh reinforcement may be obtained from [8] and [9].

Alternatively a fire engineering analysis may be carried out 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) At the fire limit state the plastic moment capacity of the slab may be used.

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

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

NOTE 2. For further information see [9].

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 **10.1.2** of BS 4449 : 1988.

4.9.2.2 *Thermal insulation requirement.* The minimum slab depth for thermal insulation (see figures 2 and 3) 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 : Part 4 recommends that the minimum depth of structural concrete over the profiled steel sheet should be 50 mm.

4.9.2.3 *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.

4.9.3 Protected composite slabs with profiled steel sheeting

The fire resistance of protected composite slabs with profiled steel sheeting may be assessed by tests in accordance with BS 476 : Part 21.

4.9.4 Composite beams

Composite beams should have their fire resistance assessed in the same way as non-composite beams, see **4.3** and **4.4**.

NOTE. For further information see [9].

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

Depth into slab (see note 2)	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

*Indicates a temperature greater than 800 °C.

NOTE 1. NW is ordinary dense structural concrete and LW is lightweight concrete.

NOTE 2. For any profile shape the depth into the concrete is measured normal to the surface of the profiled steel sheet (see figure 1).

Figure 1. Measurement of depth into concrete slab

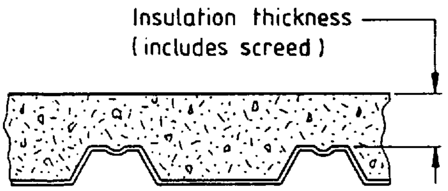

Figure 2. Insulation thickness for trapezoidal profiled steel sheets

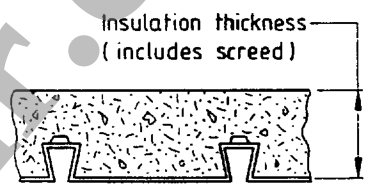

Figure 3. Insulation thickness for re-entrant profiled steel sheets

Table 13. Minimum thickness of concrete for trapezoidal profiled steel sheets (see figure 2)

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	95	115	130
Lightweight concrete	50	60	70	80	100	115

Table 14. Minimum thickness of concrete for re-entrant profiled steel sheets (see figure 3)

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

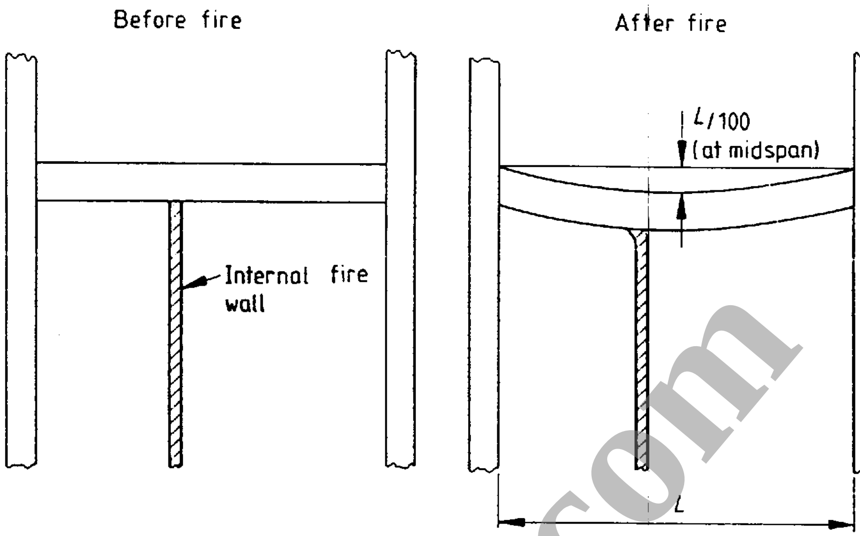

Figure 4. Effect of beam deflection on a fire-resisting wall

4.10 Walls

4.10.1 General

The appropriate thickness of fire protection to be applied to steel members incorporated into fire-resisting walls should be determined in accordance with **4.3** or by using the calculation given in appendix D. 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.

4.10.2 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 **4.10.3**.

4.10.3 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 4, 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.

4.10.4 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 [10].

4.11 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.

4.12 Ceilings

4.12.1 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 **4.12.2**.

4.12.2 Dry suspended ceiling systems

4.12.2.1 *General.* For structural fire protection the complete ceiling and floor or roof construction should be considered. Ceilings should be constructed in accordance with CP 290.

4.12.2.2 *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.

