PD 7974-3:2011

BSI Standards Publication

PUBLISHED DOCUMENT

Application of fire safety engineering principles to the design of buildings

Part 3: Structural response and fire spread beyond the enclosure of origin

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ISBN 978 0 580 60070 8

ICS 13.220.20; 91.040.01

The following BSI reference relates to the work on this Published Document:
Committee reference FSH/24

Publication history

First published January 2003
Second (present) edition, November 2011

Amendments issued since publication date

Text affected

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Foreword

Publishing information

This Published Document is published by BSI and came into effect on 30 November 2011. It was prepared by Technical Committee FSH/24, *Fire Safety Engineering*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

This Published Document supersedes PD 7474-3:2003, which is withdrawn.

Information about this document

This Published Document (PD) is one of a series of documents published under the Fire Standards Policy Committee, and is a supporting document to BS 7974, *Code of Practice on the Application of Fire Safety Engineering Principles to the Design of Buildings*. Other documents in the series are:

PD 7974-0: *Guide to design framework and fire safety engineering procedures*;

PD 7974-1: *Initiation and development of fire within the enclosure of origin*;

PD 7974-2: *Spread of smoke and toxic gases within and beyond the enclosure of origin*;

PD 7974-4: *Detection of fire and activation of fire suppression systems*;

PD 7974-5: *Fire service intervention*;

PD 7974-6: *Evacuation*;

PD 7974-7: *Probabilistic risk assessment*.

Presentational conventions

The provisions of this Published Document are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is “should”.

Commentary, explanation and general informative material is presented in smaller, italic type, and does not constitute a normative element.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a Published Document cannot confer immunity from legal obligations.

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Introduction

This Published Document is one of a series of documents intended to support BS 7974, *Code of Practice on the Application of Fire Safety Engineering Principles to the Design of Buildings*.

This and the other Published Documents (PDs) contain "state of the art" guidance and information on how to undertake quantitative and detailed analysis of specific aspects of design. It is intended that they be updated as new theories, calculation methods and/or data become available.

However, it is important to recognize that the information contained within PD 7974-3 does not preclude data, information or methods of analyses from other sources, such as published peer reviewed research, manufacturers' data or codes of practice prepared on behalf of the construction materials industry, professional engineering and technical institutions and other professional bodies.

BS 7974 was first published in 2001. Since then there have been substantial changes in understanding in the behaviour of fire in the built environment and how materials and construction systems respond at elevated temperatures. Not least, the structural Eurocodes on Fire Actions have been published as full European Standards. These have resulted in revised formulations on the behaviour of structural components in fire, as well as new data on the thermal and mechanical properties of the various materials used in building construction. One of the most significant and recent advances in the understanding of buildings in fire has come about as a result of studies of experimental major fires in full size structures and the ensuing guidance this has generated on analysing the structural behaviour of the framework and compartmentation.

However, where understanding the behaviour of construction systems and building products cannot be quantified, or there are no specific analyses of some aspects of fire spread beyond the enclosure of origin other than the performance of products based upon a fire resistance furnace test, a commentary is given on the particular issues that need to be considered and how these could be treated.

A fire-safety engineering approach that takes into account the total fire safety package can provide a more economical solution than prescriptive approaches to fire safety. In some cases, it is the only viable means of achieving a satisfactory standard of fire safety in some large and complex buildings.

A major issue in the determination of the structural response is the application of time equivalent methods in specifying an equivalent period of heating in the standard fire resistance test furnace. Any outputs should consider the consequences of failure in relation to the particular occupancy and building dimensions (height and compartment size) and its location in the building, for example, BS 9999 specifies a risk-based approach for occupant life safety in building structures.

Fire safety engineering has many benefits. The use of BS 7974 facilitates the practice of fire safety engineering and, in particular:

a) provides the designer with an organized approach to fire safety design;

b) allows the safety levels for alternative fire safety designs to be compared;

c) provides a basis for selection of appropriate fire protection systems;

d) provides opportunities for innovative design; and

e) provides information on the management of fire safety for a building.

Fire is an extremely complex phenomenon and there are still gaps in the available knowledge. When used by suitably qualified persons experienced in fire safety engineering, BS 7974 and its associated PDs provide a means of establishing acceptable levels of fire safety economically, without impeding building design.

For the purpose of this Published Document, spread of fire beyond the enclosure of origin is deemed to have taken place when any material outside of the fire enclosure ignites or suffers thermal degradation. Structural response is the interaction of loadbearing and non-loadbearing elements or frames as a result of thermal and/or mechanical actions due directly or indirectly to a fire. The level of sophistication employed to evaluate fire spread can vary. For example, a simple decision can be taken that the creation of any openings or gaps in the enclosure boundaries precipitates fire spread. Alternatively, more complex analyses can be employed to consider whether flames project from openings in the enclosure's boundaries and whether such flames ignite or degrade materials outside the enclosure.

Throughout PD 7974-3 reference is made to relevant codes of practice. Where appropriate, relevant extracts are provided in order to assist the reader in understanding the design methodologies presented and to compare and contrast between different approaches or sources of data. It is therefore important that PD 7974-3 is not used in isolation and reference is made to the relevant codes of practice, particularly in relation to additional notes and sub-clauses describing its application (see Figure 1).

1 Scope

This Published Document provides a framework for developing a rational methodology for design using a fire safety engineering approach through the application of scientific and engineering principles to the protection of people, property and the environment from fire. The Published Document considers the following issues:

a) the conditions within a fire enclosure and their potential to cause fire spread;

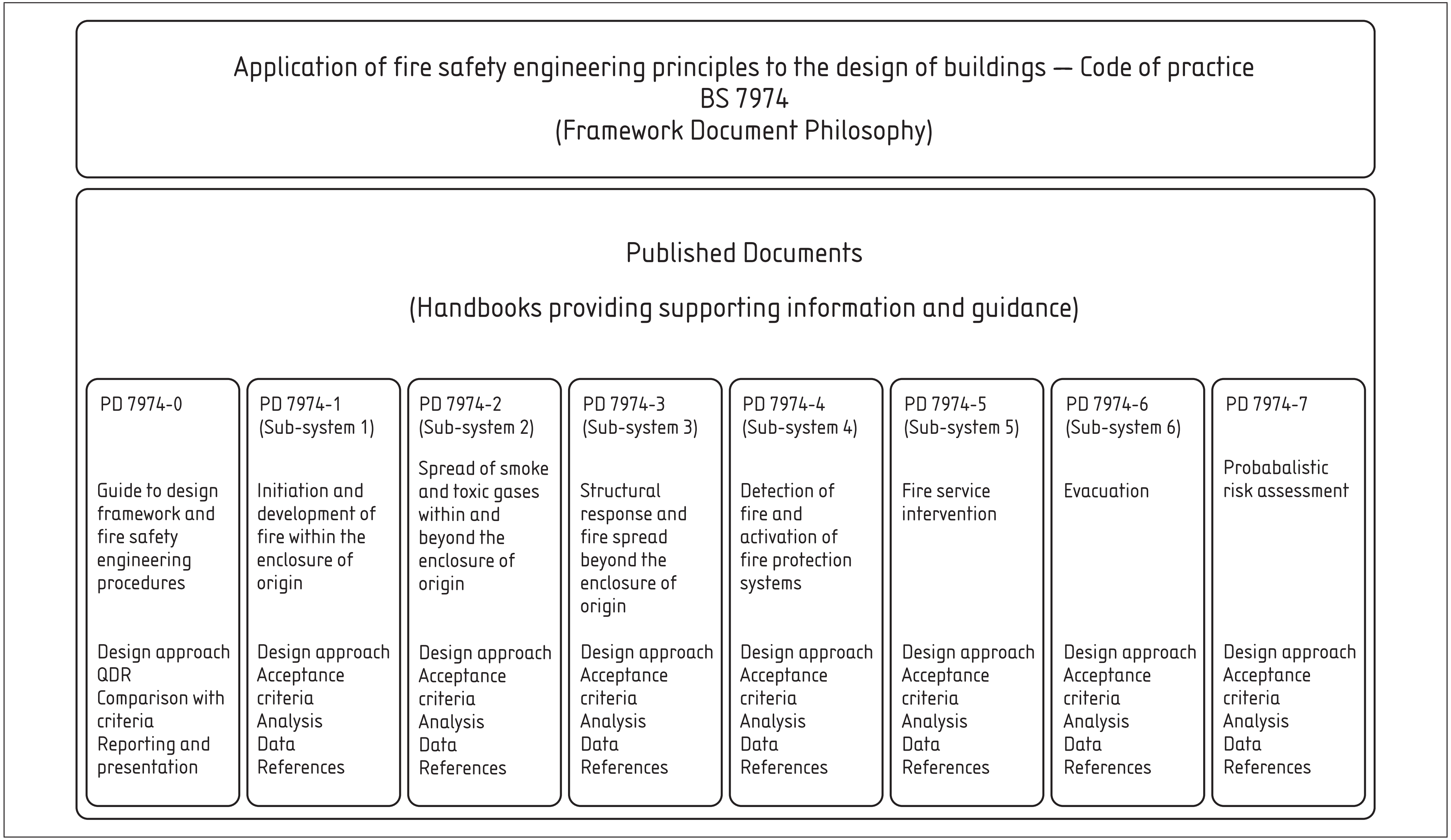
b) the thermal and mechanical response of the enclosure boundaries and its structure to the fire conditions;

c) the impact of the anticipated thermal and mechanical responses on adjacent enclosures and spaces;

d) the structural responses of loadbearing elements and their effect on structural stability, load transfer and acceptable damage according to the design and purpose of the building;

e) fire following structural impact to the building.

Figure 1 **Overview of the PD 7974 series of Published Documents**



2 Normative references

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

BS 476:1932, *British standard definitions for fire resistance, incombustibility and non-inflammability of building materials and structures (including methods of test)*

BS 476-3, *Fire tests on building materials and structures – Part 3: Classification and method of test for external fire exposure to roofs*

BS 476-4, *Fire tests on building materials and structures – Part 4: Non-combustibility test for materials*

BS 476-20:1987, *Fire tests on building materials and structures – Part 20: Method for determination of the fire resistance of elements of construction (general principles)*

BS 476-21, *Fire tests on building materials and structures – Part 21: Methods for determination of the fire resistance of loadbearing elements of construction*

BS 476-22, *Fire tests on building materials and structures – Part 22: Methods for determination of the fire resistance of non-loadbearing elements of construction*

BS 476-31.1, *Fire tests on building materials and structures – Part 31: Methods for measuring smoke penetration through doorsets and shutter assemblies – Section 31.1: Method of measurement under ambient temperature conditions*

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

BS EN 81-58, *Safety rules for the construction and installation of lifts – Part 58: Examination and tests – Landing doors fire resistance test*

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

BS EN 1363-2:1999, *Fire resistance tests – Part 2: Alternative and additional procedures*

BS EN 1364, *Fire resistance tests for non-loadbearing elements*

BS EN 1364-1, *Fire resistance tests for non-loadbearing elements – Part 1: Walls*

BS EN 1364-3, *Fire resistance tests for non-loadbearing elements – Part 3: Curtain walling – Full configuration (complete assembly)*

BS EN 1365, *Fire resistance tests for loadbearing elements*

BS EN 1365-2, *Fire resistance tests for loadbearing elements – Part 2: Floors and roofs*

BS EN 1366-1, *Fire resistance tests for service installations – Part 1: Fire resistance tests for service installations – Ducts*

BS EN 1366-2, *Fire resistance tests for service installations – Part 2: Fire dampers*

BS EN 1366-3, *Fire resistance tests for service installations – Part 3: Penetration seals*

BS EN 1366-4, *Fire resistance tests for service installations – Part 4: Linear joint seals*

BS EN 1366-6, *Fire resistance tests for service installations – Part 6: Raised access and hollow core floors*

BS EN 1991, *Eurocode 1 – Actions on structures*

BS EN 1991-1-2:2002, *Eurocode 1 – Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire*

BS EN 1992-1-2, *Eurocode 2 – Design of concrete structures – Part 1-2: General rules – Structural fire design*

BS EN 1993-1-2:2005, *Eurocode 3 – Design of steel structures – Part 1-2: General rules – Structural fire design*

BS EN 1994, *Eurocode 4 – Design of composite steel and concrete structures*

BS EN 1994-1-2:2005, *Eurocode 4 – Design of composite steel and concrete structures – Part 1-2: General rules – Structural fire design*

BS EN 1995-1-2, *Eurocode 5 – Design of timber structures – Part 1-2: General – Structural fire design*

BS EN 1996-1-1:2005, *Eurocode 6 – Design of masonry structures – Part 1-1: General rules for reinforced and unreinforced masonry structures*

BS EN 1996-1-2, *Eurocode 6 – Design of masonry structures – Part 1-2: General rules – Structural fire design*

BS EN 1999-1-2, *Eurocode 9 – Design of aluminium structures – Part 1-2: Structural fire design*

BS EN 10025-1, *Hot rolled products of structural steels – Part 1: General technical delivery conditions*

BS EN 10025-2, *Hot rolled products of structural steels – Part 2: Technical delivery conditions for non-alloy*

BS EN 10080, *Steel for the reinforcement of concrete – Weldable reinforcing steel – General*

BS EN 10147, *Continuously hot-dip zinc coated structural steels strip and sheet – Technical delivery conditions*

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

BS EN 10346, *Continuously hot-dip coated steel flat products – Technical delivery conditions*

BS EN ISO 13943, *Fire safety – Vocabulary*

BS ISO 10294-1, *Fire-resistance tests – Fire dampers for air distribution systems – Part 1: Test method*

BS ISO 10294-4, *Fire-resistance tests – Fire dampers for air distribution systems – Part 4: Test of thermal release mechanism*

BS ISO 10294-5, *Fire-resistance tests. Fire dampers for air distribution systems – Part 5: Intumescent fire dampers*

BS ISO 10295-1, *Fire tests for building elements and components. Fire testing of service installations – Part 1: Penetration seals*

BS ISO 22899-1, *Determination of the resistance to jet fires of passive fire protection materials – Part 1: General requirements*

BS ISO/TR 12470:1998, *Fire resistance tests – Guidance on the application and extension of results*

DD ENV 1187, *Test methods for external fire exposure to roofs*

ISO 834-1, *Fire resistance tests – Elements of building construction – Part 1: General requirements*

ISO 834-4, *Fire-resistance tests – Elements of building construction – Part 4: Specific requirements for loadbearing vertical separating elements*

ISO 834-5, *Fire-resistance tests – Elements of building construction – Part 5: Specific requirements for loadbearing horizontal separating elements*

ISO 834-8, *Fire-resistance tests – Elements of building construction – Part 8: Specific requirements for non-loadbearing vertical separating elements*

ISO 834-9, *Fire-resistance tests – Elements of building construction – Part 9: Specific requirements for non-loadbearing ceiling elements*

ISO 3008, *Fire-resistance tests – Door and shutter assemblies*

ISO 5925-1, *Smoke-control door and shutter assemblies – Part 1: Ambient- and medium-temperature leakage tests*

ISO 10295-2, *Fire tests for building elements and components – Fire testing of service installations – Part 2: Linear joint (gap) seals*

ISO 12468-1, *External exposure of roofs to fire – Part 1: Test method*

ISO/TR 15658, *Fire resistance tests – Guidance for the design and conduct of non-furnace-based large-scale tests and simulation*

PD 7974-0, *Application of fire safety engineering principles to the design of buildings – Part 0: Guide to design framework and fire safety engineering procedures*

PD 7974-1, *Application of fire safety engineering principles to the design of buildings – Part 1: Initiation and development of fire within the enclosure of origin (Sub-system 1)*

PD 7974-2, *Application of fire safety engineering principles to the design of buildings – Part 2: Spread of smoke and toxic gases within and beyond the enclosure of origin (Sub-system 2)*

PD 7974-4, *Application of fire safety engineering principles to the design of buildings – Part 4: Detection of fire and activation of fire protection systems (Sub-system 4)*

PD 7974-5, *Application of fire safety engineering principles to the design of buildings – Part 5: Fire service intervention (Sub-system 5)*

PD 7974-6, *The application of fire safety engineering principles to fire safety design of buildings – Part 6: Human factors: Life safety strategies – Occupant evacuation, behaviour and condition (Sub-system 6)*

PD 7974-7, *Application of fire safety engineering principles to the design of buildings – Part 7: Probabilistic risk assessment*

3 Terms and definitions

For the purposes of this Published Document the terms and definitions given in BS EN ISO 13943 and the following apply.

3.1 calculations (in support of extended application)
calculation methods that can be applied to one or more parameters of a tested construction and which are based on existing physical laws,

or have been empirically validated and form part of the process of defining the extended application

3.2 consequence

damage that would occur if the time-equivalent period was exceeded

NOTE With respect to Approved Document B of the Building Regulations for England & Wales [1]*, consequence is directly proportional to height.*

3.3 construction parameter

aspect of the design and construction of an element that may be varied and which can result in a change in the fire resistance performance

NOTE For example, a change in one or more of the dimensions of a stud in a stud framed separating element.

3.4 direct application

variation(s) in the construction and the limits of use for the element which, without further analysis, are covered by the test result in respect to the defined performance characteristics given in Interpretative Document 2 [2] and BS EN 13501-1, achieved from a fire resistance test in accordance with the appropriate European standard

NOTE 1 Direct application is arrived at by the application of simple rules that are known, or considered by the fire community, to give equal or improved fire resistance performance by the users. The rules can be used by non-fire experts.

NOTE 2 Only results from one test report can be used when considering a change of an element. Any combination and use of two or more tests reports or other technical sources is regarded as extended application and dealt with accordingly.

3.5 duration of steady burning

interval between onset of flashover and commencement of decay

3.6 enclosure

space defined by boundary elements (on all sides) around the point of origin of a fire

3.7 expert assessment

engineering analysis carried out by a suitably qualified person so that the results of a fire resistance test can be applied to a building element in which the dimensions and construction detail are different to that tested

3.8 expert judgement

qualitative process carried out by a suitably qualified person when the complexity of the influence is beyond the scope of rules, to establish the resultant effect of a variation in one or more parameters on the classification awarded

3.9 extended application

variations in the construction to establish the limits of use for an element that has been tested in accordance with the appropriate European standard, based upon an analysis by a suitably qualified person

NOTE The extended application can use the results from one or more test reports and can be based upon rules, calculations and expert judgement. As a result of the extended application, the fire resistance classification of an element with respect to defined performance characteristics given in Interpretive Document 2[2] and BS EN 13501-1 can be maintained, increased or decreased when used in practice.

3.10 factor
one of the possible variations that can be applied to a parameter

NOTE For example, a change in the stiffness as a result of a dimensional change in the stud.

3.11 factor influence
potential cause of a change in the fire resistance recorded by test, with respect to one or more criteria when a factor is changed

NOTE For example, an increase in the loadbearing capacity (R) as a result of an increase in stiffness.

3.12 fire safety
safety of a building and its surroundings in relation to life, property, business continuity and the environment

3.13 fire safety engineering
use of engineering principles for the achievement of fire safety

3.14 frequency
measure of the number of fires that are likely to occur in a particular building in a given period

3.15 isolated fire
fire that is fuel-bed controlled and is sufficiently small that it does not directly impact upon the enclosure

3.16 probability
likelihood of failure directly related to cumulative distribution curves of the time-equivalent period

NOTE Probability is derived using a Monte Carlo analysis to evaluate many thousands of fires with all relevant variables that influence fire severity.

3.17 protected enclosure
place of relative safety from the effects of fire

NOTE The effects of fire can include smoke and heat.

3.18 risk
frequency × probability × consequence of failure

3.19 rules
quantitative factors that can be applied to the result of tests when defining the limits of application as a product of research and testing

NOTE Rules are primarily used in determining the direct application of the result, as their application does not require specialist knowledge.

3.20 sensitivity analysis
calculation of changes in outputs for variations of an input parameter of interest

3.21 structural frames
arrangement of structural materials and/or elements coming together to form a building or part thereof designed to fulfil a loadbearing function

3.22 structural response
interaction of loadbearing and non-loadbearing elements or frames as a result of thermal and/or mechanical actions due directly or indirectly to a fire

3.23 thermal and mechanical parameters
aspect of the conditions of a test that can vary in practice and influence the classification system given

NOTE For example, the greater pressure differential that exists at the top of a larger element than the pressure differential at the top of the test specimen.

3.24 time equivalent
duration of exposure to standard BS 476-20:1987, BS EN 1363-1 or ISO 834-1

4 Symbols and abbreviated terms

α	coefficient of thermal expansion (mm/mmK^{-1})
α_c	coefficient of heat transfer by convection (W/m^{2}K)
α_h	area of horizontal openings in the roof related to the floor area of the compartment (m^2)
α_v	area of vertical openings in the façade related to the floor area of the compartment (m^2)
β_0	design charring rate for one dimensional charring (mm/min)
β_n	notional design charring rate (mm/min)
β_{par}	design charring rate under parametric heating conditions (mm/min)
$\gamma_{M,fi}$	partial safety factor in fire
γ_G	partial safety factor for permanent loads to be assigned a value of 1.0
Γ	compartment time factor
Δ_{bow}	lateral deflection of a wall (mm)
Δ_{head}	deflection of head of wall away from the heat source (mm)
ε	strain, emissivity
ζ	reinforcing efficiency parameter of the composite material indicating the extent to which the applied force is transmitted to the reinforcing phase
θ	temperature (°C or K)
λ	thermal conductivity (W/mK)
ρ	density (kg/m^3)
σ	Stefan Boltzman constant (5.67 × 10^{-8} W/m^{2}K^4)
σ	stress (N/mm^2)
χ_{fi}	reduction factor for flexural buckling in the fire design situation
ψ	partial safety factor
a	effective height (m)
a_0, a_1, a_2	coefficients for thermal conducitivity
A	area (m^2)
b	thermal inertia (J/m^{2}s^{½}K)
ct	combined thickness

C	specific heat capacity (J/kgK)
$d_{char,n}$	depth of charring (mm)
$d_{char,0}$	charring depth for one dimensional charring (mm)
d_{door}	thickness of a door leaf (mm)
d_{ef}	the effective cross section (mm)
d_i	thickness of insulating material, i (m)
d_t	depth of a timber beam (mm)
d_w	thickness of a wall (m)
D	depth of enclosure (m)
$e_{\Delta\theta}$	eccentricity due to variation of temperature across masonry
erfc	complex error function
E	Young's modulus (kN/mm^2)
E	integrity criteria
E	stress-strain slope (kN/mm^2)
$E_{d,t}$	design load created by the fire situation at time t
$E_{fi,d}$	design effect of actions for the fire situation
E_p	plastic modulus (kN/mm^2)
$E_{p,\theta}$	plastic modulus at temperature θ (kN/mm^2)
F	load (kN)
F_t	load at fire temperature
F_0	load at ambient temperature
f	strength (N/mm^2)
$F_{e\text{-}R}$	configuration factor describing the spatial relationship between the emitting and receiving surfaces
h	height (mm or m)
h_{net}	net incident heat flux per unit area (kW/m^2)
H	height of the enclosure (m)
H_p	heated perimeter of a section (m)
I	insulation criteria
I_{mean}	average temperature rise on homogeneous elements
k	thermal conductivity (W/mK)
k	modification factor
k_b	factor describing the thermal properties of the enclosure
k_c	reduction factor
K	thermal diffusivity (m^2/s)
L	linear dimension (mm)
M	moment (Nm)
n	ratio of temperatures
N	design value
$N_{b,fi,t,Rd}$	design buckling resistance
N_{Ed}	design value of the vertical load

O	opening factor ($m^{½}$)
p_c	compressive strength of concrete (N/mm^2)
p_y	characteristic design strength for steel (N/mm^2)
P_w	% of moisture (by mass)
P_f	effective property of the fibres
P_m	effective property of the matrix
q	heat flux (kW/m^2)
q	heat flow (kW)
Q	rate of heat flow (kW)
r	radius
R	mechanical resistance
R	loadbearing capacity criteria
S_{05}	5% fractile of a stiffness property (modulus of elasticity or shear modulus) at ambient temperature
S_{20}	20% fractile of a stiffness property (modulus of elasticity or shear modulus) at ambient temperature
t	time (s, min or h)
T	temperature (°C or K)
V	volume (m^3)
V_i	volume per unit length of an insulated element (m^3)
V_x	matrix volume fraction of a composite
w	width of the opening (m)
w_f	width of the flame front (m); ventilation factor
W	width of enclosure (m)
x	horizontal projection of the flame (m)
x_s	distance from exposed surface (m)
X	flame length along axis (m)
z	vertical projection of the flame above the window (m)
Z	flame height above opening (m)
Z_y	elastic modulus about the minor axis (cm^4)
Z_w	height above the top of the opening (m)

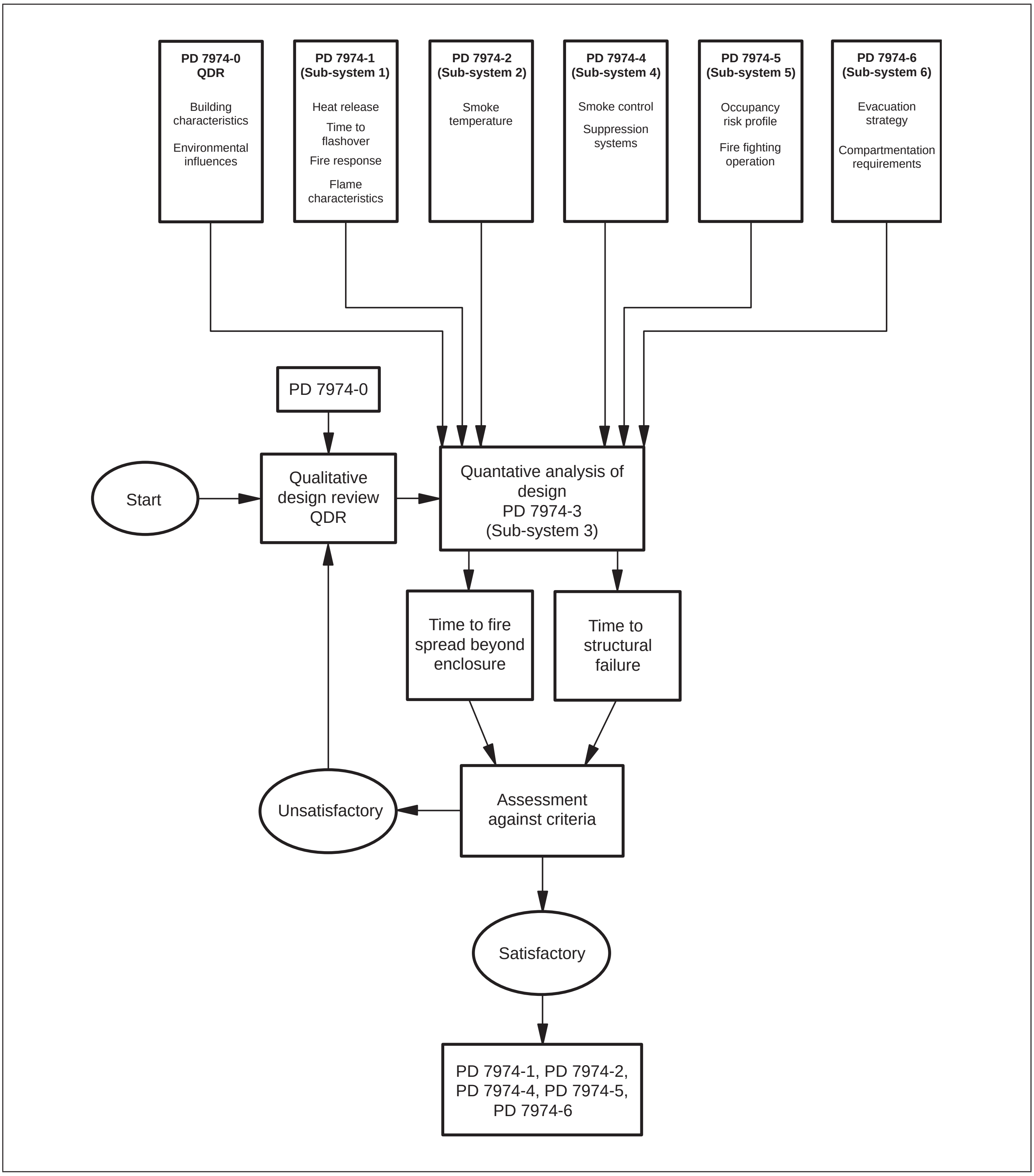
5 Interaction of BS 7974 sub-systems

5.1 General

The BS 7974 sub-systems, or Published Documents, are interrelated and have to be used together in order to achieve a satisfactory solution for the fire safety design of buildings. Decisions made using PD 7974-3 could impact upon other fire safety issues either directly or indirectly and it should not be used in isolation.

The relationships between the inputs to PD 7974-3 and the other sub-systems are shown in Figure 2.

Figure 2 **Inter-relationship between PD 7974-3 and the other sub-systems**



PD 7974-7 has inputs to all or part of the sub-systems and provides guidance on the application for probabilistic risk assessment for fire safety engineering in buildings.

5.2 Inputs into PD 7974-3

5.2.1 PD 7974-0: Building characteristics and environmental influences

5.2.1.1 General

The definition of the enclosures within the building, the characterization of their key properties and the quantification of key parameters permit determination of the effects of a fire on their integrity and structural form.

5.2.1.2 Enclosure layouts and geometries

The layout and geometry of an enclosure influences the thermal conditions expected during a fire and the mechanical response of the enclosure to such conditions. The dominant variables include W, D, A_f, A_t, A_w, A_T, h_w and w (see Clause **4**).

The identification of ventilation openings and potential routes for fire spread should take into consideration the issues discussed in Clause **8**.

5.2.1.3 Construction details

It is important to understand the constructional form of the enclosure and to identify any load paths that could be present. The details of the structural members within the enclosure might be required, including:

- identification of load-carrying members and tracing of load paths to ground;
- quantification of the dimensions and structural properties of elements of structure;
- identification of support conditions.

The thermal characteristics of the surfaces bounding the enclosure need to be understood in the context of evaluating the expected fire conditions.

The key parameters include λ, ρ and C (see Clause **4**).

Annex A gives some data illustrating the order of these properties for some typical construction materials. These properties change at elevated temperatures and should be used with caution. Ideally, high temperature properties should be used throughout the temperature range. In the absence of this, the properties at 500 °C could provide a reasonable approximation.

Other potentially significant parameters in relation to the high temperature response of materials include moisture content, water of hydration, phase changes and chemical changes that could result in exothermic or endothermic reactions.

5.2.1.4 Structural loads acting

It might be necessary to identify the nature and magnitude of the loads as actions imposed on the structural members within the enclosure. These loads could include self-weight, permanent dead loads and various live loads. Guidance on the combination of loads is given in Clause **12**.

5.2.1.5 Fire load

The quantity of combustible material, both within the enclosure and forming part of the building fabric, influences the conditions that exist during a fire. PD 7974-1 examines how to establish the quantity of fire load present and includes tables that describe typical fire load densities for different occupancies and calorific values of building materials.

5.2.1.6 Characterization of adjacent enclosures, spaces and buildings

To evaluate whether fire spreads from the enclosure of origin to some other space, it is necessary to consider whether the products of the fire as they escape the fire enclosure via openings etc., ignite combustible items in the adjacent enclosures. The designer might need to characterize the layout and geometry of the adjacent enclosure(s), the construction details and the ignitability of the fire load present (if any). If the analysis described in PD 7974-3 indicates that ignition of items occurs in an adjacent enclosure, subsequent determination of fire growth and development in the adjacent enclosure should be undertaken in accordance with PD 7974-1.

5.2.1.7 Environmental influences

The climatic conditions external to the enclosure of fire origin can influence the nature of the flames issuing from openings (see Clause **9**). It might therefore be necessary to take account of the prevailing wind direction and design velocity (m/s). Snow loads or seismic loads might need to be considered when determining the mechanical response of elements of structure.

5.2.2 PD 7974-1: Initiation and development of fire within the enclosure of origin

PD 7974-1 provides information relating to the fire (see Clause **9**) and includes:

- rate of heat release during the post ignition/combustion phase;
- time to flashover;
- temperature-time response of the fire;
- characteristics of the flames, e.g. size: length and thickness, temperature, radiation, emissivity.

Although the period leading up to flashover impacts upon the enclosure, the most important part of the fire is the interval between the onset of flashover and the commencement of decay, known as steady state burning. It is during this period that the capacity of the enclosure to contain fire and the maintenance of structural function is most challenged. The duration of steady state burning is often characterized as the period over which the fire load within the enclosure is reduced from 80% to 30% of its initial value.

5.2.3 PD 7974-2: Spread of smoke and toxic gases within and beyond the enclosure of origin

The smoke temperature determines the rate of heat transfer through fire enclosures that have little or no insulation. In atria for example, entrained air determines the smoke plume characteristics, buoyancy and temperatures on the exposed side of glazing.

PD 7974-2 also provides information on excess pyrolyzates.

5.2.4 PD 7974-4: Detection, activation and control

The time at which smoke control and fire barriers operate influences the temperature rise within the enclosure. Activation of the smoke control removes hot gases and reduces the heat transfer to structural members and enclosures. Reduction in smoke volume maintains integrity of even non-fire-rated enclosures.

The activation of fire barriers, door/shutter release and damper systems contain the fire, potentially increasing enclosure temperatures. Activation of shutters separating compartments or buildings reduces heat transfer by radiation and fire spread.

The activation of suppression systems limits the size of the fire and potential enclosure temperatures and reduces fire spread to adjacent buildings, either from flying brands or radiation. Suppression systems keep the temperature of structural components below their critical temperature, thereby preventing significant distortion or collapse. Enclosures that are not fire resistant in terms of standard furnace testing might withstand a moderate rise in temperature without loss of "notional" stability or integrity or heat transfer to the adjacent enclosure.

5.2.5 PD 7974-5: Fire service intervention

The time at which the fire service intervenes affects the structural behaviour and fire spread beyond both compartments and non-fire-rated enclosures. The type of building, e.g. single or multi storey, and the occupancy profile or risk profile influences fire-fighting operations and the need for containment.

5.2.6 PD 7974-6: Human factors – life safety

The fire safety strategy for the evacuation of the occupants determines the fire resistance requirements for the structure and compartment. Evacuation can be phased or simultaneous, with the former requiring a greater assurance on the stability and integrity of the structure.

In some types of occupancy, such as residential buildings, the normal approach to fire safety is to evacuate only the occupants directly affected by the fire. The remainder stay within their apartment, relying upon adequate compartmentalization to maintain their safety. In other cases, where the occupants could have impaired mobility and either require assistance or cannot be evacuated, refuges might be required in which a greater degree of compartmentalization would be necessary.

5.2.7 PD 7974-7: Probabilistic risk assessment

PD 7974-7 should be reviewed during the design process and provides deterministic calculations. Input data and analytical methods in PD 7974-7 should be considered in terms of their reliability, confidence in their representation (characteristic design values) and their limits of applicability.

5.3 Outputs from PD 7974-3

5.3.1 Building characteristics

Analysis undertaken in accordance with this Published Document might conclude that changes to the enclosure boundaries, e.g. failure of surfaces, opening of gaps or alteration to the structural form, such as deflection or collapse of loadbearing elements of structure, could occur. These changes in the characteristics of the building need evaluating in their own right as part of the qualitative design review (QDR) process. The variable boundary conditions should also be considered when analysing adjacent enclosures within the building.

5.3.2 Time to fire spread from enclosure

The QDR establishes what constitutes spread of fire from the enclosure of origin. The analysis described in this Published Document determines the time at which ignition of a fire occurs in a space or enclosure outside the enclosure of origin. In addition, guidance is given on how to determine the conditions generated outside the enclosure by a fire inside the enclosure, e.g. the extent of heat radiated from openings. These conditions (and their changes over time) might need evaluating in their own right as part of the QDR where they could affect other fire safety issues, e.g. the movement of persons within the building.

5.3.3 Time to structural failure

The QDR establishes what constitutes failure of the structure. Depending on the overall objectives, failure can range from collapse of a major structural element to an excessive deflection of a secondary member. In general, where the overall design objectives relate solely to saving the lives of the building's occupants, a larger degree of structural movement can be accommodated than, for example, in designs where the objectives relate to observing a maximum level of damage and financial loss. For example, research by Corus (Tata Steel) at the Swinden Technology Centre and the Building Research Establishment (BRE) [3] has shown that continuous building frameworks are potentially capable of maintaining stability despite the loss of strength of individual beams or columns. The QDR should consider whether local buckling of a column, or the loss in loadbearing capacity of a floor beam with associated deflections of the floor slab, constitutes failure in a building which remains otherwise stable. The designer should be aware of disproportionate collapse issues. The time to structural failure can only be quantified if failure has been adequately defined within the QDR. The same issues arise in concrete structures and other constructional materials where continuity of load paths is attributed to members outside the immediate fire zone.

5.3.4 Risk assessment

The outputs should also be considered in terms of consequences of failure as defined in the QDR, whether it is failure of an enclosure or compartment, fire spread or loss in structural stability.

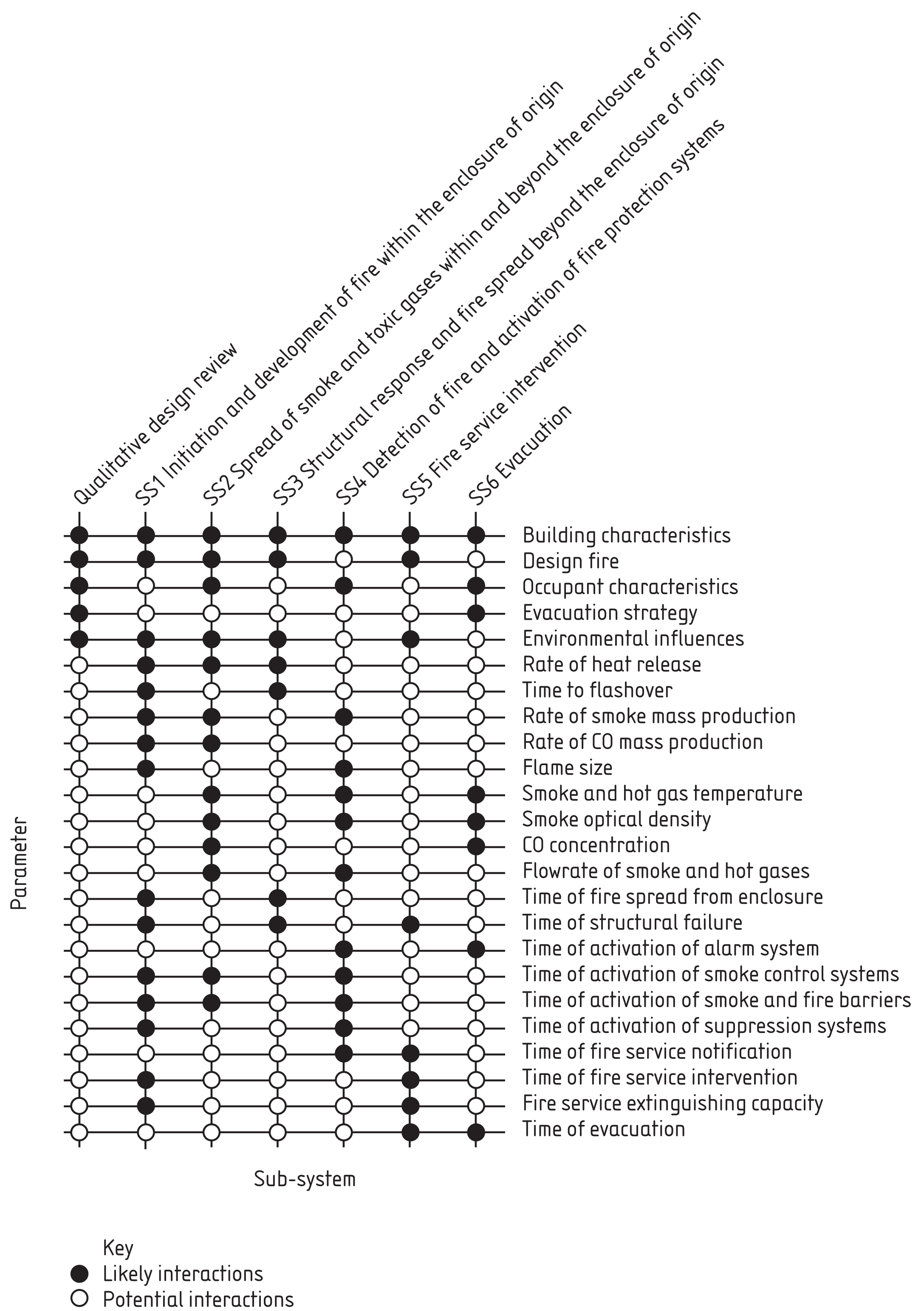
6 Design approach to PD 7974-3

6.1 General

A framework for the application of engineering approaches to fire safety in buildings is provided in BS 7974.

PD 7974-0 assists with the design process. The quantitative design analysis is divided into a number of separate sub-systems. Each sub-system can be used in isolation when analysing a particular aspect of design, or can be used in combination as part of an overall fire evaluation of a building. Figure 3 illustrates potential interactions between the sub-systems. The parameters are often inputs into one sub-system and outputs from another.

Figure 3 **Methodology for interaction of all sub-systems in the PD 7974 series**



Each of the sub-systems concentrates on the quantified analysis state of the BS 7974 framework, however additional guidance (specific to the relevant sub-system) is useful when considering the overall framework. This subclause provides guidance on the interaction between PD 7974-3 and the overall BS 7974 framework, and defines the process for quantitative analysis of the structural response and the likelihood of fire spread beyond the room of fire origin.

6.2 Interaction with BS 7974 framework

The BS 7974 framework is shown in Figure 1. The preliminary part of the framework is to conduct a QDR comprising several components, defined in BS 7974, during which the relevant sub-systems should be considered. Clause **6** provides guidance on how PD7974-3 interacts with the various components of the QDR and some of the issues to be considered within the QDR.

6.3 Personnel involved in the QDR

Input from all critical members of the design team is required for a successful PD 7974-3 analysis. The relevant parties and their level of involvement depends on their role in achieving the objective(s) of the analysis. To determine the relevant parties, it is worth considering the development of a building from concept to occupation. Figure 4 is a schematic representation of the circle of fire safety during the design and construction of a building. The key and the following list show the steps that are involved in delivering a successful fire strategy and use the example of fire load assumptions to demonstrate the process.

a) During the design phase, certain assumptions are made about the building occupancy, potential risks and how the building is managed and operated, e.g. a fire load is assumed for certain areas of the building. In some areas, sprinklers might be required to ensure fire sizes are controlled.

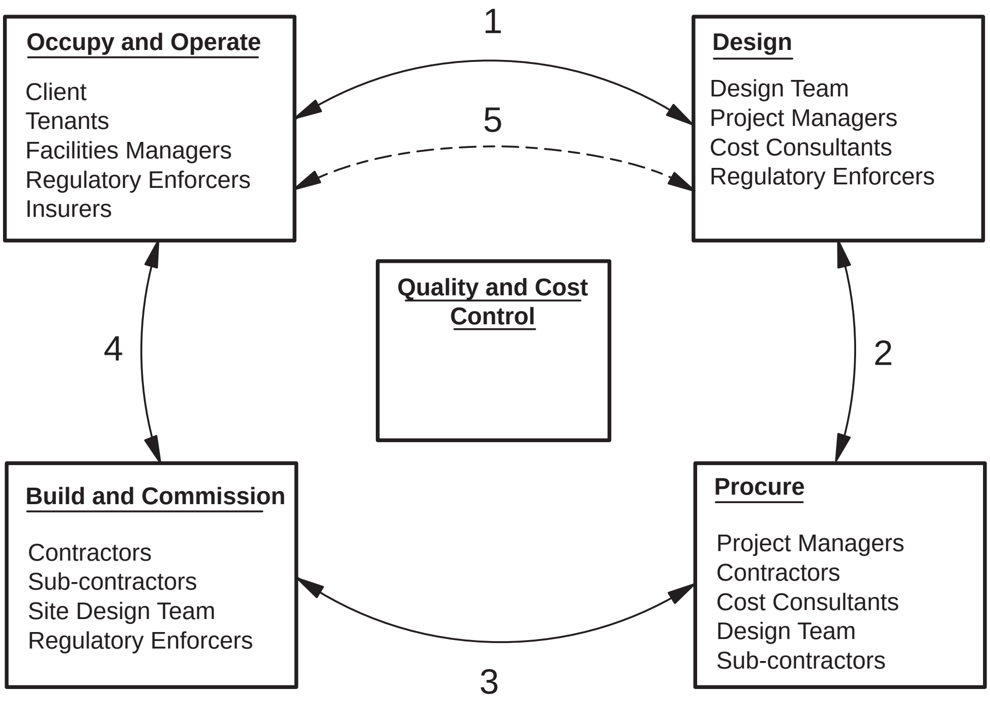
b) The design team use the tender documentation, which is likely to include the fire strategy and appropriate performance specifications to ensure that that the fire strategy is translated successfully. The tender specification should outline the areas to be covered by the sprinklers and the performance requirements of the sprinklers.

c) If there are onsite changes, these should not impact on the fire strategy and, if they do, the fire strategy should be altered to accommodate the changes. For example, the correct sprinkler systems should be installed in the correct areas.

d) The operations and maintenance strategy should ensure that fire loads are controlled where necessary and an appropriate maintenance procedure should be in place for the sprinkler system.

e) If changes are made, these should not compromise the fire strategy. The fire strategy should be revised to ensure that the objectives are achieved for the as-built and occupied condition. For example, if the occupancy is more hazardous than originally envisaged and the desired objective cannot be achieved, remedial measures should be put in place.

Figure 4 **Interaction between the various professionals as part of the process in delivering a successful fire strategy**



Key

1 The design team is responsible for developing a fire strategy that achieves the performance objectives for the building in its occupied state [see list item a)].

2 The design team has to ensure that their fire strategy is translated successfully during the building's procurement [see list item b)].

3 During the construction phase, it is important to ensure that what was designed is being built [see list item c)].

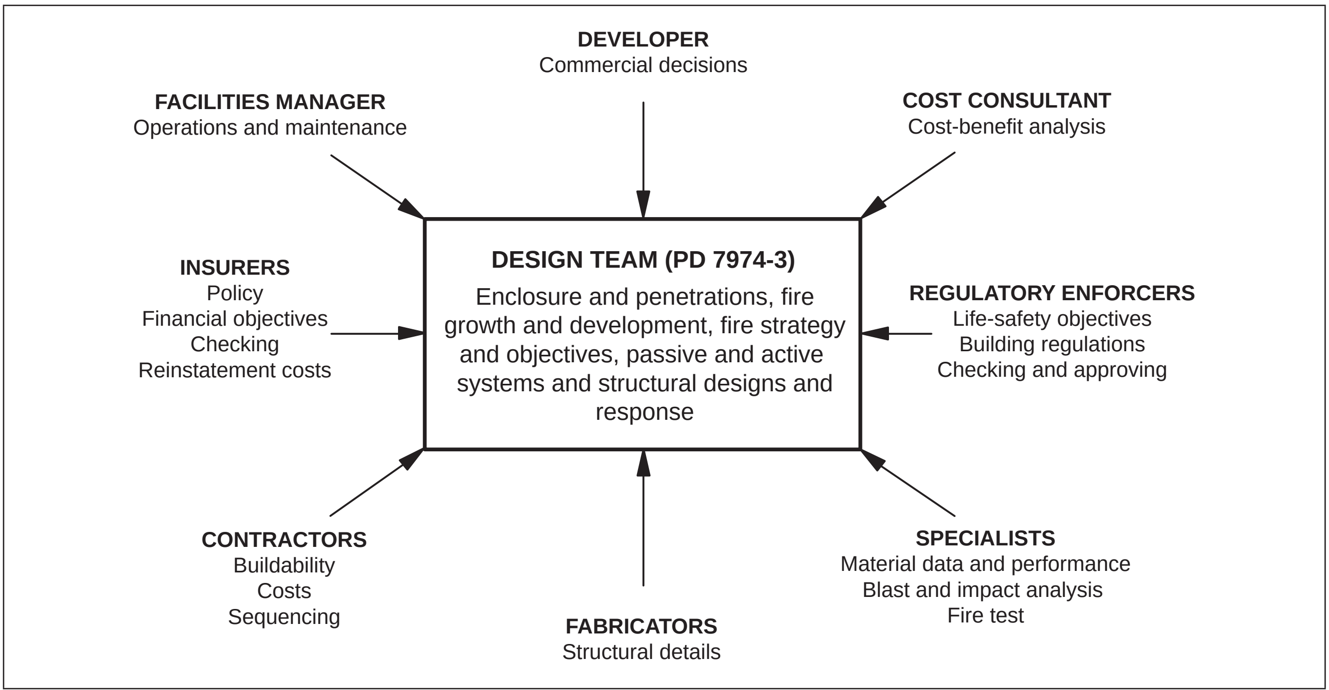
4 An operations and maintenance strategy is developed for the building [see list item d)].

5 Any assumptions made during the design phase are checked for accuracy [see list item e)].

Throughout the process, quality and cost should be managed and controlled.

Having looked at the key components of the building process, it is possible to identify the parties that might be involved in PD 7974-3, listed in Figure 5. Not all parties need to be involved, and those who are do not have to be involved throughout the whole process. This is not an exhaustive list and the person responsible for conducting the PD 7974-3 study should ensure all relevant parties are involved at the appropriate time. The personnel involved in the QDR can vary throughout the process.

Figure 5 **Interaction between the various professions and the design team in addressing PD 7974-3 factors**



6.4 Experience and qualifications

The complexity of the interactions between people, buildings and fire varies such that no single set of calculations can be applied to all buildings in all circumstances. Therefore, fire safety engineering (FSE) requires a greater degree of care and responsibility by the designer than the application of prescriptive codes. The application of FSE should be entrusted to suitably qualified (in the appropriate field) and experienced personnel. Engineering Council-recognized corporate engineers from a fire or related construction-based discipline can be considered suitably qualified.

6.5 Timing of QDR

The QDR should be carried out early in the design process so that any substantial findings can be incorporated into the design of the building before the architectural drawings are developed. However, in practice, the QDR is likely to involve some repetition as the design process moves from broad concept to detailed design.

6.6 Establish fire performance objectives

The fire safety objectives can be applied to the fire strategy as a whole (for example the structure should maintain its stability for a sufficient period to allow the safe evacuation of all occupants), or it can be more specific (for example the design of a compartment wall, including any penetrations or openings, should conform to BS 476-20).

The fire safety objectives should take account of one or more of the following: life safety [see a)], property protection and business continuity [see b)] or environmental protection [see c)].

a) Life safety objectives typically consider overall building stability, maintaining the tenability of escape routes, fire containment, prevention of external fire spread, and/or protection of fire fighters.

b) Property protection and business continuity objectives typically consider damage limitation, fire and smoke containment, and/or reinstatement costs and time.

c) Environmental objectives typically consider fire, smoke and run-off water containment.

6.7 Identification of fire hazards and possible consequences

6.7.1 General

In the context of PD 7974-3, the primary hazards and possible consequences derive from structural failure and/or fire spread beyond the enclosure of fire origin. Therefore, as part of the QDR, it is necessary to understand the mechanisms of fire spread (see Clause **7**) and potential routes for fire spread (see **6.7.4**).

6.7.2 Mechanisms for fire spread

6.7.2.1 General

Assuming that fire starts within an enclosure, it could potentially spread to adjacent enclosures or spaces as the individual or combined result of heat transmitted by:

a) conduction;

b) convection;

c) radiation;

d) mass transfer;

e) direct pyrolysis.

6.7.2.2 Conduction

The solid boundaries of an enclosure have one surface exposed to fire conditions whilst the other, non-exposed surface faces into the adjacent enclosure/space. An excessive flow of heat from the exposed to the non-exposed surfaces of the boundary elements can lead to the fire spreading to adjacent spaces, known as insulation failure of the enclosure. Heat can be transmitted by direct conduction to the non-exposed side of boundary elements or, by indirect conduction, through building components that are continuous on the outside of the enclosure, e.g. pipes, ducts, beams and columns.

Whether the heat conducted to the non-exposed surface causes the spread of fire depends upon the effect this heat has on adjacent spaces. The heat conducted to the non-exposed surface from the fire enclosure can precipitate fire spread through:

- ignition of the non-exposed surface; or
- conduction of heat from a non-exposed surface to combustibles with which it has direct contact; or
- convection or radiation of heat from the non-exposed surface to adjacent combustibles.

It is possible to inhibit this fire spread mechanism through prevention of these scenarios. However, the conductive heating of the non-exposed surface might need to be considered separately in terms of the effect on building occupants.

6.7.2.3 Convection

The flow of hot gases or flames through openings, whether fixed or a result of integrity failure in the enclosure, can cause ignition of combustible items in adjacent spaces. In addition, collapse of the boundary element, due to, e.g. its failure to remain sufficiently loadbearing under fire conditions, can also permit transmission of fire through excessive convection. Heat flow through openings is difficult to quantify, particularly in the stage between initial integrity failure and total collapse.

6.7.2.4 Radiation

The transmission of heat from openings in the enclosure can cause ignition of adjacent combustible items. Heat can be radiated from fixed openings (e.g. doors and windows) or openings which have occurred as a result of fire.

6.7.2.5 Mass transfer

Burning fuel items within the enclosure can be transferred to other enclosures through fixed or fire created openings as a result of integrity failure. Examples include the projection of flying brands and the flowing of liquid pool fires under doors with no bund protection.

6.7.2.6 Direct pyrolysis and reaction to fire

Where boundary elements are combustible and continuous outside the enclosure, it is possible that pyrolysis can extend beyond the enclosure, for example with lateral fire spread within the thickness of combustible walls and roofs. Successful fire stopping of pyrolysis routes is influenced by the characteristics of the flammable materials present and the mechanical stability of the overall system. For example, continuous members extending beyond the enclosure could permit fire spread by pyrolysis via a continuous combustible component. Fire stopping can be impaired by local collapse or deformation of the non-combustible part of the system. The collapse of enclosure boundaries can also permit fire to spread by direct pyrolysis.