4.12.2.3 *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.

4.12.2.4 *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.

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

Appendices

Appendix A. Fire design flow chart

Fire design procedures are illustrated in figure 5.

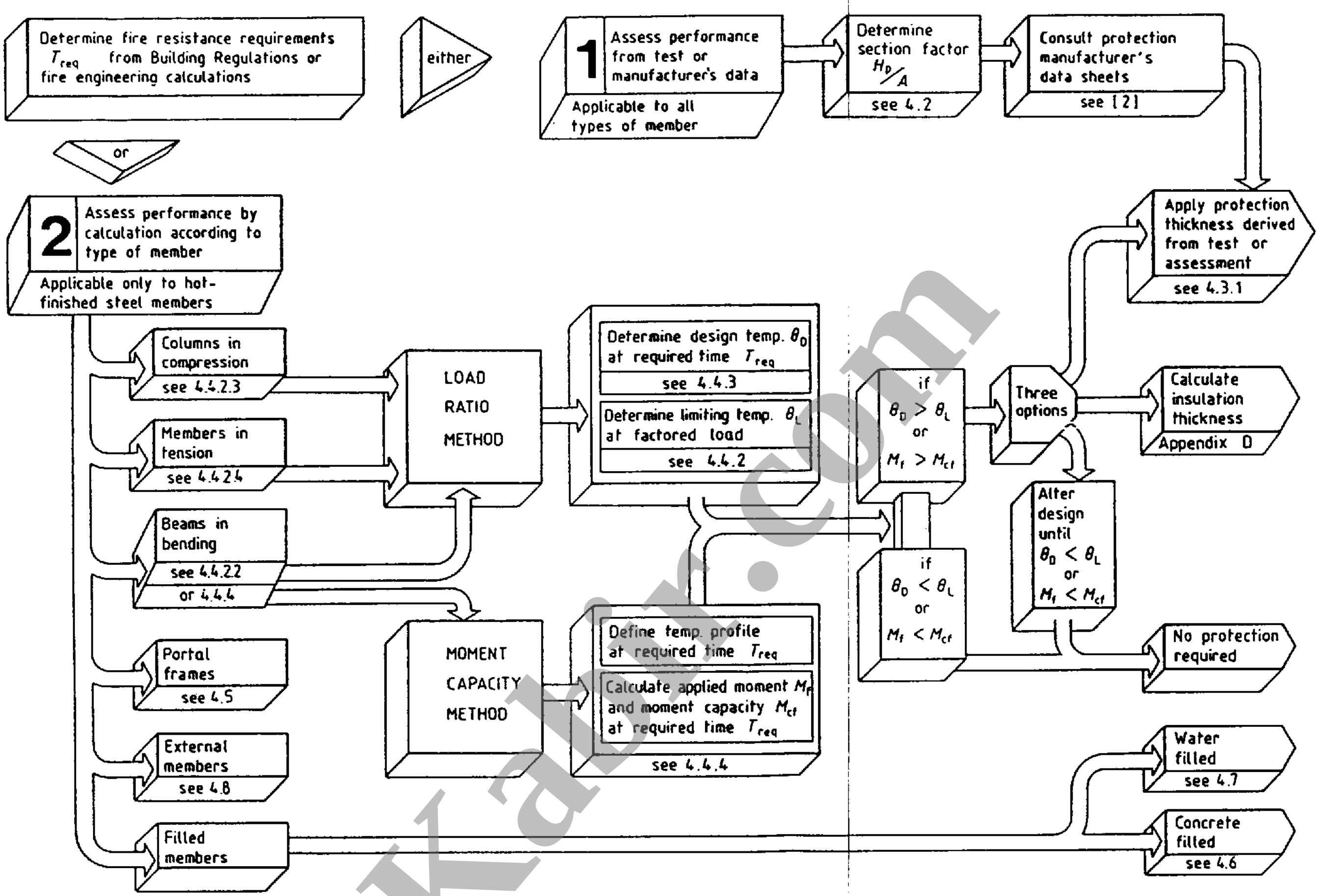

Figure 5. Fire design procedures

Appendix B. Strength reduction factors for cold formed steels complying with BS 2989

The strength reduction factors for cold formed members made from steels complying with BS 2989 may be taken from table 15. The appropriate value of strain should be determined from **2.3**.

Table 15. Strength reduction factors for cold formed steels complying with BS 2989

Strain	Strength reduction 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.