6.7.3 Factors influencing fire spread

The likelihood of fire spreading beyond its enclosure can be influenced by:

- the fire resistance of the boundaries on the enclosure, e.g. boundaries designed to be fire-resisting (in accordance with

BS 476-20, BS 476-21 and BS 476-22) can successfully resist the passage of a fully developed fire for a known minimum period;

- the anticipated fire severity in the enclosure, determined by the amount of fire load and ventilation present;
- measures to reduce the severity of a fire by reducing its capacity to penetrate enclosure boundaries, e.g. the installation of an automatic sprinkler system (direct) or by limiting the ventilation available to the fire or limiting the amount of combustibles available to fuel it (indirect);
- the size of the enclosure, e.g. fires in atria or single storey premises with high roofs are more likely to remain fuel controlled and less likely to reach flashover;
- access to open vertical shafts such as stairways, lift shafts or service ducts which can increase fire severity through introduction of ventilation and through draughts, and vertical routes which can permit fire spread in the absence of appropriate fire dampers;
- the presence of concealed spaces (e.g. above false ceilings, within hollow construction and under floors), which can increase the potential for fire to spread undetected;
- the air pressure conditions within the enclosure and pressure differentials between enclosures, which can reduce fire spread by the release of heat through ventilation. Fire and hot gases are less likely to spread into adjacent enclosures if they are maintained at a higher pressure than the fire enclosure. This principle of positive pressurization is used to protect stair shafts;
- the extent of openings within the enclosure boundaries, e.g. loss of air-tightness through poor workmanship in construction, unstopped joints and service penetrations can provide easy routes for fire spread;
- the deformation of structural elements can open gaps in an enclosure's boundaries in a gradual or sudden manner, e.g. through application of load to non-loadbearing assemblies;
- voids behind curtain walling systems, e.g. systems incorporating glazing, aluminium and steel-faced composite panels can distort or otherwise react to fire exposure in a manner that allows fire to by-pass the fire stopping.

6.7.4 Routes for fire spread

Once the enclosure has been characterized, the designer should identify all possible routes of fire transmission through the boundary surfaces. Figure 6 illustrates some of the more common direct routes for potential fire spread. In many instances, the designer should also consider the potential for fire spread between two adjoining enclosures via independent spaces. These routes of fire spread should be examined as a series of direct spread mechanisms.

All potential routes for fire spread from the enclosure should be investigated and the minimum time for fire spread, as necessary for the QDR, determined. However, design effort can be reduced in situations where expert judgement can identify those routes susceptible to the most rapid fire spread. The determination of whether fire spread takes place or not is influenced by conditions both within the fire enclosure and within the adjacent enclosures/spaces.

Figure 6 **Routes for fire transmission**

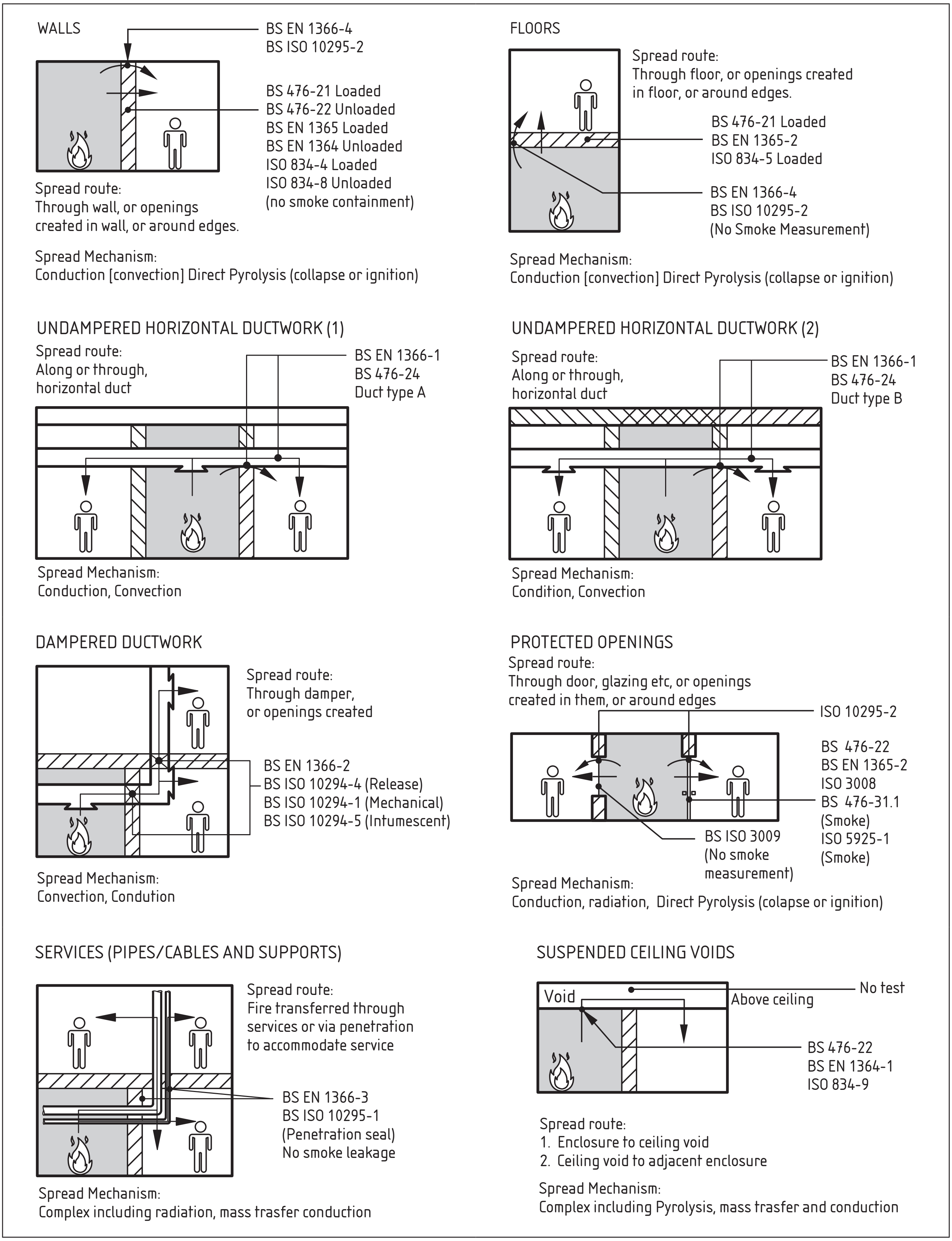
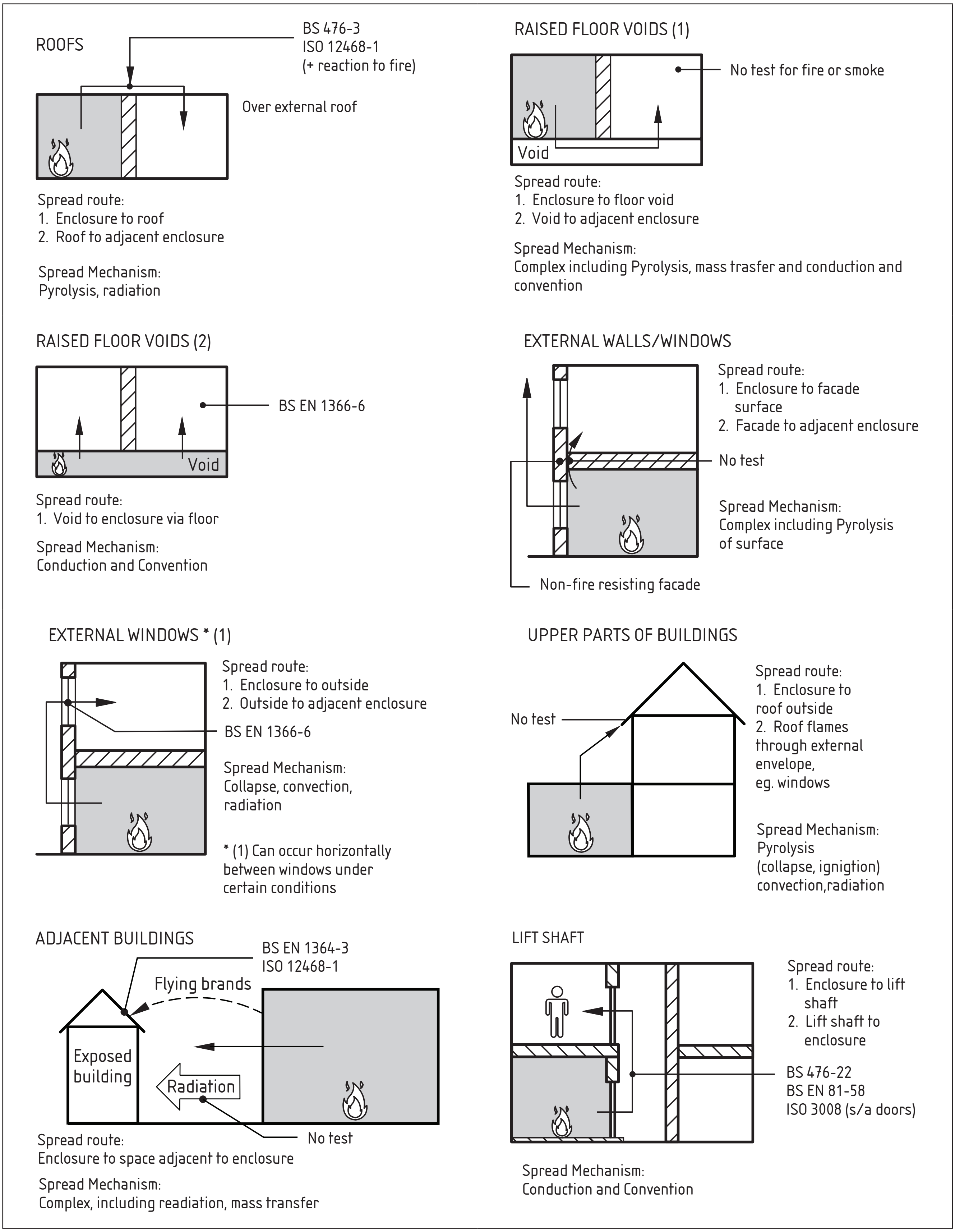


Figure 6 **Routes for fire transmission** *(continued)*



6.8 Establishing trial fire safety designs

6.8.1 General

Having identified and evaluated the potential hazards and their consequences, the designer should mitigate the hazards, and/or demonstrate that the consequences of the hazards are acceptable. The trial design is the designer's initial attempt at developing a solution likely to meet the desired fire safety objective, which is then tested through quantitative analysis. Typically, for PD 7974-3, the trial design should consider:

- fire resistance of the structure including any applied fire protection;
- extent, frequency and performance of fire barriers;
- influence of any suppression measures;
- effectiveness of, or need for heat and smoke extract;
- fire service intervention and effectiveness.

The trial design should also consider the level of redundancy, robustness and reliability that is acceptable, and specifications should be produced to ensure these.

6.8.2 Redundancy

Redundancy or diversity ensures that an alternative is available if a particular feature or system becomes compromised. For example, providing alternative structural load paths or not taking into account the beneficial effects of sprinklers in the analysis constitutes a level of redundancy.

6.8.3 Robustness

The robustness of the system is its ability to perform its function, even if the exposure conditions are not exactly as predicted and the condition of the element at the time of the event is worse than anticipated. Robustness is more important where redundancy is not possible.

6.8.4 Reliability

Reliable systems are those likely to perform as required. Reliability can be improved through the use of reliable components, where their quality and installation are accredited by a third party and these are subjected to appropriate maintenance and testing.

6.9 Identification of acceptance criteria and appropriate methods of analysis

6.9.1 General

The acceptance criteria, as defined in the QDR, should be applicable to the fire safety objective (see **6.6**) and can be deterministic or risk-based (where risk is the potential consequence of an undesirable event). Typically, the acceptance criteria are expressed in terms of loadbearing capacity, integrity and/or insulation. The acceptance criteria can use predetermined levels of acceptance, such as the criteria specified in BS EN 1363-1 and BS EN 1363-2, or they can be derived to mitigate the identified hazard.

6.9.2 Factors influencing life safety acceptance criteria

The life safety of people in or around the building, including fire-fighters and search and rescue, is compromised if the structure collapses with people inside or within range of the building. Similarly, life safety is threatened if the tenability of the space people are in, or need to pass through to exit safely, is outside the limits they can withstand, taking into account any personal protection they might have, such as breathing apparatus.

Factors to consider when assessing the ability of the building to withstand collapse for the requisite period are usually derived from the required safe egress time analysis (see PD 7974-7) and include:

- loss of loadbearing capacity of structural members;
- rate of deflection; and
- the maximum permissible stress or stain.

The following factors need to be considered when analysing the security of relevant spaces:

- integrity and smoke tightness of the elements forming the structure in respect of the leakage of fire and gaseous combustion products into protected enclosures;

  NOTE 1 This is usually the product of collapse of boundary elements, deflection or distortion, propagation of cracks and fissures, or the burning through or melting out of component parts.

- insulation and radiation of the elements forming the structure in respect of ambient temperature within the exit routes and protected enclosures.

  NOTE 2 This is usually the product of tightness but, prior to any loss of smoke tightness and/or integrity, it is influenced by the conducted/convective and radiant heat emitted by the fire separating elements of structure.

The standard criteria used in the fire resistance tests, as appropriate to the element being tested, might have little or no direct relationship to the critical tenability levels. However, there is probably a fairly high level of redundancy in the solution derived if they are used.

NOTE 3 A more detailed consideration of the applicability of the conventional criteria can be found in ISO/TR 22898.

6.9.3 Factors influencing property protection and business continuity acceptance criteria

The criteria for property protection and business continuity are likely to differ from those for life safety. Instead of calculating a time to failure, it is more likely that PD 7974-3 would be used to assess the level of potential damage that occurs for different fire scenarios and determine whether this is acceptable in terms of:

- value of capital losses;
- loss of income; and
- extent of damage and reinstatement costs.

The method of quantifying the acceptability of the enclosure’s fire behaviour with respect to the property protection and business continuity is not dissimilar to methods for life safety (see **6.9.2**), except the critical conditions relate to the ability of the structure, equipment, processes and products to withstand the conditions predicted to prevail in the spaces during fire.

If the analysis shows that the enclosure, equipment, processes and products cannot withstand the conditions predicted by the analysis, the conditions should be mitigated by the provision of enhanced integrity and insulation levels from the separating elements.

6.9.4 Factors influencing environmental acceptance criteria

The impact of fires on the environment can be considerable, particularly in buildings that house industrial or chemical processes or storage. In such situations, preventing fire spread can significantly reduce the risk of contamination and the following acceptance criteria are usually considered:

- the ability of the structure to contain the products of combustion and load and prevent fire spread into environmentally sensitive areas;

  NOTE The performance levels used can differ from those used for life safety, or even property protection, particularly the insulation criteria which reflect the nature of the contents of the risk area.

- the maximum allowable quantity of combustion gases released;
- the area of damage (direct and indirect);
- energy released by fire;
- energy and resources consumed in controlling fire.

6.10 Establish fire scenarios for analysis

BS 7974 provides considerable advice on the selection of fire scenarios. With PD 7974-3, the following should be considered.

- In order for fire or smoke to spread beyond the enclosure of origin it is often assumed that a post-flashover fire occurs, which is not necessarily true.
- When assessing the performance of structural elements it is often assumed that a fully developed compartment fire represents the worst case. This is usually true but a localized fire sometimes represents the worst case, for example in structures that are susceptible to restrained thermal expansion.
- It is often assumed that all elements being heated simultaneously represents the worst case, which is not necessarily true.
- Failure can often occur in the decay phase of a fire so it might be necessary to include decay in the analysis.
- The behaviour of real fires is very sensitive to the amount of ventilation available. Well-vented fires tend to be short-lived, the converse being true for under-ventilated fires. The performance of structural and separating elements is sensitive to the combustion temperature and duration of fire exposure and the hottest or the longest fires do not necessarily represent the worst case. Therefore, the number and range of design fires should be carefully considered to ensure a reasonable range of conditions is evaluated.

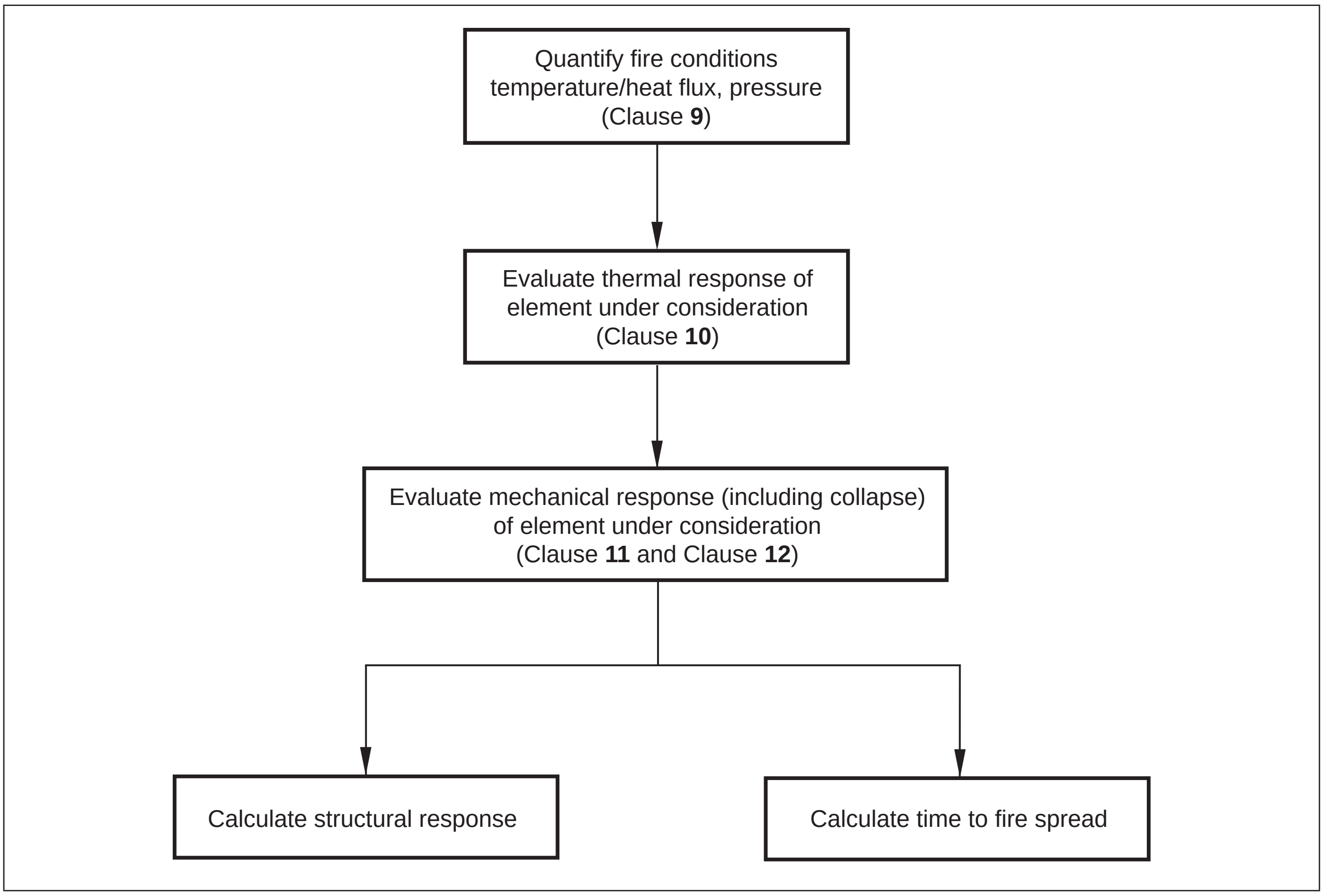
6.11 Quantitative analysis

6.11.1 General

The primary focus of each of the BS 7974 sub-systems is the quantitative analysis, and the results of the analysis should be supplied by the sub-systems as output to the assessment stage of the BS 7974 framework. The process should be completed for each of the fire scenarios and each of the potential routes of fire spread identified within the QDR.

The process for quantitative analysis that should be conducted as part of PD 7974-3 is defined in Figure 7. Each step can be conducted sequentially or combinations of the steps can be conducted simultaneously.

Figure 7 **Procedure for quantitative analysis within PD 7974-3**



6.11.2 Defining analysis methods

The science behind each of the parts of the quantified analysis of PD 7974-3 is complex and there are several methods. These include prescriptive guidance, testing, empirical relations, expert judgement and varying complexities of quantified analysis. Each method has its own associated accuracy and range of applicability. It is up to the designer to select the appropriate method for the problem in hand, and this is a function of the objectives, the potential hazards and consequences, the required accuracy and the required output.

6.11.3 Defining the fire enclosure and its openings

6.11.3.1 General

The fire conditions are directly influenced by the size, geometry and construction of the enclosure of origin. Therefore, it is important to accurately define the enclosure prior to conducting the analysis and to redefine it during the fire exposure, as the enclosure boundaries can change as a result of the changing conditions. Guidance on the characterization of the boundary is given in Clause **8**.

6.11.3.2 Characterizing the boundary of the fire enclosure

All nominally impermeable boundaries in a building inhibit the spread of fire. Certain boundaries are purpose-designed to prevent fire spread and are generally designed against codes of practice which incorporate elevated temperature of fire case data. When these do not exist, the boundaries are assessed by fire engineers with the relevant knowledge of the high temperature response of various forms of structure and/or materials, or are tested in a standard furnace to assure their fire-resisting performance. The fire safety strategy established in the QDR designates those boundaries required to resist fire spread and the duration for which they are required to do so. Even non-fire-resisting elements forming an enclosure influence the rate of growth of the fire. The ongoing capability of these initial enclosing surfaces to remain impermeable to the spread of fire is a function of their construction and the fire condition, which can be subject to change. Accordingly, the nature of the fire enclosure should be viewed as time-dependent, requiring ongoing analysis.

6.11.3.3 Characterization of fixed openings (doors, windows, vents etc.)

The fire conditions within an enclosure are influenced by the size, shape and extent of openings which permit airflow into the fire and the outflow of heat and combustion products from the enclosure. Where a combination of fixed opening conditions is a possibility for the fire enclosure, e.g. some doors open, some doors closed, options which are most conducive to fire spread (a worst case scenario) should be considered. However, the use of worst case scenarios might not be valid if good management of openings forms part of the strategy. In the case of doors, the use of closing devices without damping, or with high closing forces, can lead to the doors being wedged open. Such a practice lessens the effectiveness of the enclosure, both for containing fire and restricting oxygen availability. Abuse of the door can be prevented by the use of:

- electromagnetic hold-open devices;
- delayed closing face fixed overhead closers; and
- swing free closers.

The use of compatible hardware, such as hinges and latches increases the probability of the door being closed.

Keeping doors closed during a fire is also dependent upon a number of features, such as the type and dimensions of door edge sealing systems, and their ability to remain closed against normal fire pressure fluctuations and differential distortions between leaf and frame.

Similarly, dampers in air transfer systems that are electrically operated by an actuator linked to the detection system are more reliable than dampers that are operated by a thermal link. Penetration of the enclosure boundary by services (plastic and metal pipes and cables) is less likely to cause the enclosure to lose integrity if the sealing system can compensate for the erosion of the associated construction.

Glasses used for glazed openings are more reliable if they maintain integrity when subjected to changing temperatures.

The specification and design of sealed openings is a much neglected aspect of fire containment and expert guidance should be sought if the enclosure boundaries are critical to the performance. Any modelling of the enclosure fire dynamics, especially Computational Fire Dynamics (CFD), requires an understanding of the reliability of the enclosure to make accurate predictions. Control over the boundary is therefore critical, and it is only by attention to such details that reliability can be achieved.

6.11.4 Defining fire conditions

To predict the structural behaviour of individual elements of the enclosure, or quantify the integrity, smoke tightness and insulation characteristics of the fire separating elements of the enclosure, it is critical to have detailed information on the fire conditions to which they are exposed. The fire conditions change the longer the fire continues. As the fire grows, the amount of fuel involved increases though a shortfall in the air (oxygen) supply and the intervention of any automatic suppression system can delay this trend. The thermal response of the boundary and the structural elements can only be predicted with accuracy when the fire conditions, with respect to time, have been accurately quantified (see Clause **9**).

It is not only the temperature within the enclosure that is important. Any pressure differential between the inside of the enclosure and the adjacent areas has a significant influence on the rate at which the tenability of the protected spaces is lost. In any natural enclosure fire there is generally a gas pressure generated above a certain height, normally referred to as the neutral axis. Above the neutral axis there is a stimulus for convection (see **6.7.2.3**), whereas below the neutral axis the stimulus is very low or non-existent. In some fire strategies depressurization is used to reduce the spread of fire through openings where the provision of a robust seal is difficult. The standard fire resistance test incorporates a neutral axis and a positive pressure differential, but this is not always the case in practice, making the application of the results of standard fire tests problematic, for example with the risk of fire breaking into a building. Prevailing wind conditions could require greater provision in some situations.

When defining the fire conditions, it is therefore important to consider both the temperature and pressure conditions.

Whilst unquantifiable, turbulence in the fire could make the results of some fire tests inappropriate if the construction materials are not robust enough to withstand this.

6.11.5 Heat transfer and thermal response

Heat is transferred from the fire into elements of the structure by means of conduction, convection and radiation. Heat transfer assessments determine the amount of heat being transferred into the structure via its surfaces.

Having established the amount of heat that is transferred into the structure via its surfaces, the next stage is to determine how the heat is transferred throughout the element. The heat transfer and thermal response calculations are often conducted simultaneously (see Clause **10**).

6.11.6 Structural response

6.11.6.1 General

As an enclosure or separating element changes temperature, its structural response changes as a result of corresponding changes in material properties. The purpose of the quantitative analysis is to determine structural response as a result of the thermal response. The purpose can vary depending on the reason for conducting the analyses and can reflect life safety, property protection, business continuity and/or environmental protection objectives.

6.11.6.2 Changes in building characteristics

Analysis undertaken in accordance with this Published Document could identify changes to the enclosure boundaries such as failure of surfaces, opening of gaps, or alteration to the structural form, e.g. deflection or collapse of loadbearing elements of structure. These changes in the characteristics of the building should be evaluated in their own right as part of the QDR process as they can compromise the fire strategy objectives. The changes might also need to be considered as variable boundary conditions when analysing adjacent enclosures within the building.

6.11.6.3 Time to fire spread from enclosure

The time to fire spread from the enclosure of origin is discussed in **5.3.2**.

6.11.6.4 Time to structural failure

The time to structural failure is discussed in **5.3.3**.

7 Analysis of mechanisms of fire spread

7.1 Forms of analysis

The potential for spread of a fire from the enclosure of origin is influenced by the thermal and mechanical response of the enclosure's boundaries (walls, roof, doors and windows). The thermal and mechanical response of boundary elements can be evaluated individually, subject to checking for interaction effects between adjacent elements. For example, the thermal bowing of walls could affect the support or loading conditions on the roof enclosure.

The behaviour of any individual element can be determined by:

a) standard fire resistance tests;

b) experimental large and small-scale fire tests;

c) expert assessment; and

d) quantitative analysis of fire spread mechanisms.

Certain mechanisms are best analysed through practical tests, whilst others can be analysed using advanced modelling techniques. It is important, when using engineering calculations, to recognize the level of sophistication being employed. Table 1 summarizes the available forms of analysis. Maintaining a consistency of sophistication when evaluating the thermal and mechanical response increases the design efficiency.

Table 1 indicates the possible combinations of fire and structural analysis. It is possible that alternative combinations are justifiable. Equally, some of the combinations might not be valid and the designer should justify his decision.

NOTE For potential outputs that can be obtained from various analysis methods, see Figure 8.