Appendix C. Re-use of steel after a fire

C.1 General

Structural steel may be re-used after a fire provided that its mechanical properties have not been significantly changed and that the members have not been distorted or damaged beyond the tolerances given in BS 5950 : Part 2. Members which have been distorted or damaged should be fully assessed to ensure their strength and suitability remain unimpaired. Members outside the fire affected zone should be checked, whenever possible, to ensure that there has been no distortion or other damage due to thermal expansion.

C.2 Temperature effects on strength

C.2.1 Hot finished steels

Except for partially exposed members, hot finished steel members may be re-used provided that they remain within the specified tolerances for straightness and shape and that the loads are to be unchanged.

Partially exposed members may be subject to very high temperatures without distortion. Such members should be tested by non-destructive methods to ensure compliance with appropriate British Standards.

C.2.2 Cast steel

Cast steel members may be re-used providing they remain within the specified tolerances for straightness, shape and area.

C.2.3 Cold finished steel

Cold finished steel up to grade Z35 of BS 2989 which remains within tolerance may be assumed to have 90 % of its original design strength after a fire and should be assessed accordingly. Members should be checked for tolerance and coating integrity; remedial works or replacement should be carried out accordingly.

C.3 Connections

C.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.

C.3.2 Bolted connections

Bolts may be distorted due to thermal expansion or softening during a fire. Where distortion is suspected then such bolts should be inspected and replaced as necessary.

C.3.3 Friction grip fasteners

Pretensioned friction grip bolts may be seriously affected by the heating experienced in a fire. All suspect bolts should be replaced by new bolts.

C.3.4 Welded connections

Welded connections should be treated in the same way as the parent material. They should also be checked to ensure that no cracking has occurred due to the effects of the fire.

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C.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.

C.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 [11].

Appendix D. Calculation of thickness of fire protection material

When the thermal and physical properties of a fire protection material other than an intumescent coating are known, the thickness t (in m) necessary to achieve the required period of fire resistance may be calculated from:

$$t = k_i \, I_f \, F_w \, (H_p/A)/10^6$$

where

I_f is the fire protection material insulation factor (in m^3/kW);

F_w is the fire protection material density factor;

H_p/A is the section factor (in m^{-1});

k_i is a function of the thermal properties of the fire protection material (W/m per degree centigrade).

The function k_i varies with temperature. Values should be derived from tests using representative values for the temperature and fire resistance period.

Values of the fire protection insulation factor I_f are given in table 16 for the required period of fire resistance and steel temperature.

The physical and thermal properties should be derived from the results of fire resistance tests in accordance with BS 476 : Parts 20 and 21. Thermal properties derived solely from small scale material tests are not applicable.

Tests should be carried out at an approved testing station and the properties derived from them should be approved by a suitably qualified person.

The fire protection material density factor F_w may be obtained from table 17, or calculated using:

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

The value of μ is given by:

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

where

ρ_i is the fire protection material density (in kg/m^3);

c is the fire protection material moisture content (in % by mass);

ρ_s is the steel density (in kg/m^3).

Table 17. Fire protection material density factor

μ	F_w
0.0	1.0
0.05	0.95
0.1	0.92
0.5	0.73
1.0	0.62
1.5	0.55
2.0	0.50

Table 16. Insulation factor I_f

Steel temperature	Insulation factor for fire resistance period of:					
	30 min	60 min	90 min	120 min	180 min	240 min
°C	m^3/kW	m^3/kW	m^3/kW	m^3/kW	m^3/kW	m^3/kW
400	500	1230	2100	3000	5100	7400
450	400	980	1650	2400	4100	5900
500	325	800	1360	1980	3350	4900
550	275	680	1150	1670	2850	4100
600	240	590	990	1440	2450	3550
650	210	510	870	1260	2150	3100
700	185	450	770	1120	1890	2750
750	165	405	690	1000	1690	2450
800	150	365	620	900	1530	2200

Appendix E. Simplified method of calculation for beams with shelf angles

E.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 **E.2** from the temperature profile in **E.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 50 steel, not less than 125 mm x 75 mm x 12 mm, fixed with the longer legs supporting the concrete slabs, and the vertical leg upwards, as shown in figures 6 to 8.

(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 reduction 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 reduction factor from table 1 for 0.5 % strain.

E.2 Calculation method

In the calculations, a constant strain across the section as given in **2.3** 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 **E.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.

E.3 Temperature profile

E.3.1 General

The dimensions and temperature blocks shown in figure 6 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.