Table 1 **Overview of means of analysis for each fire spread mechanism**

Mechanism	Standard test	Experimental tests	Expert assessment	Engineering calculations
Conduction	BS 476-20 (Insulation criteria) (See **7.2.5**)	Standard experimental procedures available	Suited to expert assessment from the results of fire tests	Steady state equations describing conduction are easy to use but are overly conservative for all but very lightweight materials. Transient conduction models require software solutions (typically finite element or finite difference) and are well established and verified. The need for temperature-dependent material thermal properties, such as conductivity, poses practical limitations as the necessary data are difficult to obtain. Care should be exercised if applying models to materials likely to undergo phase change or degradation at elevated temperatures. Chemical or physical changes can be associated with exothermic or endothermic reactions.
Convection	BS 476-20 (Insulation criteria) (See **7.2.5**)	Ad hoc fire tests possible	Suited to expert assessment from the results of fire tests	If openings are known, the convection of hot gases can be modelled using various field and zone models. Such models are well established for pre-flashover fires, but their application to post-flashover fires requires more care. The opening of gaps or cracks is difficult to model in solid boundaries. Techniques permit prediction of differential movement in discontinuous boundaries, e.g. doorsets.
Radiation	BS EN 1363-2 (Radiation measurements)	Standard experiments and ad hoc fire tests possible	Generally requires full calculation	If openings are known, a range of well-established and verified radiation models are available, e.g. BS EN 1991-1-2.
Mass transfer	Not applicable	DD ENV 1187 standard test and ad hoc fire tests possible	Suited to expert assessment	Not applicable
Direct pyrolysis and reaction to fire	BS 476-20 (Integrity criteria) (See **7.2.3**) Sustained flaming on non-exposed face	Range of standard ignitability tests available	Suited to expert assessment based on knowledge of combustibility	Not available

Figure 8 **Potential outputs that can be obtained from various analysis methods**

Thermal Exposure Conditions \ Conditions/Complexity of analysis	Standard resistance tests	Semi-Natural test	Judgement or Assessment in lieu of test	Engineering Calculations: Single element	Engineering Calculations: Sub - frame	Engineering Calculations: Entire structure
Standard time/temperature curves	R E I	R E I †	R E I †	R I	R I_{mean} §	R I_{mean} §
Experimental curves	N/A	R E I †	E I	R I_{mean} §	R I_{mean} §	R I_{mean} §
Predicted maximum temperature	N/A	R E I †	I	R I_{mean} §	R I_{mean} §	R I_{mean} §
Predicted temperature curves	N/A	R E I †	E I	R I_{mean} §	R I_{mean} §	R I_{mean} §

Key

R is the loadbearing capacity criterion (as defined in European standards)

E is the integrity criterion (as defined in European standards)

I is the insulation criterion (as defined in European standards)

I_{mean} is the average temperature rise on homogeneous elements

† 1) restraint: walls – 3 edges retrained, 1 edge free; floor – 2 edges simply supported, 2 edges free;

2) complexity of restraint defined by the experiment;

3) both conventional and bespoke criteria can be determined

§ can be restricted to homogeneous materials

Appropriate fire and structural models should be selected for the scenario under consideration. For example, it is not possible to use the standard fire curve to make an accurate prediction of the performance of a whole-frame structure, but it can be used to compare the relative performance of two different whole-frame structural options. When the element is homogeneous, it might also be possible to predict the possibility of integrity failure, but this is not in absolute terms and the designer should analyse the confidence limits within which the calculated value stands.

7.2 Standard fire resistance tests

7.2.1 General

The results from standardized fire tests are used to obtain comparable information on the ability of constructions to resist the spread of a post-flashover fire under an agreed set of conditions.

When using a prescriptive regulatory approach to ensure the resistance to fire spread of a construction, the performance is measured using a standardized fire exposure or fire resistance test in which the three performance criteria designed to meet the objectives are loadbearing capacity, integrity and insulation, with the last of these requiring the measurement of radiation.

The output obtained from standard fire tests is designed to demonstrate the ability of an element to remain stable and resist excessive deflections for a period of time, and the structure to remain impermeable to the passage of flames, smoke and hot gases to the level deemed critical to prevent fire spread for that period. Finally, conductive heat transmission of the construction to the unexposed face should not be greater than the pre-set criteria (see **7.2.5**).

During the standard fire resistance test, a single element of construction of limited size is exposed to a time/temperature regime that is widely, and probably incorrectly, assumed as representative of a fully developed fire. Test methods exist for vertical and horizontal elements of a separating and non-separating nature, with or without applied load. For use in the built environment, the fully developed fire should be characterized by means of a time/temperature condition in accordance with BS 476-20, and there is extensive knowledge of the behaviour of constructions exposed to these specified conditions.

In BS EN 1363-1, the method of measuring the furnace temperature has been changed to a device (plate thermometer) that has a greater thermal inertia than the device (thermocouple) used in BS 476-20. This increases the thermal severity in the fire test furnace so construction elements tested in accordance with BS EN 1363-1 show different results. The designer should be aware of the differences in test regime and their effect on individual building components, as it is not possible to define a single test method that represents the most severe test for all types of constructions. For other fire situations, outside of the normal built environment, a suite of hydrocarbon curves exist, designed to evaluate the resistance of the structure to exposure conditions related to an accident involving hydrocarbon based fuels (see BS EN 1363-2). Whilst these curves are not intended to be used for the built environment, depending upon the fire engineering analysis of the anticipated exposure conditions, they could be appropriate to some fire engineered solutions.

An even more extreme exposure test, the jet-fire test, is specified in BS ISO 22899-1, which evaluates structural members and service penetrations that could be exposed to the fire conditions resulting from a failed valve or ruptured pipeline carrying petroleum-based products. This test was designed to simulate off-shore applications and is not designed to replicate any of the conventional built environment conditions, except possibly where liquid petroleum products are being handled.

7.2.2 Application of results of standard tests

The information produced by standardized tests, such as those in BS 476-20 and BS 476-22, are reasonably well understood by the fire safety industry, including the designer, regulators and approving authorities. In most cases, this permits a rapid appraisal to be made of an element’s ability to resist the spread of fire, and maintain structural function.

However, the use of information from standardized fire tests does not free the designer from the need to understand the relationship between the element tested and the element that exists in the actual building enclosure, or the difference between the expected fire conditions and the test conditions. Where there is a direct similarity between the test specimen and the product used onsite, e.g. beams of equal dimension or doorsets of a tested size and configuration, the only uncertainties regarding performance arise from workmanship. Quality control measures and certification schemes seek to address this risk. Additional consideration should be given in those instances where the test specimen and/or conditions do not directly correspond with realistic fire conditions and the actual building element. This issue frequently arises, as building components tend to be larger or differently configured to those submitted for fire testing.

The maximum specimen size evaluated under standard test conditions is generally 3 m high × 3 m wide, although a 4 m span is used for evaluating horizontal elements. If standard test data are justified in support of the performance of different constructions, an expert analysis should be carried out on the test results to ensure their suitability for extrapolation. BS ISO/TR 12470 (see **11.4**) describes how the extended application can be performed.

The performance of the construction when evaluated by the standardized fire resistance test is adjudged by three predetermined criteria: loadbearing capacity, integrity, and insulation. With loadbearing capacity, collapse is obviously the ultimate form of failure. However, an alternative method of judging loadbearing capacity is by exceeding predetermined deflection limits. These are rather arbitrary and they can be exceeded without necessarily causing fire spread. However, satisfying the fire resistance requirement with respect to loadbearing capacity for a period does not imply that the construction is free from deflection. Significant levels of deflection are permitted by failure criteria, as specified in BS 476-20. If the occurrence of deflection is important to the overall design, e.g. through influencing the performance of corridors forming protected routes on floors below deflected slabs, more restrictive deflection limits might be appropriate.

The use of elements of construction that have only been tested under the exposure conditions specified in BS 476-20, BS EN 1363-1, or ISO 834-1 as part of a fire-engineered building design might represent one of the largest compromises to the design that can be made to an otherwise fire safe building. These tests have been recognized in ISO/TR 22898 as having little value to the fire engineering community and ISO/TR 22898 sets out to identify the areas where the tests and their outputs need to be improved if they are to be of greater value to fire engineers.

Whilst ISO/TR 22898 was written to assist those responsible for revising the tests, or for using them in legislation, it provides information to fire safety engineers explaining the areas where the output of the current tests could compromise the fire performance of the design.

7.2.3 Determining the integrity (E) of elements

The integrity criterion used in the standard fire resistance test is difficult to translate into a fire engineering analysis of fire spread, as there are varying degrees of integrity loss that are measured by other criteria. The alternative methods used involve an oven-dry, cotton fibre pad placed 25 mm away from any gap or crack that is developing or exists in the construction being tested, or the use of two sizes of gap gauges.

The cotton pad methods have been deemed inappropriate when high levels of radiation are being emitted from the surface, i.e. when the insulation characteristics have been lost, as the pads can ignite as a result of the radiation and not the escaping gases. The two gap gauges used in these situations include the 6 mm diameter gauge, which should be capable of being moved more than 150 mm before integrity loss is registered, and the 25 mm diameter bar, which just has to pass through the element. These gauges are rigid and BS 476-20 specifies that they have to pass right through from the unexposed to the exposed face before failure is recorded. It is possible for a 200 mm × 25 mm cross-sectional area gap to exist in a construction, but for the construction to satisfy the integrity criterion as long as there is a convoluted path that prevents the gap gauge from passing right through the element. The risk of fire spread due to a loss of integrity is very different when measured by these three methods, and this should be recognized when using integrity as a performance criterion.

7.2.4 Influence of pressure differentials on integrity

Another aspect of integrity measurement is that the test is performed with a predetermined level of positive over-pressure, relative to the laboratory environment in the upper sections of the furnace. This is invariably where failure would be recognized by means of the cotton pad method. The positive pressure is used to monitor all separating elements and this can sometimes be inappropriate, e.g. when establishing the fire resistance of an external wall from the outside to the inside of a building. The need to resist a positive pressure is one of the most demanding aspects of the fire resistance test, except for largely impermeable constructions such as concrete. If positive pressure is not applied, the duration for which integrity is satisfied can be extended. Positive pressure is also inappropriate for the evaluation of enclosure walls after the roof has been lost. The standard integrity criterion is difficult to apply directly to a fire engineering assessment of fire spread and is often excessively conservative if a purely life safety case is being produced. The proximity of combustible materials, and the sensitivity of equipment being protected, should also feature in the final analysis.

7.2.5 Insulation criterion (I)

The insulation criterion of the standard test consists of a maximum temperature rise of 180 °C anywhere on the surface, and a mean temperature rise of 140 °C. These limits are considered to be safe, although this varies depending upon the fire safety objective as people coming into contact with surfaces at these temperatures risk significant burns. However, if the objective of these limits is to reduce heating of the environment or prevent igniting materials that might be in contact with them, then the traditional temperature levels set are probably lower than the environment can safely tolerate.

7.3 Experimental large and small scale fire tests

Experimental fire tests permit models or replicas of the elements to be exposed to a chosen heating regime within an experimental facility. The experimental facility can be designed on an ad hoc or project-specific basis or might be an existing facility such as a standard fire resistance test furnace. The chosen heating regime might seek to simulate the anticipated fire conditions for the real building enclosure, as predicted using fire models based on a worst case scenario (see Clause **9**). Whilst the designer on a project-specific basis can commission experimental fire tests, the resources needed make it more common for tests to be undertaken by research organizations on a building type basis, or by industry on a product basis. Experimental techniques are available for measuring the thermal properties of materials using standard apparatus. Care should be taken to ensure that the properties obtained reflect the behaviour under transient heating conditions. In some cases, where there is no effect of heating rates or chemical or physical changes, then steady state test data could be appropriate.

Enlarging the scale of the experimental test can minimize the differences between the model and the actual element. The use of larger-scale experimental fire tests means that the thermal and mechanical response of the element under examination can be deduced under boundary conditions that approximate to the real building configuration. For example, in experimental tests where levels of continuity and support can be accurately simulated, the observed behaviour is likely to be superior to that suggested in a smaller scale test, or in standardized tests. Equally, simulating the conditions that would prevail in reality through test replication of the actual fire load and ventilation conditions can be more realistic than standard furnace tests where conservative thermal conditions are assumed.

When setting up an experimental test, critical parameters should be selected, e.g. fire load, ventilation, size of test chamber, construction of elements used to form the chamber, external environmental conditions and moisture content of hygroscopic materials. Such variables can have a significant influence on the outcome of the test and it is important that there is a stated relationship between the parameters chosen within the test and the anticipated conditions in the building that is to be modelled. As specified in ISO/TR 15658, the written report describing the experimental test should describe and justify all relevant parameters.

Scale-modelling is one of the most difficult aspects of any fire simulation. Most semi-natural tests are smaller than the assembly being replicated. This might not be the case for the actual volume of the fire enclosure, but if this space is within a larger structure it is quite likely that the actual constructional elements would be subjected to greater levels of restraint by their connection with elements outside of the fire enclosure. Similarly, they can be of a cross-sectional area, or a loading condition that cannot be properly simulated at the reduced scale. As a consequence, most experimental tests incorporate a compromise for some of the critical structural parameters.

In the case of fire exposure simulations, scale-modelling is usually more difficult than it is in ambient testing because the effect of fire itself cannot be easily scaled. Whilst in mechanical tests it is possible to use a reduced load on a reduced cross-section and achieve a reasonably good prediction of what the capacity would be on a larger section, assuming that the material is homogenous, fire cannot be modelled in such a simplistic way. The thermal inertia of the section is critical, as is the heat transfer rate to the surface of the item being heated. Whilst reduced fire loads can be used, and it might be possible to alter the size or volume of the material being burnt, almost all flames behave in a similar way if there is a flaming reaction. The proportion of radiation with respect to convection varies with temperature and this cannot be altered easily. Consequently, heating as a direct result of flaming combustion almost defies any attempt to reduce its scale. This means that meaningful data can only be generated using structural sections with a similar thermal inertia and heat transfer coefficients to the sections being designed.

Any experimental testing undertaken to generate data for a project-specific application, or to establish the response of a product to non-standardized exposure conditions, should follow the procedures specified in ISO/TR 15658. By following the procedures, the results of the tests should be more comparable with data from other experimental testing, thereby extending their applicability.

7.4 Expert assessment

Data about the fire performance of generic boundary elements, obtained from the results of standard fire resistance tests or codes of practice, can be evaluated to permit assessment of the performance of an element within a real building. The results of fire tests can be extrapolated for practical applications in accordance with BS ISO/TR 12470.

Understanding the effect of the scale-difference between the tested specimen and the real building component is one of the most difficult aspects of any expert assessment. The change in scale can apply to dimensional changes, such as height, width, cross-sectional area or boundary conditions, such as applied load and restraint (see **7.3**).

Expert assessments of the effects of differences between tested elements and real building components, including the effects of scale, should be underpinned by appropriate engineering calculations undertaken by competent persons. The basis of expert assessments should be documented as part of the QDR.

7.5 Quantitative analysis of fire spread mechanisms

7.5.1 General

Each of the component mechanisms of fire spread can be quantified using engineering models and analysis. However, the level of sophistication available and the reliability of the results achieved varies widely amongst the different mechanisms. This reflects up-to-date understanding and the level of analysis typically required for practical design purposes.

7.5.2 Quantitative analysis of heat flow by conduction

The imposition of heat flux to the exposed side of a boundary causes a temperature gradient through the body of the boundary from the exposed surface to the unexposed surface where the boundary material is homogeneous. Fourier’s Law states:

$$q_{\mathrm{cond}} = -KA_{\mathrm{a}}\frac{dT}{dN} \quad (1)$$

where:

$\frac{dT}{dN}$ is the temperature gradient in the direction orthogonal to the area (K/m); and

where – indicates that heat always flows from hotter to cooler surfaces.

Heat ceases to flow between areas of equal temperatures. Under steady state conditions where no internal heat is generated, the Laplace equation can be used, where:

$$\frac{d^2T}{dx^2} \times \frac{d^2T}{dy^2} \times \frac{d^2T}{dz^2} = 0 \qquad (2)$$

where *x, y, z* are the orthogonal directions.

The general equations of conduction can be applied numerically for any particular application using, for example, finite difference or finite element techniques. In some cases exact solutions are available, such as calculating the temperature rise in semi-infinite plane wall exposed to steady heat flux. However, solution of the governing equations for applications in fire safety design is often problematic due to the transient heat flux imposed and the temperature dependency of the thermal diffusivity. The designer should recognize the additional resources required to model transient conduction into enclosure boundaries and, in particular, should avoid use of ambient heat flow techniques and ambient material properties such as U-value methods. A range of proprietary computer software is available to allow solution of transient conduction into solid members [4].

For solid homogeneous materials, it is possible to calculate the unexposed face temperature rise if the conductivity of the material is known, together with the gas temperature of the enclosure and the heat transfer coefficients of the material. The calculation is simpler if the temperature of the exposed face is known as a result of experimental testing against the appropriate exposure conditions.

If the construction is non-homogeneous it is unlikely a model exists that would enable the unexposed face temperature to be calculated with accuracy. In these situations, the unexposed face temperature can only be established by measurements undertaken during a test in order to provide effective values under well-defined heating conditions. This approach is important where changes in physical properties result in endothermic or exothermic reactions. In cases where good contact between components forming a composite can be maintained at elevated temperatures, the thermal behaviour can be established using reduced scale specimens. Where delamination at elevated temperatures is likely, unexposed face temperatures can only be established by exposing large specimens to fire. Standard tests available for establishing these measurements are based upon BS 476-20 or BS EN 1363-1.

The fire resistance test represents only one possible fire scenario and one set of exposure conditions and, in reality, unexposed face temperatures can vary from the conditions measured in the test. The thermal properties could also be influenced by the rate of heating.

Based upon the quantity of heat being conducted through an enclosure boundary, it is possible to quantify the temperature at any location through its thickness, including the temperature on the unexposed surface. The unexposed surface is outside the enclosure of fire origin and its ignition constitutes fire spread.

Predicting the occurrence of ignition is dependent on small-scale experiments that might not be readily scalable to real building assemblies. Fire spread will not occur if the temperature rise on the unexposed face remains below 180 °C at all locations and below 140 °C as a mean. These temperatures represent the failure criteria in BS 476-20 for heat flow to the unexposed face of the test specimen. These failure criteria have long been used to define insulation failure in standard fire resistance tests, and their adoption gives the designer some assurance that a conservative solution has been reached. Persistent ignition of the unexposed face requires a sufficient quantity of heat being conducted through the system at a rate in excess of its surface cooling, such that the temperature rise of a minimum thickness is raised to a level in excess of the material's characteristic pyrolysis temperature. Most organic solids undergo pyrolysis in the range of 275 °C to 375 °C. Polymeric materials are susceptible to decomposition and undergo pyrolysis in the range of 200 °C to 400 °C.

Ignition temperatures given in Table 2 are not an intrinsic property of a material but are affected by orientation of the material and its size, convection patterns, gas mixing and the presence of pilot flames. At best, ignition temperatures should be viewed as an extrinsic material property and the designer should take great care when contemplating ignition temperatures in excess of 200 °C.

Table 2 **Flash-ignition temperatures**

Material	**Flash-ignition temperature** °C
Polyethylene	340
Polypropylene	320
Polystyrene	350
PVC	390
PTFE	560
Polyacrylonitrile	480
Polyurethane (rigid form)	310
Cotton	210

7.5.3 Quantitative analysis of heat flow by convection

The flow of heat from the enclosure of fire origin through fixed openings is described in PD 7974-2. However, many of the methods described are inappropriate to describing heat flow from post-flashover fires or from small and irregularly shaped openings, such those created during a fire.

In general terms, the rate of heat flow from the enclosure of fire origin can be calculated thus:

$$q_{\text{conv}} = MC_{\text{g}}(T_{\text{g}} - T_{\text{o}}) \qquad (3)$$

The mass flow rate of gas from the enclosure is a function of the area of the opening through which flow is taking place. For post-flashover fires, it can be conservatively assumed that M has reached a steady value of:

$$M = 0.5A\sqrt{h_{\text{w}}} \qquad (4)$$

PD 7974-2 offers guidance on how to predict the temperature of hot gases given information on their convective heat content and their mass flow rate. The temperature of the gases exiting through the opening from the enclosure decreases as the distance from the opening increases. Guidance for predicting the temperature of flames issuing from openings is given in **9.4.3**.

The decision as to whether gases introduced at a given temperature into the space adjacent to the fire enclosure would cause ignition is project-specific (see PD 7974-1). A broad range of natural and organic synthetic solids can undergo spontaneous ignition when heated to 500 °C and immersed in a hot gas plume, i.e. when heated by convection only. The presence of a pilot ignition source could precipitate ignition of the solids in the temperature range of 400 °C to 450 °C.

Although not a criterion for fire spread, the designer should note that humans cannot respirate in saturated air environments at temperatures greater than 60 °C. In environments with less than 10% water (by volume) humans cannot tolerate temperatures in excess of 120 °C for more than 7 minutes or 180 °C for more than 1 minute. The development of gas temperatures of this magnitude could constitute an unacceptable spread of the effects of fire rather than ignition and actual fire spread. These issues should be agreed within the QDR prior to establishing the criteria for deciding the propensity for convective fire spread.

7.5.4 Quantitative analysis of heat flow by radiation

A fire within an enclosure radiates heat at a rate which can be calculated by:

$$q_{\mathrm{rad}} = \varepsilon_{\mathrm{g}} \sigma T^4_{\mathrm{g}} A_{\mathrm{rad}} \tag{5}$$

The magnitude of the radiative heat flux on any surface outside the enclosure of fire origin is a function of several variables, including:

- the relative positions of emitting and receiving surfaces;
- the emissivity of the emitting and receiving surfaces;
- the temperature of the receiving surface.

Accordingly, the surface material of the receiver and its location governs the level of radiation exposure outside the fire of origin. Guidance is given in **10.1.3** on how the effects of radiation can be quantified for different materials.

In many cases, it is possible to meet the design objective of analysing radiative fire spread potential using a simplified approach. If the emitting and receiving surfaces are assumed to have emissivity values of unity and the receiving surface does not increase in temperature continuously over time, the radiative flux can be characterized as:

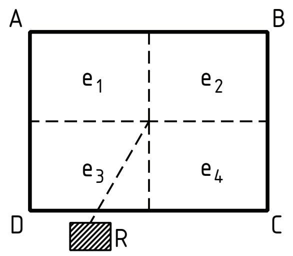
$$q_{\mathrm{rec}} = F_{\mathrm{e\text{-}R}} q_{\mathrm{rad}} \tag{6}$$

where:

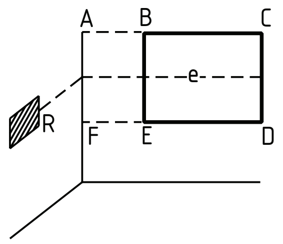
$F_{\mathrm{e\text{-}R}}$ is the sum of all configuration factors.

Figure 9 describes some commonly used configuration factors. The designer should note the relative locations of the emitting and receiving surfaces and recognize the need to combine configuration factors for most practicable situations.

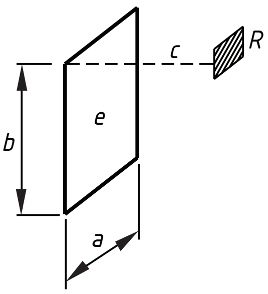
Figure 9 **Configuration factors for typical scenarios**



$$F_{\text{ABCD-R}} = F_{\text{e1-R}} + F_{\text{e3-R}} + F_{\text{e4-R}}$$

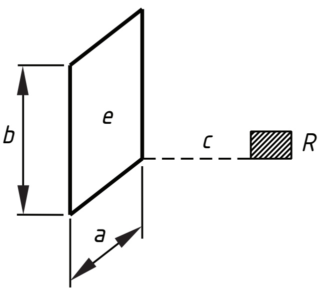


$$F_{\text{e-R}} = F_{\text{ACDF-R}} - F_{\text{ABEF-R}}$$



$$F_{\text{e-R}} = \frac{1}{2\pi}\left\{\frac{X}{\sqrt{1+X^2}}\tan^{-1}\left(\frac{Y}{\sqrt{1+X^2}}\right)+\frac{Y}{\sqrt{1+Y^2}}\tan^{-1}\left(\frac{X}{\sqrt{1+Y^2}}\right)\right\}$$

where $X = \dfrac{a}{c}$ $Y = \dfrac{b}{c}$



$$F_{\text{e-R}} = \frac{1}{2\pi}\left\{\tan^{-1}\left(\frac{1}{Y}\right) - AY\tan^{-1}A\right\}$$

where $X = \dfrac{a}{c}$ $Y = \dfrac{b}{c}$

A range of convenient methods have been developed to allow evaluation of the potential for radiative fire spread between openings from an enclosure and a parallel plane some distance away. The enclosing rectangle method and the notional aggregate area method are described by Read [5]. These methods ensure the maximum radiation received by any point in a plane parallel to the openings does not exceed 12.6 kW/m^2. The methods are typically employed in determining space separation distances between buildings and site boundaries, with the minimum distance recommended as being halfway between the openings and the plane at which the incident radiation is 12.6 kW/m^2.

These methods are also applicable to evaluation of the radiation emitted from the hot unexposed face of an enclosure. This could assist in determining a limiting temperature for the unexposed face of an enclosure. For analysis purposes T_g in equation 5 should be replaced by $T_{surface}$, the temperature of the unexposed surface (in kelvins).

It is difficult to state accurately what level of incident radiation is required to cause ignition of combustible items outside the enclosure of fire origin. Experimental studies have been limited to relatively small-scale specimens. Furthermore, there is the complication as to whether pilot ignition is present in addition to the radiative flux, as might occur as a result of burning brands being emitted from the fire. Lawson and Simms [6] suggest that fibre insulation board ignites under an incident flux of 24 kW/m^2 and that corrugated cardboard spontaneously ignites under a radiative heat flux of 17 kW/m^2. Accepting the need for some factor of safety in design, the widely used threshold of 12.6 kW/m^2 is reasonable.

Determination of more accurate radiation thresholds for ignition might be possible based on an analysis of the increase in surface temperature of the object exposed. Kanuray [7] suggests that natural and synthetic organic solids undergo spontaneous ignition under radiation when surface temperatures reach 600 °C. The presence of pilot ignition, e.g. flying brands, reduces the threshold surface ignition temperatures to between 300 °C and 410 °C.

Whilst specific guidance on the calculation of the thermal response of materials is given in Clause **10**, a useful surface temperature model for ignition hazard analysis is given by:

$$T(x,t) = T_0 + \frac{\varepsilon_{\text{rec}} q_{\text{r}}}{\alpha_{\text{c}}}\left[erfc\left\{\frac{0.5x_{\text{s}}}{\sqrt{Kt}} - \exp\left(\alpha_{\text{c}} x_{\text{s}} + K\alpha_{\text{c}}^2 t\right)\right\} erfc\left(\frac{0.5x_{\text{s}}}{\sqrt{Kt}} + \frac{\alpha_{\text{c}}}{K}\sqrt{Kt}\right)\right] \tag{7}$$

where:

t is the time from start of exposure(s).

Equation 7 assumes that the exposed solid surface can be treated as semi-infinite and is exposed to a radiating hot gas, e.g. a flat ceiling above an opening.

In addition, the potential for a radiative heat flux to cause remote ignition and fire spread from the enclosure could affect persons outside the enclosure of fire origin to an intolerable level. For example, people might not be able to escape past an opening from the fire enclosure because of excessive radiation. This hazard has been recognized in building design, leading to protection of external openings adjacent to stairways and control of glazed openings onto corridors.

The acceptable level of radiative heat flux to humans is largely a function of the duration of cumulative exposure. People can sustain higher levels of radiation for short intervals. Table 3 offers some guidance on acceptable levels of exposure. The tenability issue is considered in PD 7974-5. The necessity of considering the spread of the effects of fire should be determined during the QDR (see PD 7974-0).