E.3.2 Exposed steelwork

The temperature θ_1 of the lower flange (block 1) should be determined from **4.4.3**. 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 18, in which B_e and D_e are as defined in **4.4.3.2** (see also figure 6).

E.3.3 Embedded steelwork

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

$$\theta_x = \theta_R - Gx \quad \text{but } \theta_x \geqslant 300\ ^\circ\text{C}$$

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 7;

G is the temperature gradient (in °C/mm), given in table 19.

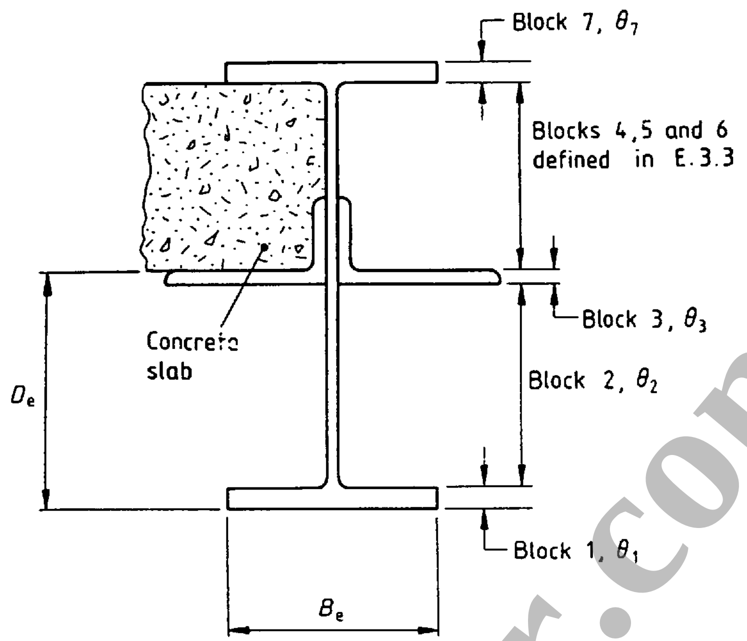


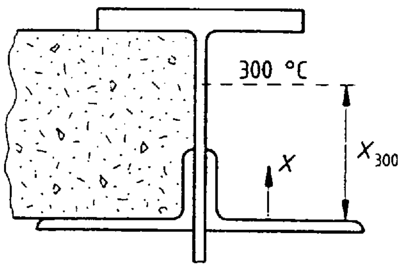
Figure 6. Temperature blocks for beams with shelf angles

Table 18. Block temperature

Aspect ratio	Block temperature for a fire resistance period of:								
	30 min			60 min			90 min		
	θ_2	θ_3	θ_R	θ_2	θ_3	θ_R	θ_2	θ_3	θ_R
	°C	°C	°C	°C	°C	°C	°C	°C	°C
$D_e/B \leqslant 0.6$	$\theta_1 - 140$	475	350	$\theta_1 - 90$	725	600	$\theta_1 - 60$	900	775
$0.6 < D_e/B \leqslant 0.8$	$\theta_1 - 90$	510	385	$\theta_1 - 60$	745	620	$\theta_1 - 30$	910	785
$0.8 < D_e/B \leqslant 1.1$	$\theta_1 - 45$	550	425	$\theta_1 - 30$	765	640	θ_1	925	800
$1.1 < D_e/B \leqslant 1.5$	$\theta_1 - 25$	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 19. Temperature gradient

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

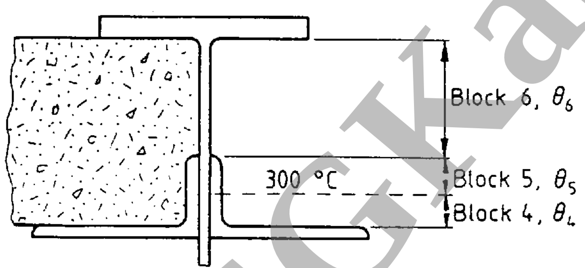


NOTE 1. The definitions of blocks 4, 5 and 6 vary depending on whether or not the 300 °C line lies within the depth of the shelf angles, see figure 8.