Table 3 **Maximum permitted radiation dose to building occupants**

Radiative heat flux kW/m^2	**Duration of exposure** s
< 2.5	> 300
2.5	30
10	4

8 Characterization of the fire enclosure and openings

8.1 Concepts and principles

8.1.1 General

The failure of structural elements and/or the degree of fire spread within a building is directly related to the severity of the fire, in terms of the temperatures reached (and in some instances the rate at which they are achieved), and the magnitude of any pressure differentials that develop over the elements bounding the enclosure of origin. These fire parameters are, however, influenced by the response of the enclosure itself changing as the enclosure perimeter is breached, or by a change in geometry due to collapse or partial collapse of the structural members supporting or forming the enclosure.

8.1.2 Influence of the enclosure on rate of fire growth, temperature reached and time to flashover

One of the significant characteristics of the enclosure that influences the fire conditions is the thermal diffusivity, given by $k/\rho C$, as this dictates the rate at which the thermal energy is stored or dissipated. Forms of construction that have a high thermal inertia (ability to absorb heat), or a high rate of heat loss, result in a slower build up of temperature in the enclosure than when the walls are highly insulating with a low thermal inertia (in the surface layers).

This influence is modest when comparing various forms of masonry/concrete constructions, but the impact could be significant when comparing enclosures constructed primarily from insulating composite panels (even if they do not contribute to the fire load), and single layers of monolithic, non-insulating glass.

Plasterboard-lined stud walls should not be treated the same as masonry or concrete because, whilst the response is initially similar, assuming the concrete is plastered over, the masonry/concrete wall continues to absorb heat after the plaster has become fully calcined. In the case of plasterboard, the cavity void and/or any thermal/acoustic insulation no longer absorbs heat.

The rate of temperature build-up and the temperatures reached are not solely determined by $k/\rho C$, but are influenced by the amount of air, and hence oxygen, that is available to complete the combustion process. If the enclosure is made up of non-designated fire-resisting elements (compartment floors and/or walls) it might be wrong to assume that there is any restriction on the availability of air, as there is normally no legislative requirement to fit devices to ensure that windows and doors are closed in such spaces. It might be permissible, however, to differentiate between spaces that are inhabited or uninhabited when considering the possible locations of fire. In a store room or service duct, where it is normal practice to keep doors closed, it can be assumed that the door will be closed, even though it is not a fire-rated door and the enclosure is likely to be un-fenestrated. Any fire breaking out in these areas grows more slowly than in well-ventilated inhabited rooms. At this stage, any study of the fire behaviour of the enclosure is not concerned with the escape of fire or gaseous products of combustion, purely the availability of oxygen and its influence on the fire severity and growth.

During such an analysis it should be determined whether active smoke control is to take place elsewhere in the building, in which case certain doors might be required to be open in order to provide make-up air, especially if they are not recognized as fire-resisting doors. If this is the case then the cause and effect matrix needs to be consulted to establish the influence this could have on the fire growth.

8.1.3 Defining the enclosure for the pre-flashover calculation

When the enclosure is formed of elements designed to resist fire spread (e.g. compartment walls/floors), it is vital that all openings in these elements are closed at the time of the fire. Usually, all doors are fitted with self-closing devices that close after persons have passed through the opening, during normal operating, non-fire conditions. The fire engineered design should pre-empt the tendency by users to wedge the doors open by ensuring that the doors are user-friendly by means of:

- closing devices that are "damped" and have good control over the closing speed and latching actions;
- electro-magnetic hold-open devices, linked to a fire detection system;
- "swing-free" closing devices that only become active in the event of a fire being detected;
- delayed-action closers, albeit the time for which they hold the door open needs to be set on the basis of the risk;
- other devices linked to a controlled management system that ensures robust procedures are in place to close the door, or that any other device is adequately maintained and checked to ensure that it remains operative.

However, the user-friendliness of a door assembly is not only dependent upon the specification of the closer because other components, hinges, latches, acoustic and smoke seals, can introduce user resistance and should be specified by the designer to reduce any temptation to abuse the self-closing function.

It is not only doors that compromise the fire integrity of an enclosure boundary; boundaries are frequently breached by ducts and services. Mechanical dampers operated by a fusible link are commonly fitted into ducts at positions where they pass through fire-resisting walls and floors. Periods of neglect can render them inoperative due to corrosion and/or the build-up of dirt/detritus. Research has indicated that, due to laminar flow, fusible links can suffer some delay before operating. These problems can be addressed at the design stage by specifying:

- motorized dampers linked to the fire detection system;
- intumescent dampers which operate once the temperature reached in the duct exceeds the activation temperature of the intumescent material.

  NOTE Modern intumescent dampers can operate rapidly and have good ageing characteristics.

Windows in a fire-resisting wall are rarely openable, except under strict management control for cleaning or other purposes. If, however, a window is openable it, like a door, should be fitted with an adequate automatic closing device. Due to the potential for radiation exposure as a result of the presence of fire glass, the closer used should also demonstrate its ability not to ignite on the protected face. When larger openings are protected by roller shutters, vertically or horizontally ("side-winders"), they should be linked to the fire detection system and close in a controlled manner as defined in the cause and effect matrix. This information enables the fire containment of the enclosure to be defined at the onset of ignition.

8.1.4 Defining the behaviour of the enclosure post-flashover

When modelling/quantifying the fire exposure conditions within the enclosure after fire development, it is important to establish when the boundary elements forming that enclosure would fail to contain the fire. This permits the fire engineer to quantify the influence that failure would have on the fire exposure conditions and the tenability of the adjacent spaces. Boundary elements that do not have a specified fire separating function are harder to predict than elements that are, even in the trial design, designated with a particular fire separating function. Experience shows that it is almost impossible to quantify when integrity failure is likely to occur by anything other than tests, and tests are rarely complex enough to provide an accurate prediction of the behaviour in an actual building (see Table 1).

When the bounding elements forming the enclosure are not designed to have a fire separating function, the duration for which they contain the fire is the sum of the pre-flashover and post-flashover duration. Flashover does not occur until the ambient conditions reach at least 600 °C. For most traditional construction materials, exposure to temperatures below 600 °C (pre-flashover) does not change their state significantly and, they resist penetration by fire for an extended period. The boundary contains the fire and remains at its original dimensions throughout this exposure.

The exceptions to this rule are:

- composite panels of all core types where glue failure causes the delamination of facings;
- glasses where the critical surface temperature differentials are exceeded, e.g. soda/lime glass where the critical δT = 150 °C to 210 °C;
- cellulosic boards, including door assemblies, whose surface starts to be consumed once ignition temperatures have been reached, i.e. between 320 °C and 450 °C;

  NOTE Even then there is a significant delay before penetration of such components takes place due to the consumption of the material by the fire.
- low melting point services passing through the boundary where melting/fusion takes place leaving holes in the element which compromise the fire containment capabilities of the boundary, unless protected.

If the boundary elements do not include materials that are compromised by exposure to pre-flashover temperatures, the boundary is considered not to change shape or form during the pre-flashover phase. Where the boundary could be compromised by exposure to flashover conditions, this should be taken into account by the fire safety engineer responsible for quantifying the behaviour of the enclosure, who might in turn require the services of a material/construction specialist to determine the specific responses of these elements.

8.1.5 Predicting the integrity behaviour of the enclosure boundary (post-flashover)

It is simple to calculate the duration for which a boundary resists the effect of the post-flashover fire conditions if the elements have been subjected to the standard fire resistance test exposure. Such elements are assigned a period for which they satisfy integrity and, where appropriate, insulation. These values can be used with some confidence to determine the duration for which the enclosure can retain its original shape and resistance to penetration, assuming there is a reasonable correlation between the standard temperature/time curve and that predicted for the real building fire.

When tested elements are used to form the enclosure boundary, they only perform as tested if all critical aspects of their tested design are used in the final building. The effect of keeping a door closed after flashover depends upon numerous design features such as the face of the door to which the closer is attached. The type, dimensions and position of heat-activated sealing systems can result in the door achieving 60 minutes or 5 minutes. Unlatched fire doors aid evacuation, but are very demanding on the specification of the components forming the doorset. If a closer in an enclose opens in 10 minutes or less, the fire safety strategy is seriously compromised.

Care should be taken when applying the results of standard fire resistance tests to buildings because of the many differences that are likely to exist between the test and their use in practice. Annex D provides guidance for the fire safety engineer regarding the field of application of fire tests, the limits that apply to the extended application and how the construction might need modification to accommodate the intended use. Similar approaches can be used to take changes in exposure conditions into account if there is a poor correlation between the standard temperature/time conditions and those predicted for the building. Where the differences are extreme, improvised testing or testing with a modified fire curve can be considered.

When the boundary incorporates elements that are not designed to resist fire spread for any significant duration, it is difficult to quantify the positive or negative effects that this can have on the enclosure.

NOTE Table 13 gives some guidance as to what duration standard constructional materials can achieve when the exposure conditions are below the critical levels.

To avoid having to define the performance of enclosures that are not constructed from elements with a determinate level of fire resistance, the performance of non-fire-resisting sub-compartments is often ignored. This can lead to false predictions of the fire growth pattern by overestimating the availability of oxygen. It can also mask the possibility of localized severe exposure within the main compartment, resulting in the early failure of temperature-sensitive elements of the construction that can pass through or over the smaller non-fire-resisting enclosure. This could be critical in any CFD modelling, where windows breaking due to high localized temperatures could seriously change the assumed model. The rate of heating is the main cause of critical differential temperatures being reached on the surface of glass and this rate could be higher than predicted if a severe enclosure fire is introduced rapidly into the larger fire compartment, depending upon the relative volumes of both fires. The global compartment model should only be used once the impact of not using the enclosure of fire origin has been thoroughly evaluated.

8.1.6 Structural impact followed by fire

A review of past fires reveals that an extreme event often precedes the fire, such as an accidental explosion, an impact or a terrorist attack. Such an event has the potential to severely damage an enclosure, compromising its integrity and possibly structural capacity. The QDR can be used to identify the probability of such an event occurring. If the probability becomes significant, the enclosures and compartments within the zone likely to be affected by the event should be built in a robust manner.

The following considerations should be made.

- Separating elements (including compartmentation, partitions, windows, doors, etc.) are removed or damaged by a physical impact. This would change the fire characteristics and could reduce the performance of separating elements.
- The destruction of, or damage to, structural members should be considered within the analysis of the fire performance of the structure. Elements can either be removed from the analysis or deliberately weakened to account for damage.
- Passive protection could be destroyed, dislodged or weakened by a physical impact, and it might not be appropriate to assume that all elements of the enclosure have remained fully protected following the impact. Some forms of protection are better at resisting extreme events than others.
- Active forms of compartmentation, such as dampers and shutters, might no longer work as designed following a physical impact or damage to power supply, etc.
- The performance of active systems, such as smoke control and sprinkler systems, could be compromised by a physical impact.

The scope of this Published Document does not include robust design for these purposes, but the designer should, where necessary, identify a number of measures to make the construction more robust.

There should be no attempt to model the dynamics of the pre-flashover fire conditions until the considerations listed above have been resolved in a robust manner. Otherwise the maximum temperature, the time to reach it and pressures developed within the enclosure could be in error.

8.2 Accepted analytical methods

8.2.1 General

The definition of the enclosure, its key properties and the key parameters permit the determination of the effects of a fire on the integrity of the enclosure.

8.2.2 Enclosure layouts and geometries

The layout and geometry of the enclosure influences the thermal conditions expected during a fire and the mechanical response of the enclosure to such conditions.

Where there is more than one opening into an enclosure, the total area for the enclosure, A_w, can be considered as the cumulative total of individual ventilation areas, i.e.

$$A_w = A_{w1} + A_{w2} + ...A_{wi} \tag{8}$$

The characteristic width of openings for the enclosure, w, can be calculated from the cumulative total of individual opening widths as follows:

$$w = w_1 + w_2 + ...w_i \tag{9}$$

Where an enclosure contains multiple openings, their characteristic height, h_w, can be taken to be the weighted mean height, i.e.

$$h_w = \frac{h_{w1}A_{w1} + h_{w2}A_{w2} + ...h_{wi}A_{wi}}{A_w} \tag{10}$$

The identification of ventilation openings and potential routes for fire spread should take into consideration the issues discussed in **8.1**.

Where enclosures contain openings in the horizontal plane, the flow patterns of fire gases become more complicated. Pettersen et al. [8] provides guidance on considering horizontal openings where the extent of horizontal openings satisfies the relationship:

$$\frac{A_h\sqrt{h_{roof}}}{A_w\sqrt{h_w}} \le 1.76 \tag{11}$$

The area of equivalent ventilation in such circumstances should be determined by use of a modification factor, f_k, as follows:

$$A_w = f_k A_h \tag{12}$$

The modification factor, f_k, is a function of the geometry of the enclosure and is described in a nomogram in Figure 10.

If the relationship described in equation 11 is not achieved, the nomogram cannot be used and the flow through the horizontal openings is too significant to be treated using simplistic models. In such cases, PD 7974-2 gives guidance on the flow of hot gases via horizontal vents.

Alternatively, BS EN 1991-1-2 provides a method for considering horizontal openings with calculations on time equivalent of exposure with the ventilation factor calculated thus:

$$w_f = (6.0/H)^{0.3}[0.62 + 90(0.4 - \alpha_v)^4 / (1 + b_v\ \alpha_h)] \ge 0.5 \tag{13}$$

where:

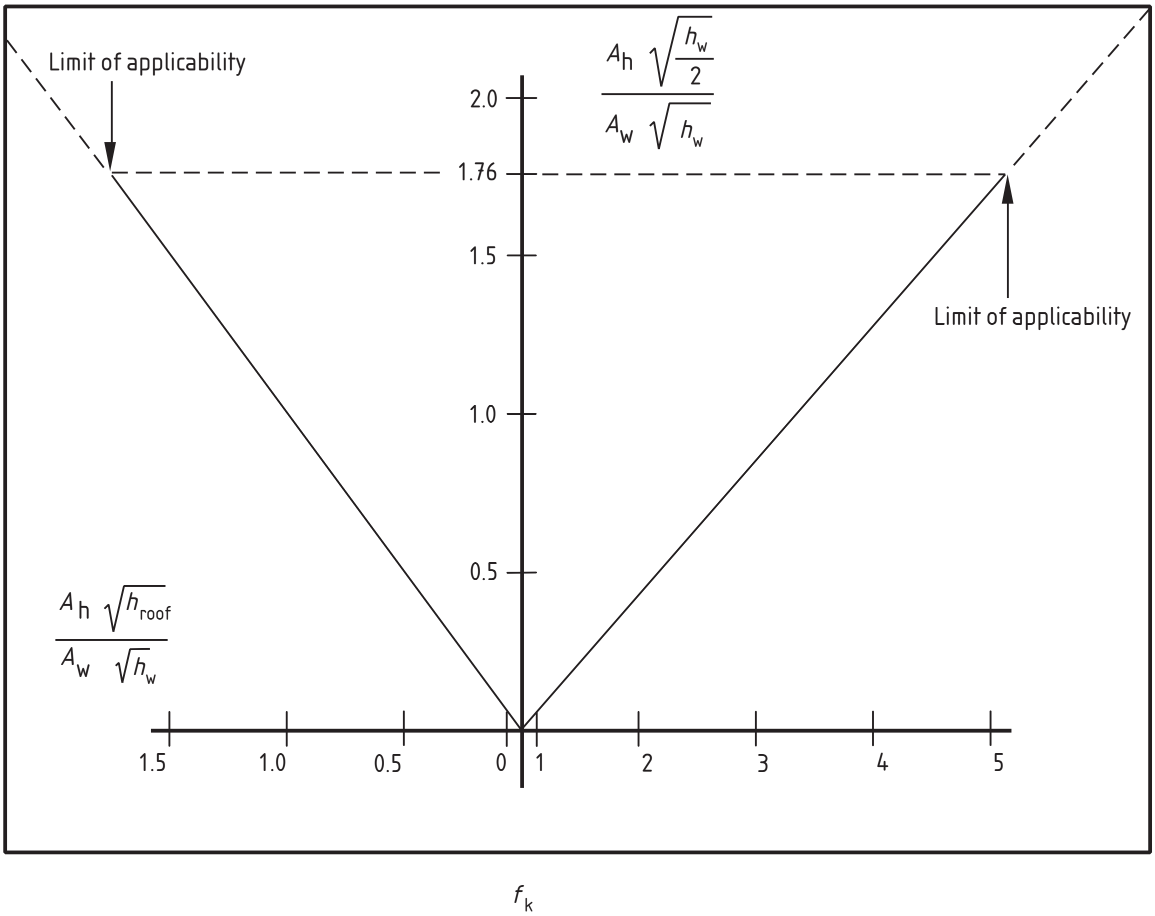
α_v is A_v/A_f which is the area of vertical openings in the façade (A_v) related to the floor area of the compartment (A_f), where the limit $0.025 \le \alpha_v \le 0.25$ should be observed;

α_h is A_h/A_f which is the area of horizontal openings in the roof (A_h) related to the floor area of the compartment (A_f);

$$b_v = 12.5(1 + 10\alpha_v - \alpha_v^2) \ge 10.0 \tag{14}$$

H is the height of the fire compartment in metres.

Figure 10 **Nomogram for modification factor for ventilation**



8.2.3 Thermal properties of the enclosure linings

The thermal properties of the enclosure linings can be usefully combined as a set of parameters that are widely used in analysis.

The thermal diffusivity, K, (m^2/s) is defined as:

$$K = \frac{\lambda}{\rho C} \tag{15}$$

and provides a measure of the surface temperature expected on the enclosure boundary.

The thermal inertia, b, ($W/m^2s^{½}K$) is defined as:

$$b = \sqrt{\lambda \rho C} \tag{16}$$

and provides a measure of the heat absorbed into the boundary element.

Information on the thermal properties for some materials is given in Annex A. The thermal diffusivity can differ by at least two orders of magnitude and has a significant effect on the compartment temperatures and heat loss through the linings.

The thermal inertia for the enclosure, b, is averaged spatially to consider the differing thermal inertia of walls, floors, ceilings, etc. as follows:

$$b = \frac{A_1 b_1 + A_2 b_2 + ...A_i b_i}{\left(A_1 + A_2 .....A_i\right) - A_v} \qquad (17)$$

where:

A_i is the area of element i (m^2);

A_v is the area of ventilation in the vertical plane (m^2);

b_i is the thermal inertia of element i ($W/m^2s^{½} K$).

If enclosure surfaces comprise layers of different materials, it is the layers directly exposed to fire that have the greatest impact on the temperatures achieved within the enclosure. Rules are provided in BS EN 1991-1-2 for the treatment of multiple layers.

Assuming, for example, that the fire conditions within the enclosure are required for a one hour period, and that the enclosure is bounded by multi-layered lining materials having diffusivities of 5×10^{-7} m^2/s, then generally only the layers within 15 mm of the exposed surface need be considered when calculating the thermal inertia, b.

9 Characterization of the fire conditions

9.1 Characterization according to air temperature/time profile and total heat flux

9.1.1 Forms of analysis

The fire conditions within an enclosure are generally described in terms of either an air temperature/time profile (see **9.1.2**) or a total heat flux/time profile (see **9.1.3**). The level of detail required in the description of the fire conditions is influenced by the level of analysis subsequently employed. For example, the maximum temperature elements are exposed to determines their mechanical response. In such circumstances, details of the entire temperature/time profile within the enclosure are unnecessary once the maximum temperature is known (see Table 1 for guidance on appropriate combinations of analysis methods for thermal and mechanical response).

Fire development can be considered in four distinct fire phases:

1) Ignition

Ignition has occurred but the fire has not entered the growth phase. Incipient fires can self-extinguish or be extinguished prior to entering the growth phase. It is impossible to predict the duration of the incipient phase and, therefore, it is normally assumed that the fire enters the growth phase immediately after ignition.

2) Growth

The amount of material being consumed by the fire increases and the fire grows. The rate of burning and rate of growth is a function of the fuel load characteristics, the amount of combustible material and the amount of oxygen available.

3) Full development

The fire is unchecked and grows until it reaches a maximum size in terms of its dimensions or the rate of heat output. The rate of heat release in the fully-developed phase is governed by the type or amount of fuel available (fuel-controlled) or by the amount of ventilation that is available for the combustion process (ventilation-controlled).

4) Decay

Following a sustained period of burning, the amount of fuel load available for combustion reduces and the fire begins to decay. The rate of heat release reduces to the point at which the fire becomes extinct.

The nature of the fire varies depending on the fire phase and size of the fire relative to its surroundings. Fire can be considered as free-field, or fully interacting with its surroundings.

There are three different types of fire:

a) Isolated (see PD 7974-1)

Relative to its surroundings, the fire is effectively uncontained but is sufficiently small for there to be no interaction between the fire and the enclosure.

b) Contained (see PD 7974-2)

The products of heat and combustion are contained by the fire enclosure, but flash-over conditions have not been reached. Contained fires are characterized by two-zone behaviour: a high level smoke zone above a smoke-free zone. The fire is assumed to be in the smoke-free zone.

c) Compartment

If flashover conditions develop, the fire becomes a compartment fire. Such fires are characterized by single-zone behaviour where the fire has consumed or is consuming the entire contents of the fire enclosure.

9.1.2 Temperature-based description of fire conditions

PD 7974-1 gives guidance for establishing the temperature/time behaviour of a fire. However, quantification of the fire conditions for fire spread (as opposed to life safety of occupants) is often considered separately where there could be environmental, heritage or property loss issues, using the following approaches:

a) characterization of fire conditions using standardized models of fire (by fire type or occupancy type), i.e. fire resistance furnace tests;

b) determination of the fire conditions from experimental investigation;

c) determination of the fire conditions through engineering calculations.

9.1.3 Heat flux based description of fire conditions

The total heat flux, q_{total}, produced under fire conditions, is defined by:

$$q_{total} = q_c + q_r \tag{18}$$

where:

q_{total} is the total heat flux (kW/m^2);

q_c is the convective heat flux component (kW/m^2);

q_r is the radiative heat flux component (kW/m^2).

The fire temperature and the nature of the fuel being burned largely determine the relative magnitude of the convective and radiative heat fluxes.

9.2 Characterization using standardized models of fire and standard furnace tests

9.2.1 General

PD 7974-1 describes the use of fire curves that reflect the temperature/time development of fires in enclosures prior to flashover in terms of slow, medium, fast or ultra-fast growth. In post-flashover conditions, alternative standardized temperature/time relationships should be employed.

These standardized temperature/time relationships originated from furnace fire test methodologies and do not necessarily represent real fire conditions within an enclosure. For example, the furnace heating curve given in BS 476-20 (BS EN 1363-1 and ISO 834-1) is largely based upon the original work by Ingberg [9], during the 1920s. This has been adopted globally and serves as a bench-marking process for regulatory use whereby the performance of one material/product can be compared to another for specific applications in building construction.

In contrast, other, more recently developed furnace heating curves, such as the hydrocarbon and tunnel curves, do accurately represent the fire conditions which they describe.

With the exception of the tunnel curves, various standard temperature/time relationships are given in PD 7974-1 (see **9.2.2**).

9.2.2 Standard temperature/time relationship

A standard temperature/time fire model used to characterize fires in enclosures containing typical cellulosic contents is given by:

$$T_g = 20 + 345 \log_{10} (8t + 1) \tag{19}$$

where:

T_g is the mean fire gas temperature in the enclosure or furnace (°C);

t is the time from ignition (in minutes).

The fire model described by equation 19 is the basis of fire resistance tests undertaken to BS 476-20:1987, BS EN 1363-1 and ISO 834-1. While this model is referred to as the cellulosic curve, a proportion of the fire loading in most buildings is made up of hydrocarbon (plastic) based materials, which could give rise to higher temperatures in a shorter time period. Other factors, such as the quantity and distribution of the fire loading, the available ventilation and enclosure properties can give rise to a temperature/time response that can either be faster or slower than that given by equation 19.

9.2.3 Hydrocarbon temperature/time relationship

A temperature/time fire model used to characterize more severe fires in enclosures containing hydrocarbon-based contents is specified in BS 476-20 (see Annex D) by:

$$\theta_g = 1\,100(1 - 0.325e^{-0.1667t} - 0.204e^{-1.47t} - 0.471e^{-15.833t}) \tag{20}$$

The above relationship is similar to that specified in ISO/TR 834-2 and BS EN 1363-2 by:

$$\theta_g = 1080(1 - 0.325e^{-0.167t} - 0.675e^{-25t}) + 20 \quad (21)$$

This type of furnace test is considered representative of a hydrocarbon pool fire.

9.2.4 External temperature/time relationship

A temperature/time fire model used to characterize less severe fires immediately outside typical enclosures, and emanating from within the building, e.g. flames issuing from adjacent windows, is given by:

$$T_g = 660(1 - 0.678e^{-0.32t} - 0.313e^{-3.8t}) + 20 \quad (22)$$

9.2.5 Smouldering temperature/time relationship

A slow heating curve is also described in BS EN 1363-2, in which the temperature within the furnace attains 300 °C after 20 minutes and then joins the cellulosic curve. This is not used as a thermal action for design purposes, but could be used for reactive fire protection materials which rely upon a chemical/physical change in order to function as an insulator.

However, where fire safety devices rely only upon the action of hot smoke to function it is recommended that, until there is an internationally agreed temperature/time curve, the smouldering fire curve specified in BS EN 1363-2 by the following equation is used:

For $0 < t \leq 21$:

$$T_g = 20 + 154t^{0.25} \quad (23)$$

For $t > 21$:

$$\theta_g = 20 + 345\log_{10}\left(8t + 1\right) \quad (24)$$

9.2.6 Rijkswaterstaat (RWS) tunnel curve

The RWS tunnel curve originated from tests carried out in 1979 on a model tunnel in which the temperatures attained 1 350 °C after 60 minutes. The RWS curve was developed by the Rijkswaterstaat Ministry of Transport in the Netherlands and was based on the assumption that, in a worst case scenario, a fuel, oil or petrol tanker fire with a heat release of 300 MW could occur and last for up to 120 minutes.

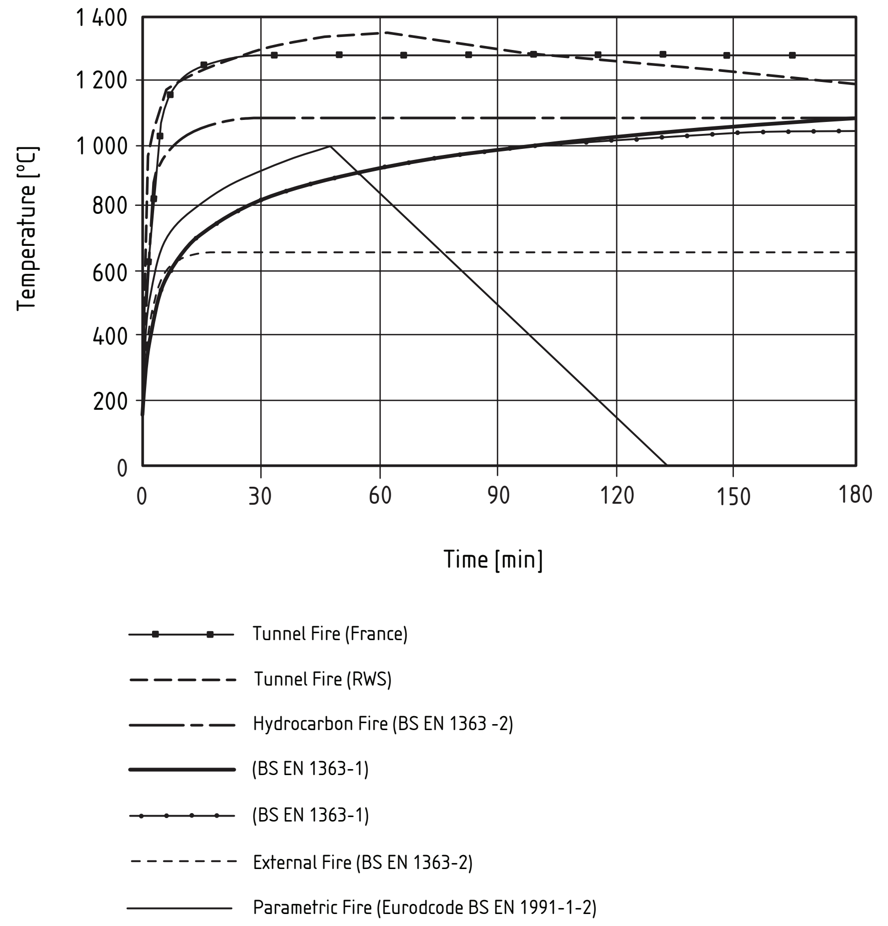
The primary difference between the RWS and the hydrocarbon heating conditions is that the hydrocarbon curve is based on the temperatures expected from a fire occurring within a relatively open space, where some dissipation of the heat would occur. In contrast, the RWS curve is based upon the conditions encountered by a fire occurring in an enclosed area where there is little or no opportunity of heat dissipating into the surrounding atmosphere.