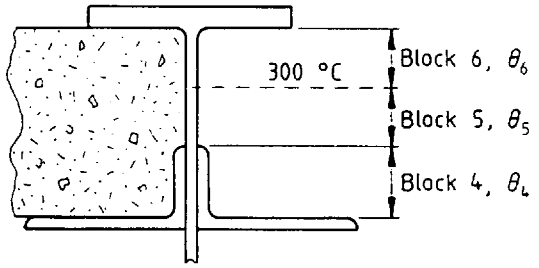
NOTE 2. The location of x_{300} is given by the following expression

$$x_{300} = \frac{\theta_R - 300}{G}$$

Figure 7. Definition of dimension *x*



(a) In angle leg



(b) Above angle

Figure 8. Definition of blocks 4, 5 and 6

Appendix F. Simple method of calculation for portal frame buildings

F.1 Symmetric ridged or flat roofed portal frames

Portal frames, fabricated from universal sections, designed in accordance with BS 5950 : Part 1, may be checked for compliance with the recommendations given in **4.5** at the fire limit state using the following rules. They are not applicable to frames with tapered rafters, for which reference should be made to specialist literature.

This method applies to steel portal frames which satisfy the following.

(a) The frame should be constructed from hot rolled I or H sections. The column may he tapered but the rafter may not.

(b) The frame may or may not have haunches.

(c) The frame may be single bay or multi-bay.

(d) The rafter adjacent to the boundary should be symmetrical about the centre of its span.

(e) The ratio of the span of the rafter adjacent to the boundary, to the height to the eaves should be not less than 1.6.

(f) The columns in the boundary wall should be adequately restrained in the plane of the wall, see **F.4**.

(g) The columns on the affected boundary should have the same fire resistance as the wall, up to the underside of the haunch, or up to the underside of the rafter if no haunch exists.

The bases and foundations of any column on an affected boundary should be designed, at the fire limit state, to resist the following forces and overturning moment.

(1) vertical reaction:

$0.5\, W_f S L$ + dead weight of wall

(2) horizontal reaction:

$K(W_f SGA - M_{pr} C/G)$ but $\geqslant M_{pc}/10Y$

(3) overturning moment:

$K\,\{W_f SGY(A + B/Y) - M_{pr}(CY/G - 0.065)\}$ but $\geqslant M_{pc}/10$

where

W_f is the factored load at time of collapse (in kN/m^2);

S is the spacing of frames, centre-to-centre (in m);

G is the clear span between ends of haunches (in m);

Y is the vertical height at end of haunches (in m);

L is the overall span (in m);

M_{pc} is the plastic moment of resistance of column at 20 °C (in kN·m);

M_{pr} is the plastic moment of resistance of rafter at 20 °C (in kN·m);

A and C are factors obtained from table 20;

K is a modification factor, taken as 1 for a single bay frames or obtained from table 21 for multi-bay frames;

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

For frames without haunches, Y should be taken as the column height and G should be taken as equal to the span L. Thus B is zero.

Table 20. Factors A and C for various rafter pitches

Rafter pitch	A	C
degrees		
0	1.01	1.05
3	0.99	1.02
6	0.93	0.96
9	0.85	0.88
12	0.76	0.79
15	0.68	0.70
18	0.61	0.62
21	0.54	0.56
24	0.49	0.50
27	0.44	0.45
30	0.40	0.41

Table 21. Modification factor K for multi-bay frames

Pitch	Range of span/height ratio	Modification factor
$\leqslant 3°$	$\geqslant 2.5$	1.0
	$\geqslant 1.7$ < 2.5	1.3
$\leqslant 6°$	$\geqslant 2.3$	1.0
	$\geqslant 1.6$ < 2.3	1.3
$\leqslant 9°$	$\geqslant 2.1$	1.0
	$\geqslant 1.6$ < 2.1	1.3
$\leqslant 12°$	$\geqslant 1.8$	1.0
	$\geqslant 1.6$ < 1.8	1.3
$> 12°$	$\geqslant 1.6$	1.0

F.2 Columns and bases

When designing columns and bases to resist overturning due to rafter collapse the following γ_m factors should be used:

(a) holding down bolts — 1.0 against yield strength; or 1.2 against ultimate tensile strength, whichever is more onerous;

(b) baseplate — 1.2 against formation of a plastic hinge;

(c) column — 1.2 against formation of a plastic hinge.

F.3 Foundations

Foundations should be designed to ensure that the ultimate bearing capacity of the soil is not exceeded.

F.4 Restraint

The requirement for restraint in the plane of the wall (item (f) of F.1) should be met by satisfying the following conditions:

(a) either:

(1) by providing four equal diameter holding down bolts in each base plate, spaced symmetrically about the section in the longitudinal direction, at a minimum spacing equal to 70 % of the flange width, or

(2) by the stanchion being set in a concrete base which is capable of resisting an overturning moment in the longitudinal direction equal to that resisted by the holding down bolts in item (1) above.