The RWS curve simulates the initial rapid growth of a fire using a petroleum tanker as the source, and the gradual drop in temperature reflects the reduction in fire load as the petroleum is burnt off. It has been used as a standard for the protection of tunnels in the Netherlands since the 1980s.

The heating portion of the RWS tunnel curve can be obtained by replacing 1 080 °C in equation 21 of the hydrocarbon curve with 1 350 °C.

In France, a similar curve to RWS is adopted for tunnels. Figure 11 compares standardized furnace heating curves currently in use.

Figure 11 **Nominal standard fire curves**



9.2.7 Duration of fire to be adopted in design

All fires have a finite fuel supply and, once the fuel is exhausted and starts to decay, conditions become less severe and the fire eventually extinguishes. Whilst the amount of fuel or fire load present is an important variable in determining fire duration, so too is the burning rate of the fire. This is influenced by variables including the extent of available ventilation and the thermal properties of the enclosure.

Characterization of fire conditions in terms of a standardized temperature/time profile for a set duration is a key design decision in determining the propensity for fire spread. The decision should reasonably reflect less quantifiable concerns such as the risk of a post-flashover fire occurring within the enclosure, and the consequences of fire spread. For this reason, recommendations on the design duration of standardized fire conditions are given in a range of codes of practice and regulatory documents (see Foreword).

The appropriate duration, as defined in the Building Regulations ([1], [10] and [11]), of the standard heating curve (see equation 19) is influenced by the:

a) nature of occupancy of the enclosure;

b) presence of an automatic sprinkler system;

c) height of enclosure above ground level;

d) depth of enclosure below ground level;

e) size of the compartment.

Alternatively, the design duration of the standardized fire conditions can be determined by using the engineering calculation of the time-equivalent value as described in **9.2.8**.

9.2.8 Equivalent time of fire exposure

The anticipated fire conditions within the enclosure can be characterized with reference to a set duration of the standardized gas temperature/time relationship described in **9.2.1** (see BS 476-20). The duration of exposure, known as the equivalent time of fire exposure, or time-equivalent value, is derived empirically.

The time-equivalent value can only be applied to the structural response of loadbearing elements and it is not appropriate for use where the insulation or integrity of enclosures is considered.

A number of researchers have proposed methods for correlating durations of exposure in the standard test to real fires. The most notable have been developed by Law [12], Harmathy [13] and Pettersen [14].

In BS EN 1991-1-2 exposure of boundary elements to the standardized fire conditions represents an equivalent level of thermal exposure as exposure to the full duration of a fire within the building enclosure. It also states the application time-equivalent excludes timber structures, although the severity of the fire is independent of the construction materials.

The background to these relationships was developed from DIN 18230-1 and subsequently in CIB W14 [15].

In BS EN 1991-1-2 the duration of time equivalence, t_e, is given by:

$$t_e = k_b w_f q \quad (25)$$

where:

t_e is the duration of heating in a standard fire resistance test furnace (min);

k_b is the factor that describes the thermal properties of the enclosure;

NOTE In NA to BS EN 1991-1-2 where no detailed assessment of the thermal properties of the enclosure is made, or for building surfaces with high levels of insulation, e.g. proprietary wall insulation systems, k_b should be allocated a value of 0.09.

q is the design fire load density per unit floor area (MJ/m^2) as input from the QDR.

In other instances k_b can be evaluated using Table 4.

Table 4 **Values of k_b**

b J/m^{2}s^{½}K	k_b min m^2/MJ
$b < 720$	0.09
$720 \leq b \leq 2\,500$	0.07
$b > 2\,500$	0.05

w_f is the ventilation factor and is defined by:

$$w_f = 6.0\, H^{-0.3}\left[0.62 + 90(0.4 - \alpha_v)^4\right](1 + b_v \alpha_h)^{-1} \geq 0.5 \quad (26)$$

where:

$\alpha_v = A_v / A_f$ is the area of the vertical openings in the façade (A_v) related to the floor area of the compartment (A_f);

$\alpha_h = A_h / A_f$ is the area of the horizontal openings in the roof (A_h) related to the floor area of the compartment (A_f);

A_v is the area of ventilation in the vertical plane (m^2);

A_h is the area of ventilation in the horizontal plane (m^2);

A_f is the floor area of the enclosure (m^2);

b_v is given by:

$$b_v = 12.5\left[1 + 10(A_v / A_f) - (A_v / A_f)^2\right] \geq 10 \quad (27)$$

Equation 25 has been shown, through large scale fire tests, to exhibit a reasonable degree of correlation with the behaviour of protected steel members observed in fire resistance tests (see Kirby et al. [16]).

For construction materials, reinforced concrete, protected steel and unprotected steel, BS EN 1993-1-2:2005 introduces a correction factor k_c, which is equal to unity except for unprotected steel for which a value of 13.7 × O is given. O is the opening factor for the compartment and introducing this factor into the formulation reflects the low thermal mass of bare steel. Steel temperatures closely follow the compartment temperatures and these are governed by the heat release rate, a function of the opening factor. However, work by Kirby and Tomlinson [17] demonstrated that the time-equivalent relationship for unprotected steel can only be validated for fire resistance periods up 30 minutes using a correction factor $k_c = 1$, and found no correlation with the factor 13.7 × O.

Information on the fire load densities for different occupancies is given in PD 7974-1 and is associated with specific percentage fractiles. BS EN 1991-1-2:2002, Annex E, introduces multiplication factors based upon specific fire precautions that can be incorporated into the design of buildings. For example, in BS EN 1991-1-2:2002, Table E.2, the inclusion of automatic fire detection and alarms would enable factors of 0.87 and 0.73 for heat and smoke to be applied to the design fire-load density to reflect the reduced risk in fire safety. The UK considered this approach unacceptable in the engineering calculations for fire severity.

By applying a number of design factors to the fire-load, it would be possible to reduce the design fire-load density to such a low value that flashover might not even occur. It is therefore incorrect for fire severity to be linked with precautions that have little to do with the actual fire. Equivalent fire severity should be determined based entirely upon engineering calculations, with risk and consequences to life safety carried out separately, taking into account the size of the building, its occupancy and construction parameters.

Engineers should be wary in the use of time-equivalent calculations as the outputs provide only part of the solution and should not be applied in isolation. For instance, the engineering (deterministic) calculations would give the same level of equivalent fire severity irrespective of whether the fire occurred in a 2-storey or a 30-storey building. However, the consequences of failure and the level of risk are substantially greater in the taller building and therefore a higher safety factor should be applied to the engineering outputs. This has been addressed in BS 9999 which applies a risk-based approach to developing the fire resistance times for building occupancy characteristics, as presented in BS 9999:2008, Table 26.

The background to the analysis in BS 9999 is based upon the premise:

risk = frequency × probability × consequence of failure.

9.3 Determination of fire conditions from experimental investigation – characteristic design fires by occupancy

Over the years a range of experimental fires have been observed in realistic simulations of typical buildings [17, pp.17-26]. Interpreting the test results can inform the designer about anticipated fire conditions in typical occupancies, such as offices and car parks. Adopting such design fires might be reasonable where the enclosure is well represented by the underlying experimental set-up, particularly with regards to:

- fire load density;
- nature of the fire load;
- extent of ventilation;
- geometry of the enclosure;
- thermal properties of the enclosure.

Alternatively, the fire engineer could design and commission appropriate experimental fire tests to model a particular enclosure scenario, observing the principles set out in BS 6336, BS 476-33 and ISO/TR 22898. Only competent persons or bodies, such as research institutions and test laboratories, should undertake experimental fire tests.

The results of ad-hoc fire tests should be subject to careful evaluation before being integrated into the design process. As with the use of standardized fire models, less quantifiable factors relating to risk and consequence should be considered when arriving at a set of design fire conditions. Consideration of such factors could result in the application of safety factors to the observed fire temperature/time profile.

9.4 Determination of fire conditions through engineering calculations

9.4.1 Prediction of maximum fire temperature within enclosures

The temperatures developed in post-flashover fires are discussed in PD 7974-1. The key relationships are repeated here, but reference should be made to PD 7974-1 to confirm the basis of the approach and the limits to be observed.

The maximum temperature attained during a ventilation-controlled fire within enclosures bounded by materials having a thermal inertia in the range of 720 $J/m^2s^{1/2}K$ to 2 500 $J/m^2s^{1/2}K$ is given by:

$$T_{g,max} - T_0 = 6\,000(1 - e^{-0.10\eta})\eta^{-0.5}(1 - e^{-0.05\psi}) \quad (28)$$

where:

$T_{g,max}$ is the maximum temperature in the fire enclosure (K);

T_0 is ambient temperature (293 K);

η is $A_t/(A_w\sqrt{h_w})(m^{0.5})$;

ψ is $L/(A_wA_T)^{0.5}$ (kg/m^2);

where:

A_T is the internal solid area of the enclosure (m^2);

L is the total fire load in the enclosure as an equivalent quantity of wood (kg).

Where an enclosure is likely to experience a through-draught condition from openings on opposing walls, the temperatures attained within the enclosure are lower, as determined by the relationship:

$$T_{g,max} - T_0 = 1200(1 - e^{-0.04\psi}) \quad (29)$$

NOTE These relationships are based upon the work by Law [18].

In BS EN 1991-1-2 these relationships have been modified to give similar, but different, temperature outputs using the following relationships for:

a) no through-draught conditions:

$$T_{g,\,max} - T_0 = 6\,000(1 - e^{-0.10\eta})\eta^{-0.5}(1 - e^{-0.036\psi}) \quad (30)$$

and

b) for through-draught conditions:

$$T_f - T_0 = 1200\left[\left(A_f q\right)/17.5 - e^{0.002\,28\psi}\right] \quad (31)$$

It is possible for extremely high temperatures to be computed which are unrealistic in the context of building fires. NA to BS EN 1991-1-2 suggests that an upper limit of 1 750 K should be applied to gas temperatures within compartment fires.

9.4.2 Characterizing transient fire conditions within enclosures

A heat energy balance equation can be used to predict the temperature/time profile of the hot gases within an enclosure represented by equation 32:

$$Q_{\mathrm{total}} = Q_{\mathrm{c}} + Q_{\mathrm{w}} + Q_{\mathrm{r}} + Q_{\mathrm{g}} \qquad (32)$$

where:

Q_{total} is the rate of heat release in the enclosure (kW);

Q_{c} is the rate of heat loss by convection through openings (kW);

Q_{w} is the rate of heat loss to the enclosing construction (kW);

Q_{r} is the rate of heat loss by radiation through openings (kW);

Q_{g} is the rate of accumulation of heat in hot gases (kW).

Gas temperature/time curves have been derived for enclosures having average thermal properties (e.g. brickwork, blockwork or plaster) as a function of fire loading and ventilation. These relationships were first introduced in DD ENV 1991-2-2:1996 and have been substantially modified in the light of more recent research and further analysis carried out in Europe (including the UK). These are presented in BS EN 1991-1-2 which superseded DD ENV 1991-2-2, with further modifications given in NA to BS EN 1991-1-2.

The temperature of the fire gas, T_{g}, in the heating phase is given by:

$$T_{\mathrm{g}} = 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) \qquad (33)$$

where:

T_{g} is the fire gas temperature in the enclosure (°C);

t^* is the modified time (hours) and = Γt.

where:

t is the time from ignition (hours);

Γ is the compartment time factor (dimensionless)

and = $\left(\dfrac{O}{b}\right)^2 \left(\dfrac{1160}{0.04}\right)^2$

b is the thermal inertia ($\mathrm{J/m^2s^{1/2}K}$) and is subject to the limit $100 \le b \le 2\,200$;

O is the opening factor ($\mathrm{m^{1/2}}$) subject to the limit $0.01 \le O \le 0.20$ and

$= (A_{\mathrm{w}}\sqrt{h_{\mathrm{w}}}) / A_{\mathrm{t}}$

The maximum temperature, $T_{\mathrm{g,max}}$, is reached after a time t_{max} when:

$$t^* = \Gamma t_{\mathrm{max}} \qquad (34)$$

with:

$$t_{max} = \frac{0.2 \times 10^{-3}}{O} q \; t_{lim} \qquad (35)$$

where:

t_{max} is the time when the maximum temperature is attained (hours);

q is the design fire load density per unit area of enclosure surface (MJ/m^2) and = L/A_t subject to the limit $50 \le q \le 1\,000$ MJ/m^2;

L is the total fire load in the enclosure (MJ);

t_{lim} is the fire growth rate which = 25, 20 or 15 corresponding to slow medium and fast growth rates.

BS EN 1991-1-2 gives further advice on the transition from ventilation to fire load controlled fires. Since the publication of DD ENV 1991-2-2:1996, the value 0.2 in equation 35 has been under further consideration and might be formally amended in the near future.

The temperature decay, at times beyond the time of maximum temperature, has also been modified since the publication of DD ENV 1991-2-2:1996 and is now described by the following relationships:

$$T_g = T_{g,max} - 625(t^* - t^*_{max}x) \text{ for } t^*_{max} \le 0.5 \qquad (36)$$

$$T_g = T_{g,max} - 250(3 - t^*_{max})(t^* - t^*_{max}x) \text{ for } 0.5 < t^*_{max} < 2 \qquad (37)$$

$$T_g = T_{g,max} - 250(t^* - t^*_{max}x) \text{ for } t^*_{max} \ge 2 \qquad (38)$$

where:

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

Full-scale fire tests conducted by Corus and Fire Risk Sciences (FRS), a division of the (BRE), have provided reasonable correlation for compartments constructed using typical materials typical used in buildings, such as concrete floors and lightweight concrete blockwork. This is an improvement on an earlier method developed by Pettersen et al [8]. Tests carried out by Lennon [19] of FRS as part of a major European Coal and Steel Society (ESCS) research programme, have demonstrated that the parametric expressions specified in BS EN 1991-1-2 either correlate to experimental data or overestimate (safe) compartment temperatures.

Another major improvement in the parametric expressions since the publication of DD ENV 1991-2-2:1996 is in the treatment of the thermal properties of the enclosure boundaries ($b = \sqrt{\rho c \lambda}$) on the temperature/time response of the fire. This now attaches greater significance to the properties of the material at the surface or just below the internal surface of the enclosure exposed to fire.

In NA to BS EN 1991-1-2 there is no limitation in the use of the enclosure fire models described in equations 33 to 38 on the floor area, and the compartment height need not be limited to 4 m. Although compartment heights much greater than 4 m can overestimate the (safe) temperatures considerably, alternative computational fluid dynamics (CFD) or plume models provide more realistic temperatures.

Recent research in the UK and Australia has shown that compartments with openings in a single wall, with a width to depth ratio in excess of 1:6 experience uneven burning along the length of the compartment. Further and more detailed consideration is required in such instances.

Equations 33 to 38 can be used to develop solutions using computer software and have the benefit of producing full temperature/time relationship histories for fully-developed fires bounded by enclosures. Nonetheless, the designer should seek full details of model validation prior to its application for real building design and should be confident of its suitability for purpose and the validity of its underlying assumptions.

9.4.3 Characterizing the condition of fires spreading from openings in enclosures

When examining the potential for convective or radiative fire spread from an enclosure, it might be necessary to characterize the shape, size and temperature profile of flames emerging from openings in the enclosure boundaries.

In the first instance, the fire can be assumed to completely fill the area of the opening of the enclosure, burning at a temperature not less than the gas temperatures assumed within the enclosure. When evaluating the potential for fire spread to surfaces outside the enclosure, the emissivity of the fire at its source should be taken as unity.

Another convenient, if conservative approach, is to assume that the entire area of the opening acts as a heat radiator. The level of radiation can be calculated from the enclosure gas temperatures or can be allocated notional values as shown in Table 5.

Table 5 **Notional radiation levels from openings in enclosures**

Enclosure characterization	**Radiation from opening**
Residential, office, assembly, recreation or open-sided car parks	84 kW/m^2
Shop, commercial, industrial, storage or other non-residential	168 kW/m^2

More advanced analysis allows consideration of the failure of boundaries, e.g. glazing, and the emergence of the fire through the plane of the boundary enclosure, i.e. flames issuing through openings.

The flame height, width and horizontal projection from the enclosure can be calculated using the methods developed by Law and O'Brien [18].

In BS EN 1991-1-2 the relationships have been modified and provide similar outputs to the original work in which both through-draught and non-through-draught conditions are considered.

It is possible to achieve extremely high temperatures for both the internal compartment and external flames, which can exceed the temperatures expected in building fires. In NA to BS EN 1991-1-2, it is recommended that upper limits of 1 750 °K and 1 850 °K are applied respectively. It is recommended the method is used only for fire loads greater than 200 MJ/m^2.

Where the calculated flame height is negative, it indicates the tip of the flame is no higher than the top of the window.

The methodology is based upon achieving steady state conditions and Law and O'Brien [18] suggest that, once the fire load exceeds

50 kg of wood /m^2 of floor area, any further increase in fire load would have little effect on the temperatures of the fire within the enclosure and the external flaming. Whilst the method was developed to characterize flames issuing from external windows, it can also be used to define flames issuing from internal openings into larger spaces, e.g. from doors/windows into atria.

The flow of hot gases from an enclosure entrains air and forms a smoke plume in the adjacent space. The recommended mass entrainment [20] into the plume above an opening, $M_{opening}$, is given by:

$$M_{opening} = 0.68(A_w\sqrt{h_w})^{\frac{1}{3}}(Z_w + a)^{\frac{5}{3}} + 1.59A_w\sqrt{h_w} \tag{39}$$

where:

$M_{opening}$ is the mass flow rate in plume at height, Z_w, (kg/s);

A_w is the area of opening (m^2);

h_w is the height of opening (m);

Z_w is the height above the top of the opening (m);

a is the effective height (m) = $2.4A_w^{\frac{2}{5}}h_w^{\frac{1}{5}} - 2.1h_w$.

Entrainment equation (39) assumes a convective heat release, q_{conv} (kW), from the enclosure of:

$$q_{conv} = 1260A_w\sqrt{h_w} \tag{40}$$

Details on the methods used for determining the spread of smoke and hot gases are given in PD 7974-2.

9.5 Effect of automatic fire suppression systems on fire conditions

Automatic fire suppression systems control the growth and spread of a fire. Accordingly, fires starting within enclosures containing such systems can be considered controlled within an area of burning consistent with the spatial configuration of the suppression system. For example, a maximum fire area of 12 m^2 can be assumed when evaluating conditions in an enclosure protected by an automatic sprinkler system with an array of heads on a 3 m × 4 m grid.

Suppression systems that actively remove heat from the enclosure, e.g. water-based, can reduce the severity of a fire in terms of enclosure temperatures. This effect is difficult to quantify, although it is often assumed that the heat-release rate of the fire remains fixed at the point at which the system is first activated. However, when characterizing the fire condition in terms of time equivalence or the heat flux from flames from openings, the reductions in Table 6 can be applied.

Table 6 **Effect of automatic sprinklers on expected fire conditions**

Characterization of fire condition	Effect of sprinklers
Time equivalent value (see **9.4.2**)	Value reduced to 61% of value calculated in accordance with **9.4.2**
Heat flux from opening in enclosure (see **9.4.3**)	Heat flux reduced to 50% of value described in Table 5

10 Analysis of thermal response

10.1 Thermal response of elements within enclosure

10.1.1 General

It is essential to predict the thermal response of any element when exposed to fire conditions. This can be important in different contexts, for example, the thermal response of the:

- enclosure boundaries can influence the heat balance within the enclosure and, accordingly, the ongoing fire conditions;
- enclosure boundaries can influence their mechanical response;
- members or objects outside the fire enclosure can determine whether fire is transmitted outside the enclosure; and
- the structural members can influence their mechanical response.

The thermal response of structural members can be determined from:

a) empirical data based upon fire resistance tests or tests performed under natural (real) conditions (**10.1.2**);

b) simplistic calculations of the temperature response from tests (**10.1.3**);

c) advanced calculations (**10.1.4**).

The thermal properties for each construction material are given in Annex A.

10.1.2 Empirical data

The response of various construction assemblies to a heating environment has been observed under a range of conditions. The behaviour of construction assemblies subjected to the BS 476:1932 test was compiled in the 1953 National Building Studies Research Paper No. 12 [21]. The results of most fire tests are in commercial confidence and the designer can obtain detailed thermal information directly from product manufacturers if necessary. However, some information is in the public domain, particularly for generic building materials such as steel, concrete, masonry and timber.

10.1.3 Simplistic calculations

Several materials have been extensively evaluated under the standard fire resistance test using either full-size or indicative specimens. The results have then been used to develop relationships between the size of member versus heating rates under different fire conditions and temperature profiles/gradients through the structural element in the form of design charts or nomograms for use by non-specialist fire engineers.

Where the thermal properties of a structural element are known, the temperature/time response can be calculated using basic heat transfer theory.

The energy from the fire environment is imparted to any exposed elements within, or outside, the enclosure by means of an imposed heat flux containing both convective and radiative components. The net heat flux incident upon the structure, q_{net}, can be related to the expected temperature of a fire within the compartment using the relationship:

$$q_{net} = q_{net,c} + q_{net,r} \tag{41}$$

where:

q_{net} is the net incident heat flux (kW/m^2);

$q_{net,c}$ is the convective heat flux (kW/m^2);

$q_{net,r}$ is the radiative heat flux (kW/m^2).

The heat fluxes can be calculated directly from the gas temperatures within the fire enclosure by calibrating the theoretical models against the results observed in fire resistance tests. In DD ENV 1991-2-2 heat transfer coefficients were applied for various materials in order to reflect national experience. These are no longer included in BS EN 1991-1-2.

Heat transfer by convection follows Newton's Law, where the heat flux is proportional to the difference in temperature between that of the exposed element, $T_{surface}$, and the surrounding hot fire gases, T_g. The convective heat flux per unit surface area, q_c, of the element is given by:

$$q_c = \alpha_c (T_g - T_{surface}) \tag{42}$$

where:

q_c is the convective heat flux (kW/m^2);

α_c is the coefficient of heat transfer by convection (W/m^{2}K);

T_g is the temperature of the fire gases (K);

$T_{surface}$ is the temperature of the exposed surface element (K).

The convective heat transfer coefficient, α_c, is a function of the fire gases flow pattern and velocity, and can be difficult to quantify in practice. For fully-developed fires the contribution of convection to the hot-face heat transfer is small and α_c should be assigned a value of 25 kW/m^{2}K (independent of temperature). This value could also be conservatively used for less severe or growing fires. For more severe fully-developed fires a higher value for α_c of 50 kW/m^2 is more appropriate. At the non-exposed face, α_c should be assigned a value of 4 kW/m^2 or 9 kW/m^2 where the effects of radiation are considered.

When evaluating the radiative heat transfer between fires and enclosure surfaces, the relationship becomes more complex as the ongoing interaction between the fire and the receiving surface causes the amount of radiative heat transfer to change continually. The interaction is modelled reasonably well by the relationship:

$$q_r = \phi \varepsilon_m \varepsilon_f \sigma \left[(T_g)^4 - (T_{surface})^4 \right] \tag{43}$$

where:

T_g is the temperature of fire gases within the compartment (K);

$T_{surface}$ is the surface temperature of the exposed element (K);

σ is the Stefan-Boltzmann constant, i.e. 5.67 × 10^{-8} W/m^{2}K^4;

ϕ is the configuration factor, describing the geometrical relationship between the radiating hot gases and the receiving surface;

NOTE In the absence of further analysis, the configuration factor can be set at unity. Practical values for the configuration factor are illustrated in Figure 9.

ε_m is the surface emissivity of the member;

ε_f is the emissivity of the fire.

The emissivity of the fire is taken as 1.0. The emissivity of the material varies according to Table 7.

Table 7 **Guidance on the material surface emissivity of construction materials**

Material	**Surface emissivity** ε_m
Concrete	1.0
Steel (carbon)	0.8
Stainless steel	0.63
Timber	1.0
Masonry	1.0
Aluminium	0.3–0.7
Glass	1.0
Plastics	1.0
Gypsum plaster	1.0
Mineral fibre	1.0
Generic fire protection materials	1.0

NOTE The emissivity relationships and material parameters have changed since BS EN 1991-1-2 was published.

10.1.4 Advanced calculations

The temperature response of the exposed elements to the imposed heat flux is governed by the geometry and construction of each of the exposed structural members. The prediction of the temperature response of the element itself requires solving the governing equation of transient conduction subject to the appropriate boundary conditions, as described by equation 44 within homogeneous solids.

$$\frac{\delta}{\delta x}\left(K\frac{\delta T}{\delta x}\right)+\frac{\delta}{\delta y}\left(K\frac{\delta T}{\delta y}\right)+\frac{\delta}{\delta z}\left(K\frac{\delta T}{\delta z}\right)+\frac{Q}{\rho C}=\frac{\delta T}{\delta t} \qquad (44)$$

where:

T is temperature (K);

x,y,z are planes of reference;

t is time (s);

Q is internally generated heat (kW);

ρ is density (kg/m^3);

C is specific heat capacity (J/kgK);

K is thermal diffusivity (m^2/s).

Given the transient conditions inherent in equation 44 it is not possible to offer an exact solution, even for relatively simple boundary conditions. Numerical analysis is typically used to find practical solutions using the finite element or finite difference methods. Computer software packages have been used to solve equation 44. However, the designer should be satisfied that any model used is valid. Typically the use of numerical models requires the choice of a time step for analysis. The smaller the time step, the more likely it is that a convergent solution and accurate results will be achieved. However, small time steps also increase the number of calculations required and increase model run times. Within finite, element-based systems a similar effect is encountered with the allocation of mesh size. The designer should be satisfied that key model parameters such as time step and mesh size have been set at a level at which reductions cannot meaningfully increase the accuracy of the solution.

Detailed discussion of the available software that has been developed for each of the construction materials, where it exists, is beyond the scope of this Published Document so only empirical data and simplistic calculations are discussed in **10.2** to **10.8**.

10.2 Thermal response of reinforced concrete members

10.2.1 General

Concrete covers a wide range of products, which essentially consist of a mixture of Portland cement and aggregates that can be siliceous (flint, granite and gravel), calcareous (limestone) and lightweight (sintered fuel ash, expanded clays and shales).