(b) and either:

(1) by providing a masonry wall with a height not less than 75 % of the height to the eaves, which is connected to the column and which restrains the column in the plane of the wall at normal temperatures, or

(2) in any other case by designing the horizontal members which restrain the column in the plane of the wall, as steel members to the appropriate Part of BS 5950. These members do not require fire protection.

Table 22. Percentage dead weight of roof cladding systems remaining at time of rafter collapse

Systems	Roof cladding					
	Inner lining		Insulation		Outer covering	
	Material	Percentage dead weight	Material	Percentage dead weight	Material	Percentage dead weight
		%		%		%
Site assembled constructions	Mineral insulation board	100	Glass or mineral fibre Thermoplastic foams Thermosetting foams	100 0 70	Steel Aluminium Fibre cement	100 100 100
	Plaster board	0	Bonded thermoplastic foams Glass or mineral fibre Unbonded foams	0 0 0	Steel Aluminium Fibre cement	100 10 10
	Plaster board	50	Bonded thermosetting foams	50	Steel Aluminium Fibre cement	100 50 50
	Steel	100	Glass or mineral fibre Thermoplastic foams Thermosetting foams	100 0 70	Steel Aluminium Fibre cement	100 100 50
			Fibre insulating board	70	Steel Aluminium Fibre cement	100 100 100
	Thin linings, e.g. foil, embossed papers and plastics	0	Mineral or glass fibre Most foamed plastics	0 0	Steel Aluminium Fibre cement	100 0 10
	Aluminium	0	Phenolic foams	50	Steel Aluminium Fibre cement	100 50 50
	Fibre cement	10	Unbonded glass or mineral fibre Thermosetting foams Thermoplastic foams	10 10 0	Fibre cement	10
Factory assembled bonded systems	Foil, paper or plastics facings	0	Urethane or isocyanurate foams Phenolics	70 80	Steel Aluminium	100 100
	Steel	100	Thermosetting foams	80	Any	100
	Aluminium	0	Thermosetting foams	100	Any	100
Roof lights	Single skin plastics rooflights	0				
	Double skin plastics rooflights	50				

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BS 476	Fire tests on building materials and structures Part 20 Method for determination of the fire resistance of elements of construction (general principles) Part 21 Methods for determination of the fire resistance of load bearing elements of construction
BS 2989	Specification for continuously hot-dip zinc coated and iron-zinc alloy coated steel: wide strip, sheet/plate and slit wide strip
BS 4360	Specification for weldable structural steels
BS 4449	Specification for carbon steel bars for the reinforcement of concrete
BS 4848	Specification for hot-rolled structural steel sections Part 2 Hollow sections
BS 5950	Structural use of steelwork in building Part 1 Code of practice for design in simple and continuous construction: hot rolled sections Part 2 Specification for materials, fabrication and erection: hot rolled sections Part 4 Code of practice for design of floors with profiled steel sheeting
BS 8110	Structural use of concrete Part 2 Code of practice for special circumstances
CP 290	Code of practice for suspended ceilings and linings of dry construction using metal fixing systems

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British Constructional Steelwork Association Ltd.
British Railways Board
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Department of the Environment (Housing and Construction Industries)
Department of the Environment (Property Services Agency)
Health and Safety Executive
Institution of Civil Engineers
Institution of Structural Engineers
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Structural use of steelwork in building
Part 8. Code of practice for fire resistant design

Corrections

Clause 1.3 Major symbols

In line 5, Symbol for 'M', delete and substitute 'M_f'.

AMD 8858/November 1995

Clause 4.4.2.3 Load ratio for columns

In item (b), lines 6 and 7, equations for 'R', delete 'F' in both cases and substitute 'F_f'.

In the list of definitions, line 1, delete 'F' and substitute 'F_f'.

AMD 8858/November 1995

Clause 4.4.2.4 Tension members

In the list of definitions, line 1, delete 'F', and substitute 'F_f'.

AMD 8858/November 1995

Table 13. Minimum thickness of concrete for trapezoidal profiled steel sheets (see figure 1)

In the heading, delete '(see figure 1)' and substitute '(see figure 2)'.

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Table 14. Minimum thickness of concrete for re-entrant profiled steel sheets (see figure 2)

In the heading, delete '(see figure 2)' and substitute '(see figure 3)'.

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Appendix G Bibliography

In item 7, line 3, delete the date '1981' and substitute '1989'.

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