The densities of concrete vary enormously from 1 200 kg/m^3 to 2 000 kg/m^3 for lightweight concrete to between 2 000 kg/m^3 to 2 900 kg/m^3 for normal weight and high strength concrete. The density of concrete changes little with temperature, with the exception of limestone aggregates whose density reduces at temperatures exceeding 800 °C.

All concretes with free moisture show an increase in specific heat at around 100 °C as the free moisture evaporates. Further, chemically combined water is lost at temperatures up to 450 °C. The net effect of temperature on the thermal conductivity of concrete during heating is a complex interaction between the conductivity of water, air (porosity), the cement paste and the aggregate.

The thermal conductivity of lightweight concrete is significantly less than either siliceous or calcareous concretes, meaning that the thickness of concrete slabs can be significantly reduced whilst still achieving insulation criteria on transmission of heat from the fire side to the non-fire side of a floor. This also has the additional benefit of a reduction of the dead load.

In general concrete is a good insulator, and this is important in providing protection to the steel reinforcement. The thickness of cover to the steel reinforcement for either loadbearing or non-loadbearing is specified to ensure steel temperatures do not exceed critical levels during the intended fire duration. This also depends upon the type of steel reinforcement, e.g. hot rolled, cold formed and high tensile pre-stressed wires.

A major disadvantage to concrete is the effect of spalling, in which surface material is lost. Spalling can occur during both heating and cooling and depends upon the moisture content, the type of aggregate and the heating regime. The thickness of cover specified in the codes is designed to prevent failure of structural elements due to spalling. However, for high-strength concrete additional requirements can be specified, such as the inclusion of polypropylene fibres, which are designed to minimize the build-up of vapour pressure from moisture that can give rise to explosive spalling. Concrete elements can be fire protected using conventional lightweight materials to either reduce the propensity to spalling, or to make up for deficiencies in the thickness of cover to the reinforcement.

10.2.2 Empirical data based upon fire test results

Recent fire tests conducted by BRE (FRS), provide further data on the temperatures attained by structural elements under various conditions:

a) a natural fire test on reinforced high-strength concrete on the concrete building at the BRE test laboratory at Cardington [19];

b) fire resistance tests to evaluate the effect of polypropylene fibres on the performance of high strength columns [22];

c) natural fire tests on pre-cast hollow core slabs [23];

d) report by the Comité Euro-International du Béton [24].

10.2.3 Simplistic calculation of the temperature response of concrete

One of the earliest UK studies on the temperature profiles developed in concrete elements during the standard fire test was presented in a National Building Studies report [25]. The data are presented as post-fire investigation studies on concrete structures in real fires, as well as laboratory studies to measure temperature contours during the heating of concrete samples in the BS 476:1932 fire resistance test. Figure 12a) and Figure 12b) are extracts from the National Building Studies report on temperature profiles in concrete and illustrate the temperatures measured at various depths within a specimen from which temperature contours were established.

Figure 12a) **Time-temperature curves, at depths shown from surface for 1:2:4 Portland cement concrete with ham river sand and gravel aggregate – heated 2 hours**

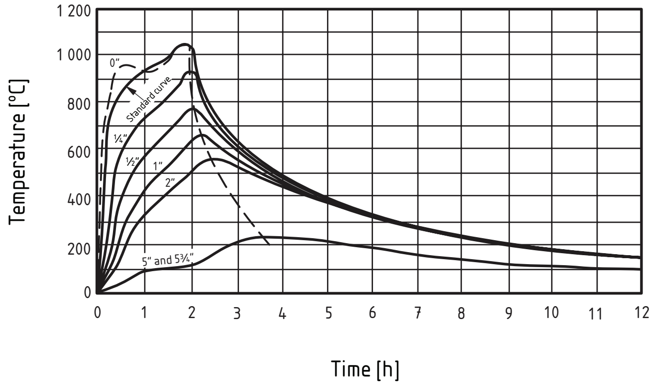
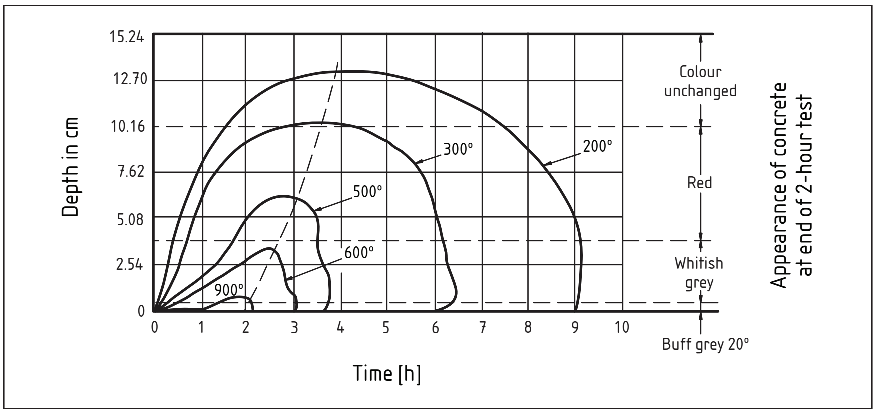


Figure 12b) **Time-isotherms and colour changes for 1:2:4 Portland cement concrete with ham river sand and gravel aggregate – heated 2 hours**



Information on the development of temperature within concrete members exposed to the standardized fire conditions is given in equations 45 and 46 (see BS 476-20:1987). The information is presented as a series of temperature contours and profiles through the cross-section of the heated member. Further data are provided in a report by the Comité Euro-International du Béton [24].

Wickström [26] proposed a relatively straightforward method for calculating the temperature profile within concrete members when exposed to the standardized fire (see BS 476-20) or to real (parametric) fire conditions (see **10.5.3**).

The temperature rise (T_x) at any depth beneath the surface of a concrete member heated to a temperature (T_s) by exposure to a gas atmosphere temperature (T_g) is given by:

$$T_x = n_x T_s \tag{45}$$

and

$$T_s = n_s T_g \tag{46}$$

where:

n_x, n_s are functions of time (t).

For convenience, time can be scaled to account for the variation in surface thermal properties between the concrete being considered and a nominal standard mix.

$$t_s = (\gamma / \gamma_i)t \tag{47}$$

where:

t_s is the scaled time (hours);

γ = $\sqrt{\Gamma}$;

Γ is the compartment time factor ($m^{5/2}$ K/$s^{½}$ W);

γ_i = $\sqrt{b / 1550}$;

$$\sqrt{k_c \rho_c c_c}\,\Gamma = \left(\frac{(A_w \sqrt{h_w}) / A_t}{b}\right)^2 \left(\frac{1\,160}{0.04}\right)^2 \tag{48}$$

When predicting the response of normal weight concrete to the standard BS 476-20:1987 heating regime, the scaling of time is unnecessary and t_s can be set to equal t.

The ratio between the fire's temperature and the surface temperature of the concrete is given by:

$$n_c = 1 - 0.0616 t_s^{-0.88} \quad (49)$$

where:

t_s is the scaled time in hours (see equation 47).

The ratio n_x between the surface temperature and the temperature at a depth x beneath the surface is given by:

$$n_x = 0.18 \ln(U_x) - 0.81 \quad (50)$$

where:

$$U_x = \frac{K_c}{4.17 \times 10^{-7}} - \frac{t_s}{x^2} \quad (51)$$

where:

K_c is the thermal diffusivity of concrete (m^2/s);

x is the depth (m).

Equation 49 applies to concrete for which conductivity is assumed to reduce linearly from approximately 1.25 W/mK to 0.5 W/mK between 100 °C and 1 200 °C. For the relevant material properties of concrete see **10.2.1**. Equations 45 to 51 can be simplified for applications considering the temperature development in normal weight concrete heated under conditions specified in BS 476-20:1987. In this case the temperature (T_x) at a depth x metres beneath the surface at time t hours is given by:

$$T_x = 345 \log t(480+1)\left(1 - 0.0616 t^{-0.88}\right)\left(0.18 \ln \frac{t}{x^2} - 0.18\right) \quad (52)$$

The empirical method can be applied to concrete members heated on parallel faces simultaneously, whereby n_x is simply the superimposed total of the n_x values calculated for each separate face. The method also accommodates heat flow at square corners, again through superimposition of the contributions from the orthogonal faces n_x and n_y as follows:

$$T_{xy} = \{n_s(n_x + n_y - 2n_x n_y) + n_x n_y\} T_s \quad (53)$$

BS EN 1991-1-2 provides a series of isotherms through reinforced concrete members based on the following thermal properties:

(i) specific heat given in Annex A;

(ii) moisture content of 1.5%;

(iii) thermal conductivity at the lower limit described in Annex A;

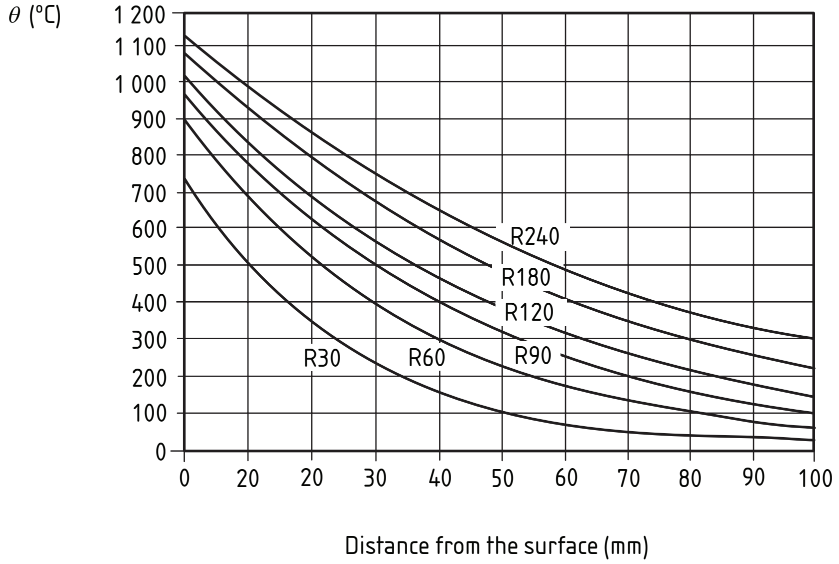
NOTE For moisture contents greater than 1.5%, the specified temperature profiles are conservative.

(iv) an assumed emissivity for concrete of 0.7.

The temperatures at various distances from the surface of concrete slabs exposed on one side as a function of fire resistance period are given in Figure 13.

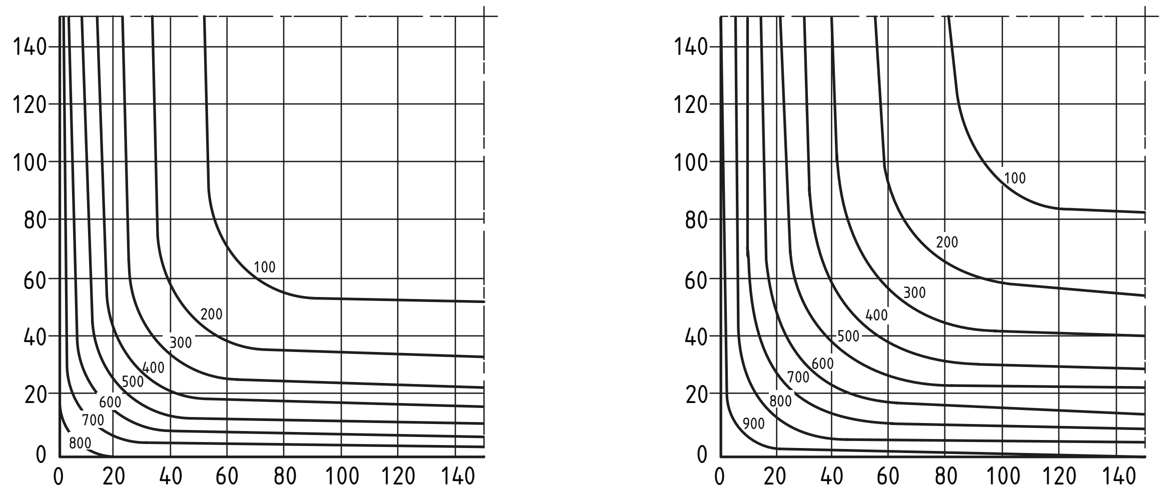
These temperatures can also be represented as a series of isotherms for different column sizes as illustrated in Figure 14.

Figure 13 **Temperature distribution in slabs exposed to the standard fire on one side**



NOTE R30, R60 etc. are fire resistance classes with respect to the loadbearing capacity criterion R, where the numbers are the periods of fire resistance, in minutes, in a standard fire test.

Figure 14 **Temperature profiles at distances from the surface (mm) for a 300 mm × 300 mm concrete column for various fire resistance periods**

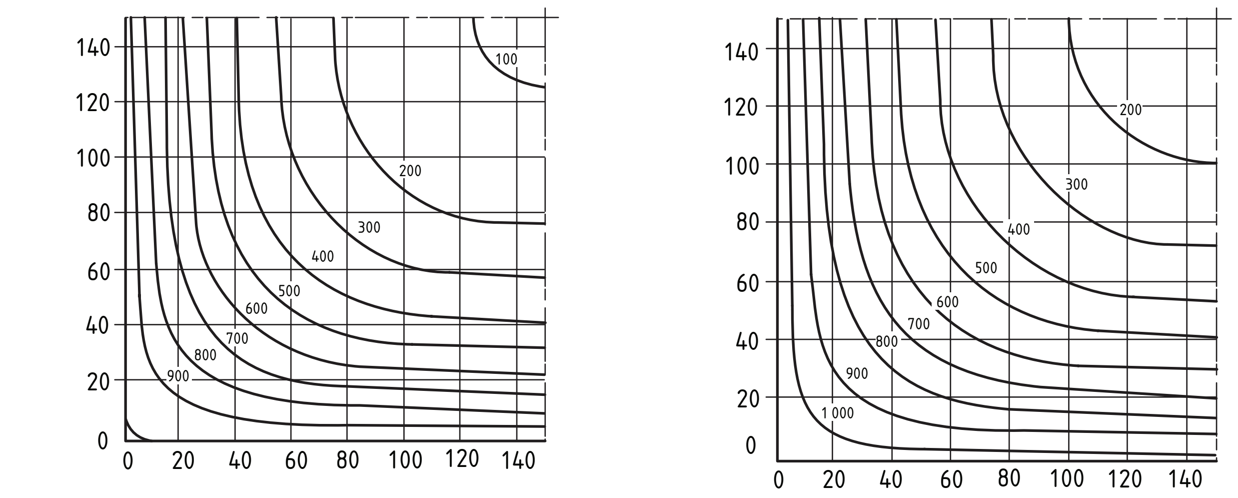


a) Fire resistance = 30 min

b) Fire resistance = 60 min

NOTE The axes show distance from the corner in millimetres. The curves are isotherms for different temperatures in degrees Celsius.

Figure 14 **Temperature profiles at distances from the surface (mm) for a 300 mm × 300 mm concrete column for various fire resistance periods** *(continued)*



c) Fire resistance = 90 min

d) Fire resistance = 120 min

NOTE The axes show distance from the corner in millimetres. The curves are isotherms for different temperatures in degrees Celsius.

10.2.4 Fire protection for concrete

Due to the good insulation characteristics of concrete structures fire protection is not normally required for structural elements. Usually the main requirement is to ensure there is sufficient cover to the steel reinforcement so that it remains below critical temperatures. However, fire protection can be applied to high strength members to prevent spalling or to make up for deficiencies in the concrete cover.

In BS 8110 where plaster, except Gypsum, or sprayed mineral fibre is used, the thermal insulation can be assumed to be equivalent to the same thickness of concrete, and therefore can be used to make up deficiencies in the cover thickness.

However, by carrying out calculations the thickness of other types of insulation can be substituted, providing they have been evaluated for their adhesion to the substrate.

10.3 Thermal response of iron and steel members

10.3.1 General

The thermal response of carbon steels is very similar at elevated temperatures, and small changes in chemical composition have little effect on their heat transfer characteristics.

Wrought and cast iron were the primary construction metals used in structural frames of buildings during the 18th and 19th centuries and, although they are no longer manufactured for this purpose, they frequently have to be considered in the refurbishment and conversion of Victorian buildings. The thermal properties of cast and wrought iron at elevated temperatures are not well established, however, for temperatures up to 600 °C (which are unlikely to be exceeded in design) those given for carbon steels can be adopted.

Stainless steels are available in a wide range of compositions, broadly divided into five groups according to their metallurgical structure. The majority of stainless steels used in building construction are austenitic, providing good corrosion characteristics that enable them to be used without any protective treatments. Austenitic stainless steels are highly alloyed with chromium and nickel, and these impart noticeably different thermal properties to conventional structural (carbon) steels.

During heating, carbon steels experience a change in magnetic domain at around 730 °C (Curie point), as the material changes from ferromagnetic to paramagnetic. This is an endothermic action resulting in a peak in specific heat. Austenitic stainless steel does not go through this change, with specific heat rising slowly with temperature.

Between 720 °C and 860 °C carbon steels go through a phase change from ferrite to austenite. This results in a change in the relationship between thermal elongation and temperature, which slows down until the phase change is completed. The rate of expansion of austenitic stainless steel is greater than carbon steels and, since it is already austenitic, carbon steels do not experience any phase changes.

The thermal conductivity of carbon steels decreases with temperature until around 800 °C and then remains almost constant until 1 200 °C. In contrast, the thermal conductivity of stainless steel increases linearly with temperature and, until around 1 000 °C, remains lower than carbon steels.

The density of carbon steels remains constant at 7 850 kg/m^3, whereas stainless steel has a density of 7 900 kg/m^3 at ambient temperature, reducing to 7 450 kg/m^3 at 1 000 °C.

10.3.2 Empirical data based upon fire test results

The temperature development in unprotected (bare) steelwork exposed to BS 476-20 and BS 476-21 fire tests is documented by Kirby and Wainman [27] for a range of steel sections sizes and configurations. The data are supplemented by computer simulations reported in Wainman et al [28].

A number of natural fire tests have been conducted by British Steel (Corus) using either wood cribs, a combination of wood and plastic cribs, and real furniture. These were carried out at BRE Cardington in either purpose-built compartments or on an 8-storey steel frame building. In each test programme the temperatures of both unprotected and protected steel members were extensively monitored ([14] and [29]). In addition, FRS (BRE) also conducted two fire tests on the 8-storey steel frame building (see [3]).

10.3.3 Simplistic calculations of the temperature response of steel members

10.3.3.1 Unprotected steel

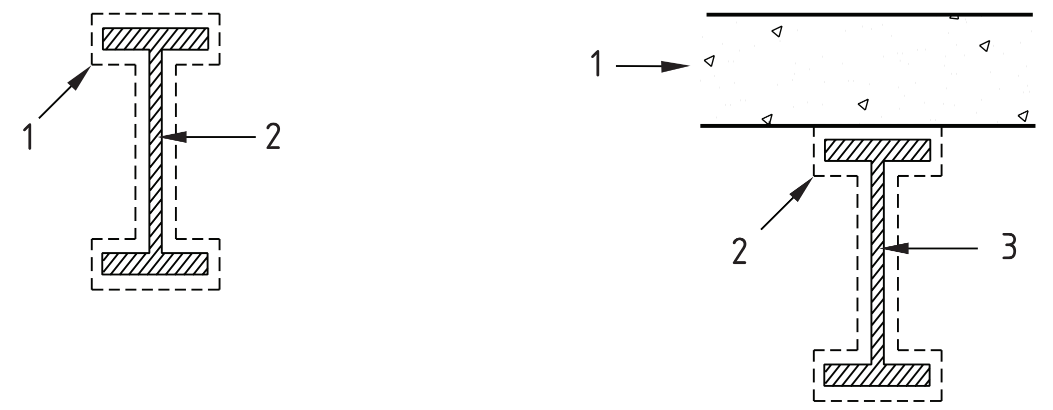
10.3.3.1.1 General

The temperature rise in a metal member exposed to a fire is largely determined by the ratio between its heated perimeter (H_p) and its cross-sectional area (A), sometimes referred to as the section factor. The parameters A_m/V in place of H_p/A are becoming increasingly common. The units of the section factor are m^{-1} and structural steel members in buildings typically have values in the range of 50 m^{-1}

to 250 m^{-1}. The larger the section factor, the more rapidly a metal member is expected to increase in temperature. Conversely, metal members with a small section factor have a slow rate of temperature rise and, in some instances, have sufficiently large thermal capacity so as to not require any additional fire protection.

The section factor, by definition, requires knowledge of the geometry and configuration of the member used in the building. This is illustrated in Figure 15 in which a steel member 254 mm × 102 mm × 28 kg/m universal beam (UB) attracts different section factors as a result of its configuration/extent of exposure to fire.

Figure 15 **Calculation of section factors**



a) Exposed stanchion H_p/A = 250 m^{-1}

Key

1 H_p

2 254 mm × 102 mm × 28 kg/m UB

b) Partially exposed beam H_p/A = 230 m^{-1}

Key

1 Concrete slab

2 H_p

3 254 mm × 102 mm × 28 kg/m UB

The determination of H_p for various configurations of steel members is given in Figure A.16 and Figure A.17.

The section factors, H_p/A (A_m/V), associated with many common steel members, are published by the Association for Specialist Fire Protection (ASFP) [30] and are also available from Corus [31].

Empirical calculations on the temperatures attained by unprotected steel members are reported in Wainman et al [28].

10.3.3.1.2 Temperature rise of unprotected steel

The mean temperature rise, $\Delta\theta_{a,t}$, of an unprotected steel beam during exposure to fire within an enclosure over a time interval Δt is given in BS EN 1993-1-2 by the relationship:

$$\Delta\theta_{a,t} = k_{sh} \frac{\dot{h}_{net,d}}{\rho_a C_a} (A_m / V) \Delta t \quad (54)$$

where:

$\Delta\theta$ is a temperature increment (K);

ρ_a is the steel density (kg/m^3);

C_a is the specific heat capacity of steel (J/kgK);

A_m/V is the section factor (m^{-1});

$\dot{h}_{net,d}$ is the net incident heat flux per unit area (W/m^2);

Δt is the time interval (s) – recommended maximum value 5 seconds;

k_{sh} is a shadow factor.

For I sections the shadow factor can be determined as follows:

$$k_{sh} = 0.9(A_m / V)_b / (A_m / V) \tag{55}$$

where:

$(A_m/V)_b$ is the box value of the section factor.

The shadow factor does not apply to sections with convex shapes, such as hollow sections.

The relevant thermal properties of metallic materials are documented in Annex A.

For periods greater than 60 minutes it can be conservatively assumed that the steel member is at the same temperature as the furnace temperature.

An improved prediction of the temperature response of steel members within an enclosure exposed to heating regimes, including BS 476-20:1987, is given by:

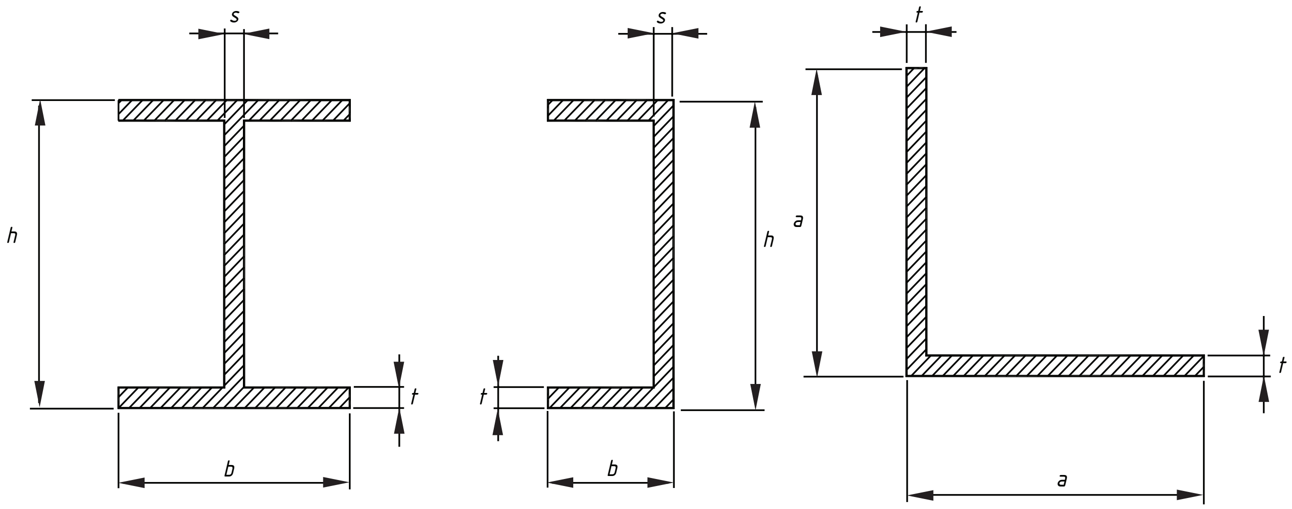
$$\Delta\theta_{a,t} = \frac{\dot{h}_{net,d}}{\rho_a C_a}(EF)\Delta t \tag{56}$$

The parameter *EF* is the element factor. Whilst conceptually similar to the section factor, the element factor relates only to the critical element of the steel member being considered, e.g. the web or the flange. Examples of calculations of the element factor are given in Table 8 and Figure 16.

Table 8 **Calculation of element factors (*EF*)**

Member	**Element factor**
Beams, channels, columns	$EF_{flange} = \frac{2(b+t)-s}{bt}$ $EF_{web} = \frac{2(h-2t)}{(h-2t)s} = \frac{2}{s}$
Angles	$EF_{leg} = \frac{2a+t}{at}$
Circular hollow sections	$EF_{tube} = \frac{1}{\text{thickness}}$

Figure 16 **Calculation of element factors**



a) Beams, channels and columns

b) Angles

10.3.3.2 Simplistic calculations of the temperature response of protected structural steel members

10.3.3.2.1 General

Where the temperature attained by an unprotected metal member during a fire could result in the loadbearing capacity being exceeded, protection is usually necessary to limit the temperature rise to an acceptable level. Typical forms of protection include:

a) profiled encapsulation with non-reactive insulating materials (see **10.3.3.2.2**);

b) boxed encasement with insulating boards which can include multi-layers and air spaces;

c) profiled encapsulation with intumescent coatings;

d) in-filling with concrete or blockwork;

e) in-filling with water;

f) active cooling systems.

NOTE Items d) to f) are discussed in Annex C.

10.3.3.2.2 Profiled or boxed protection with passive (non-reactive) insulating material

Protection of metalwork with insulating materials can be in the form of profiled or boxed systems. The thickness of protection required to provide specific levels of fire resistance is derived by means of an empirical relationship based upon standard furnace tests on both loaded and unloaded members.

DD ENV 13381-4 describes the assessment methods for determining the protection requirements for structural steel members to meet specific levels of fire resistance. These include:

a) differential equation (variable λ);

b) differential equation (constant λ);

c) regression analysis;

d) graphical analysis.

Any of the methods of analysis listed in a) to d) above can be adopted, although the experimental data should meet certain acceptability criteria.

All fire protection manufacturers who wish to market their products in Europe have to go through this type of test programme, and subsequent analysis to provide specifiers with information on the thickness requirements as a function of H_p/A (A/V), degree of exposure and fire resistance period. Some manufacturers also provide data based upon limiting temperature criteria over the steel temperature range 350 °C to 750 °C.

NOTE Information of this type is also published by the Association for Specialist Fire Protection Ltd [30].

The method of calculating the section factor varies according to the type of insulation (box encasement or profile encapsulation). The same method applies for both profile protection and unprotected steel members (see Figure A.16). However, for box encasement the section factor is significantly reduced since H_p is now taken as the inside of the fire protection system. This reflects the reduced exposure condition to radiated heat. Therefore, for the same thickness and type of insulation material, a steel element protected with a board system performs better than a steel element protected with a profile system.

NOTE Figure A.16 illustrates how the parameter H_p is determined.

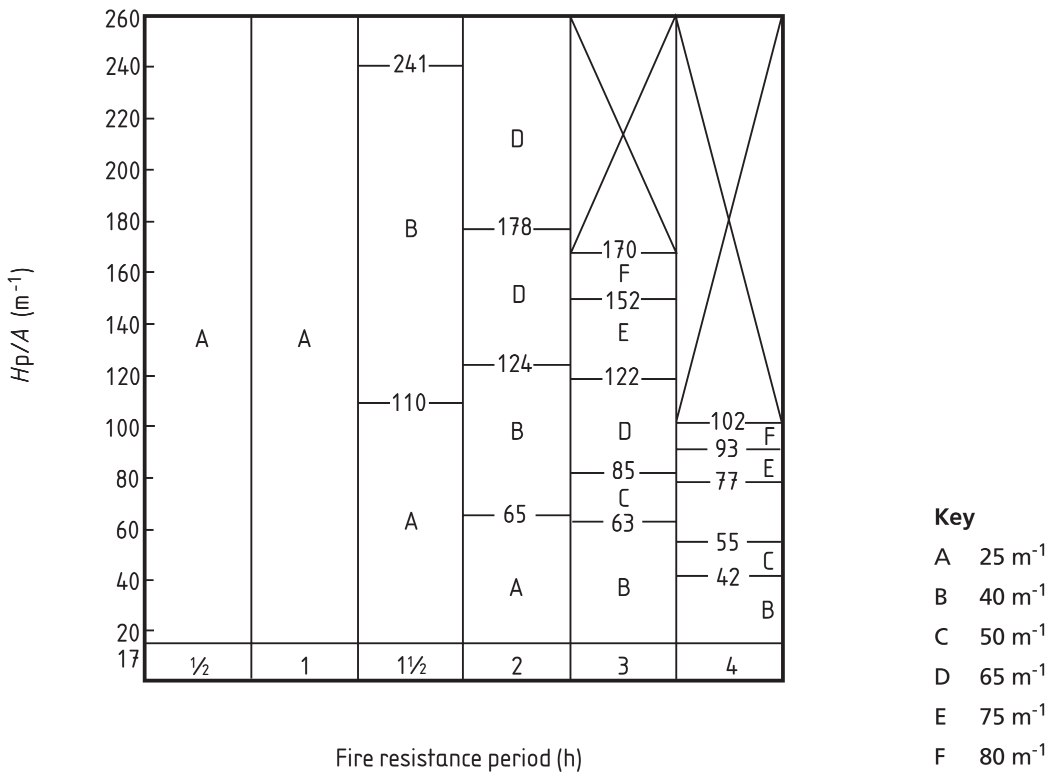
Table 9 illustrates the relationship between section factor and protection thickness for different fire resistance periods for a spray-applied system.

Table 9 **Typical set of coating thicknesses for a profile spray-applied protection system**

H_p/A (m^{-1}) up to:	Dry thickness in mm to provide fire resistance of:					
	½ h	1 h	1½ h	2 h	3 h	4 h
30	10	10	10	11	18	25
50	10	10	10	16	26	36
70	10	10	14	20	32	44
90	10	10	16	23	37	51
110	10	10	18	25	40	56
130	10	11	19	27	43	60
150	10	11	20	29	46	63
170	10	12	21	30	48	66
190	10	12	22	31	50	69
210	10	13	22	32	52	71
230	10	13	23	33	53	73
250	10	13	24	34	54	75
270	10	14	24	34	55	76
290	10	14	24	35	56	78
310	10	14	25	36	57	79

Figure 17 illustrates an example of the relationship between section factor and protection thickness for different fire resistance periods for a box encasement system.

Figure 17 **Typical set of board thicknesses for a box encasement fire protection system**



10.3.3.2.3 Profiled protection with passive (reactive) insulating material

The protection of metalwork with thin coatings that intumesce or expand on exposure to heat is a convenient way of maintaining an aesthetic form whilst providing insulation from the effects of fire. The required thickness of coating, specified as a dry film thickness (DFT), is a function of the section factor of the member, the fire resistance rating required and the fire's critical temperature.

Typically, manufacturers of intumescent coatings can advise on appropriate DFTs based on the results of fire resistance tests and expert assessments. For structural steel, guidance is also given in the ASFP publication on fire protection to structural steel in buildings [30]. The designer should recognize that, for loadbearing capacity, many DFT values are based on recommended limiting temperatures of 550 °C and 620 °C for columns and beams respectively. In fact, detailed analysis might conclude that limiting temperatures in excess of these values could be appropriate (see Clause **12**). This might permit rationalization of the DFT values or the achievement of increased fire resistance ratings. For other purposes, such as control of expansion or distortion, other limiting temperatures are applicable.

The methods of test and assessment for intumescent coatings are the same as those given in DD ENV 13381-4 and are described in BS EN 13381-8.

The designer should be aware of the need for proper preparation, priming and sealing of intumescent coatings and should take note of the quoted DFTs. Figures quoted by manufacturers vary according to whether they include primer and top sealing coats. Some coatings are susceptible to damage from moisture or dampness during application. Furthermore, the designer should establish a method for confirming the final on-site DFT thickness, as the insulation properties are sensitive to relatively small changes such as 0.1 mm in coating thickness. The designer should also be satisfied that the intumescent coating system is suitable for the application and environmental conditions. The stickability of the coating should be assured, e.g. by a paint primer being applied prior to the coating, given the extent of deflection expected. This is typically evaluated as part of the fire resistance test on a loaded beam. For maintenance of intumescent coatings it is imperative that any subsequent decoration is compatible with the fire protection system.

10.3.3.2.4 Special consideration for protecting cellular and castellated beams

Steel beams are available with fabricated openings of various shapes and sizes that enable greater depths to be achieved than the original section. Cellular beams can also be fabricated from steel plate.

The rules governing the determination of the section factor for protected (all types) solid steel members do not apply to cellular and castellated sections and therefore the section factor for these should be calculated as follows (see [30] and [32]):

$$\text{Section factor } (\mathrm{m}^{-1}) = 1400/t \qquad (57)$$

where:

t is the thickness (mm) of the lower steel web as formed from rolled or fabricated plate.

For active fire protection (intumescent coatings), the size of the post between the holes has a major influence on the structural performance of the beam (see [30] and [32]). These examples are limited to ceullar beams with web posts not less than 30% and circular holes up to 80% of the section depth. Several steel fabricators have developed software enabling cellular beams with other types of hole arrangements to be suitably fire protected.

The designer could have difficulty in obtaining reliable thermal properties for protection materials, particularly those whose properties are temperature-dependent. It is possible to back-calculate the thermal properties from fire test results to determine “effective values”. These values should only be viewed as valid in the context of particular calculation methods and should not be regarded as physically meaningful. It can be useful to refer to the manufacturers of the fire protection material and adopt protection solutions that have been validated through fire testing and empirical assessment.

10.3.3.2.5 Calculation of the temperature rise of protected steel

The passage of heat through a thin, non-reactive fire protective material in contact with a metal section can be calculated from first principles using the following relationship:

$$\Delta T_\mathrm{m} = \frac{1}{\rho_\mathrm{m} C_\mathrm{m}} \left(\frac{H_\mathrm{p}}{A} \right) \frac{k_i}{d_i} (T_\mathrm{g} - T_\mathrm{m}) \Delta t \qquad (58)$$

where:

ΔT_m is a temperature increment of metal (K);

k_i is the conductivity of insulating material (W/mK);

d_i is the thickness of insulating material (m);

T_m is the temperature of metal (K);

T_g is the fire gas temperature (K);

ρ_m is the density of metal (kg/m^3);

C_m is the specific heat capacity of metal (J/kgK).

Equation 58 ignores the potential for heat to be stored in the insulating coating itself, as might occur with thicker protective coatings. In such instances, the temperature rise is more accurately given by:

$$\Delta T_m = \left[\frac{\lambda_i / d_i}{C_m \rho_m}\frac{A_i}{V_i}\left(\frac{1}{1+\Phi/3}\right)(T_g - T_m)\right] - \left[(e^{\Phi/10} - 1)\Delta T_g\right] \quad (59)$$

where:

$$\Phi = \left(\frac{C_i \rho_i}{C_m \rho_m}\right) d_i \left(\frac{A_i}{V_i}\right) \quad (60)$$

C_i is the specific heat capacity of insulation (J/kgK);

ρ_i is the density of insulation (kg/m^3);

d_i is the thickness of insulation layer i (m);

A_i is the cross section area of an insulated metal element (m^2);

V_i is the volume per unit length of an insulated element (m^3).

Equations 58 to 60 do not apply to intumescent coatings and alternative methods are being developed. However, it is possible by back-calculating to derive a factor that describes the effective insulation characteristics, though this alters dramatically as the material changes its physical and thermal properties.

Equation 59 can be simplified to predict the thickness of protective coating (d_i) necessary to achieve a defined period of fire resistance on exposure to the standard heating regime for different failure temperatures as specified in BS 476-20 for steel or under real fire/characteristic exposure conditions.

Much of the calculation of temperature rise requires some knowledge of the thermal properties of the fire protection material. This can be complicated where the material contains moisture, as a dwell occurs at approximately 100 °C where heat is absorbed due to the latent heat of vaporization. The duration of the dwell time, t_d is approximated by:

$$t_d = \frac{P_w \rho_i d_i^2}{5k_i} \quad (61)$$

where:

t_d is the dwell time (min);

P_w is the % of moisture (by mass);

k_i is the thermal conductivity of an insulation layer.

NOTE k_i is not necessarily a constant above 100 °C.

In addition, heat can be absorbed or emitted as a result of chemical changes, e.g. release of water of hydration or burning off of binders. These effects are not covered by equation 61.

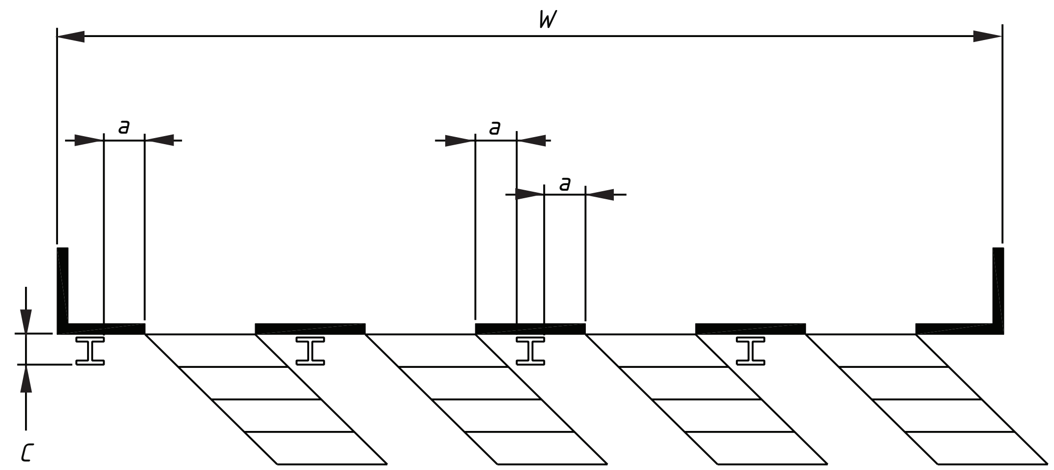
10.3.3.3 Temperatures attained by external members

The prediction of the temperature development in steelwork outside the enclosure but subject to flame impingement and/or radiation from openings is described by Law and O'Brien [18], and has subsequently been included in BS EN 1993-1-2. The techniques described vary in complexity. It might suffice for the designer to recognize that unprotected steelwork exposed to flames from openings cannot reach temperatures in excess of the temperatures of the flames themselves.

The thermal response of external members developed by Law and O'Brien is based upon steady state fire conditions. In many situations this can be too onerous as, e.g. when the fire load is low and steady state conditions might not be achieved or can only be sustained for a short period of time. More realistic results can be achieved by replacing the internal fire and gas temperatures with a full thermal history of the fire and calculating the flame temperatures through an iterative time step process. In addition, no account is made for the massivity (section factor) of the section which, for short duration fires, has a significant effect on the heating rates and the maximum steel temperatures attained.

Figure 18 and Table 10 show that careful positioning of external columns, with respect to the openings and the area outside a compartment wall, can avoid direct exposure to the flames issuing from the openings. These take into account deflection of the flames by 45° due to wind.

Figure 18 **Compartment parameters**



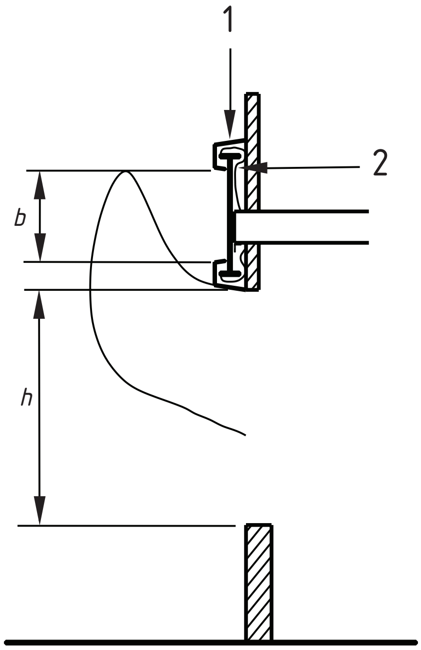
Plan: shows flame defection by wind, *A* = *a* or *C*, whichever is larger

Table 10 **Location of columns between windows to avoid direct flame impingement**
Dimensions in metres

Window height *h*	**Values of *A* for compartment width *W***			
	9	18	36	72
1	1.4	2.3	2.3	2.3
2	0.8	1.1	1.1	1.1
3	0.6	0.8	1.0	1.0
4	0.3	0.7	0.9	0.9
5	0.3	0.7	0.8	0.8

A similar analysis can be applied to the location and shielding required for spandrel beams above openings (see Figure 19 and Table 11).

Figure 19 **Spandrel beam with shielded flanges**



Key
1 Shielding to flanges
2 Spandrel beam
b Maximum exposed web height
h Window height

Table 11 **Spandrel beams**
Dimensions in metres

Window height *h*	**Maximum exposed web height** *b*
1.0	1.6
1.5	0.7
2.0	0.5
+ 2.0	0.4

The values are based upon steady state conditions, compartments containing a fire load density of 50 kg/m^2 and a critical steel temperature of 550 °C. For much lower fire loads, steady state burning conditions might only last for a short time or might not occur at all. It would therefore be more realistic to base the calculations on a history of compartment temperatures.

10.4 Thermal response of timber

10.4.1 General

There are numerous types of timber, varying in density according to the species and the environment in which they grow. In fire, however, they all behave in a predictable manner. Timber and wood-based products primarily consist of cellulose and lignin and, when exposed to heat, burn steadily with all exposed surfaces charring away at an empirically derived rate.

Large fissures in timber allow the heat to penetrate, and the additional exposed surface effectively increases the perimeter. The rate of combustion of timber products is dependent upon their density, moisture content and grain orientation, with timbers of high density generally burning at a slower rate than low density timbers. However, there are some exceptions to this and the diffusivity of the charcoal is a more accurate predictor. At all stages of exposure to heat timber shrinks, and shrinkage in the longitudinal direction is approximately 10% of that transverse to the grain orientation. The core of a timber structural element is insulated from heat which causes drying, but whereas longitudinal shrinkage is usually negligible in practice, shrinkage of the cross-section can be significant.

In structural elements composed of timber, the surface area exposed to fire in relation to its volume governs their performance in fire. Sharp corners, splits or fissures in the elements affect the surface-to-volume ratio. Therefore, glulam, laminated veneer lumber (LVL) and other structural timber composites can perform more consistently than solid sawn timber which can be prone to fissuring.

The type of adhesive in composite timbers has a major impact on fire performance. Urea, resorcinol and melamine adhesives generally perform better than epoxy-based adhesives.

The most important property of all timbers is the charring rate. The effective cross-section of the residual timber in fire, beneath the char, controls the structural performance of the member. Fixing an insulated board, such as plasterboard, alters the profile of the residual unburnt timber. The use of impregnated flame retardants can improve the surface spread of flame characteristics but the timber still chars, possibly at a faster rate than untreated timber. Similarly, intumescent paint or varnish only produces an ignition delay because once the wood below the surface produces steam, i.e. exceeds 100 °C, the protected layer is pushed-off from the surface.

The specific heat of timber is almost constant with temperature except at around 100 °C where a peak is observed as free moisture is driven off.

The thermal conductivity of the uncharred timber is influenced by moisture content and density, although the values used are usually apparent rather than actual. At around 500 °C the thermal conductivity increases significantly with temperature.

10.4.2 Empirical data based upon fire test results

The performance of joints is crucial, particularly steel plates and bolts which could be exposed to fire. For fire-resistance purposes these have to be buried within the timber elements and covered with timber plugs or covered with traditional fire protection systems, such as plasterboard linings (see BS EN 1995-1-2 and Hartl [33]).

Some limited data are also available from the Timber Research and Development Association (TRADA) [34] on fire tests carried out in real buildings.

10.4.3 Simplistic calculation of the temperature response of timber

The most important calculations are primarily concerned with establishing the depth of char, or unburnt timber, for any given type of exposure condition and fire resistance period. These form the basis for determining loadbearing capacity (see Clause **12**). For soft wood, the char line occurs at approximately 300 °C.

Calculations for the charring rates of timbers, both solid and composite, are given in Annex A.

Calculations on the depth of char are also given in Clause **12** as part of the procedures for determining the loadbearing capacity by either the "reduced cross section" or "reduced properties" methods.

The charring rate of glued laminated timber members can be treated in the same way as solid timber when any of the following adhesives are used:

- phenolic and aminoplastic resin;
- resorcinol formaldehyde;
- phenol formaldehyde;
- phenol-resorcinol formaldehyde;
- urea-formaldehyde;
- urea-melamine-formaldehyde.

The fire performance of solid timber members is well documented. The fire performance of separating elements based upon timber components, e.g. timber joisted floors and stud walls protected by a variety of lining materials, have been established empirically by fire resistance tests. Where the lining material is generic, the results of a number of such fire resistance tests have been analysed.

The contribution these proprietary linings make to the fire resistance of elements can be found in trade literature. These data have to be used with care because factors, such as load ratios and slenderness ratios, might not be obvious.

Proprietary linings can be treated as solid timber in terms of their charring rate. Complex "glulam" beams, where higher grade timbers are used at the extremities of the section with low grade timbers in the core, can char at different rates.

Impregnation with flame retardant salts has been shown to increase the charring rate of timber.

10.4.4 Simplistic calculations of the temperature response of protected timber

The carbonaceous char formed from timber is an insulating material itself and, provided the residual unburnt timber has sufficient cross-section to support the applied loads, passive protection using traditional materials is not required. However, for slender elements additional passive protection is required.

For structural elements consisting of composite multi-layered systems, such as floors and walls, the designer should choose a combination of constructional details to meet specific fire resistance requirements on loadbearing capacity, insulation and integrity. These include the size of the loadbearing member, type of insulating material (thickness, single or multiple layers) and fixing details. Rules involving the attribution of indices to each part of the protection are provided to represent the contribution of added layers of insulation to the fire performance of the construction.

BS EN 1995-1-2 adopts a quantitative approach to assessing the protection requirements for timber elements.

The objective of any protection system, either fixed or applied to timber elements, is to slow down or stall the commencement of charring. In BS EN 1995-1-2 the following are considered:

- the start of charring is delayed until time t_{ch};
- the potential for charring to begin prior to time of failure of the protection t_f but at a slower rate than the charring rate without protection;
- after the failure of the protection, the charring rate is increased above the charring rate for the unprotected timber until time t_a;
- at the time t_a when the charring depth equals either the charring depth of the same member without fire protection or 25 mm (whichever is less), the charring rate reverts to the charring rate of the timber.

In BS EN 1995-1-2 a set of nomograms forms the basis for calculating the charring rates for the following conditions:

- variation of charring depth with time when $t_{ch} = t_f$ and the charring depth at time t_a is at least 25 mm;
- t_{ch} t_f t_a variation of charring depth with time when $t_{ch} < t_f$.

10.5 Masonry

10.5.1 General

Masonry blocks and bricks are fired clay, brickearth or shale, autoclaved aerated concrete, dense or lightweight concrete and artificial stone. They can be solid, hollow or cellular and are bonded in a regular pattern using mortar, which can be a single or double leaf cavity construction.

The thermal properties of masonry are dependent upon the materials used and the type of mortar and whether they have rendered surfaces of mortar or plaster.

Once constructed, masonry retains a certain amount of free moisture which is driven off at around 100 °C. In some materials, at higher temperatures, chemically combined water is also lost. Both of these affect the specific heat and thermal conductivity.

Masonry is a good insulator and generally performs well in fire. However, in a fire masonry walls are usually exposed to heat from one side only, with the result that a temperature gradient is developed. This can generate thermal stresses which cause bowing of the wall towards the fire.

10.5.2 Empirical data based upon fire test results

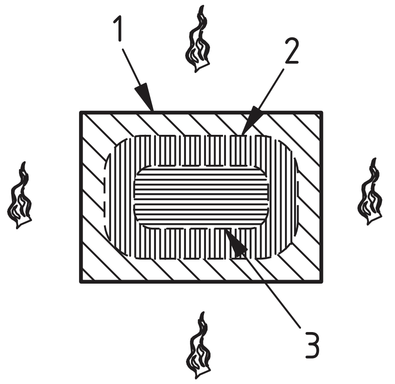
Masonry structures have been extensively tested in the standard fire resistance furnace and the majority of information is presented in the form of tables. Tabulated fire resistance periods are given in BS EN 1996-1-2.

NOTE For further information see [35].

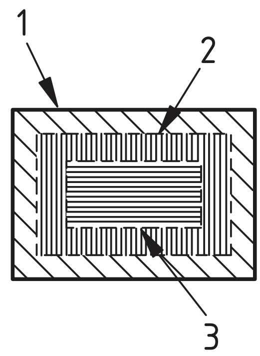
10.5.3 Simplistic calculations of the temperature response of masonry members

BS EN 1996-1-2 allows for the calculation of thermal distribution using two approaches as part of the process of establishing the structural performance of masonry constructions. These are illustrated in Figure 20.

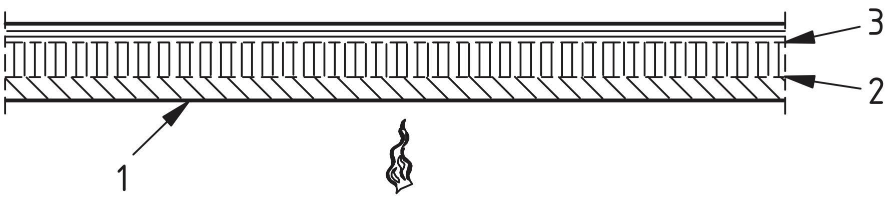
Figure 20 **Calculation methods for determining the temperature profiles though masonry elements**



a) Cross section of column exposed to fire with real isotherms



b) Cross section of column exposed to fire with idealized isotherms for simplified calculation



c) Separating wall cross-section

Key

1 Boundary of original cross section

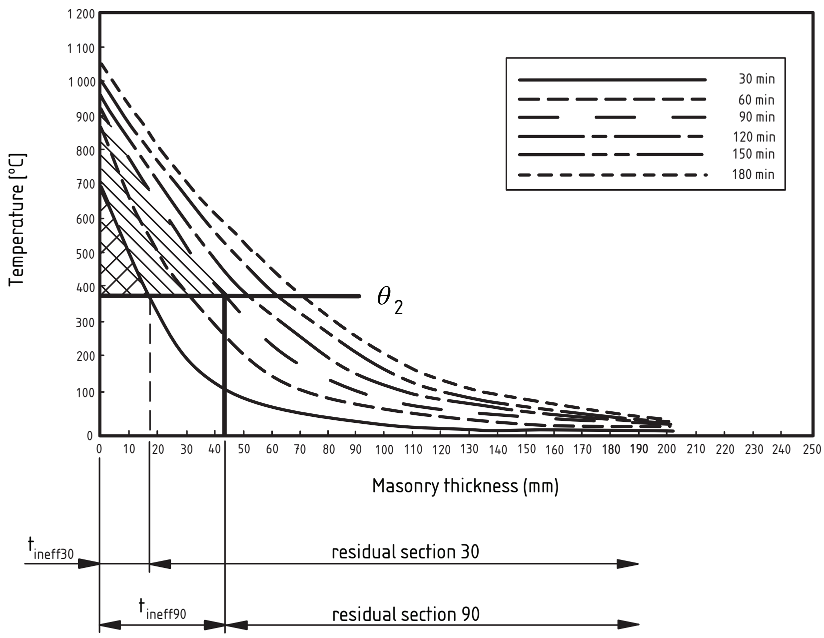
2 Isotherm for $\theta = \theta_2$

3 Isotherm for $\theta = \theta_1$

The methodology relies upon calculating the following temperature isotherms, up to 100 °C (θ_1), and between 100 °C and a temperature above which the material can be assumed to have no strength (θ_2).

Isotherms are given for various types of masonry as a function of thickness, for example Figure 21 which is an example for autoclaved concrete masonry.

Figure 21 **Temperature gradient through autoclaved concrete masonry with a density of 400 kg/m^3 to 800 kg/m^3**



Key

$t_{ineff30}$ thickness of wall that has become ineffective in 30 min

$t_{ineff90}$ thickness of wall that has become ineffective in 90 min

θ_2 temperature above which masonry is structurally ineffective

10.5.4 Fire protection to masonry elements

It is usually necessary to protect masonry structures from fire, however the masonry structures are often used to provide a decorative finish and, in such cases, can be considered part of the system for providing insulation performance.

10.6 Thermal response of aluminium

10.6.1 General

Aluminium is non-combustible, so aluminium structures do not burn. Aluminium alloys broadly belong to one of two basic groups:

a) non heat-treatable alloys including:
- EN AW – 3 000 (aluminium-manganese alloys);
- EN AW – 4 000 (aluminium-silicon alloys);
- EN AW – 5 000 (aluminium-magnesium alloys); and

b) heat-treatable alloys including:
- EN AW – 2 000 (aluminium-copper alloys);
- EN AW – 6 000 (aluminium-magnesium alloys);
- EN AW – 7 000 (aluminium-zinc-magnesium alloys).

Aluminium alloys are widely used for a range of products in the construction industry due to their lightness, ease of fabrication and good anti-corrosion qualities. However, they melt at around 590 °C to 650 °C. Despite their ability to reflect radiant heat (80% to 55% for weathered surfaces and 97% for polished surfaces), in fire situations where their loadbearing capacity needs to be retained they have to be protected with established propriety fire insulating materials.

In several applications the low thermal mass and good thermal conductivity of aluminium alloys are an advantage in fire, e.g. in glazing systems where the temperature differential between the frame and the glass pane is reduced compared to timber frames, thereby reducing thermal stresses and the likelihood of early failure.

10.6.2 Empirical data based upon fire test results

There is little information on the performance of aluminium loadbearing members in fire and, therefore, reference should be made to individual manufacturers.

10.6.3 Simplistic calculations of the temperature response of aluminium alloy members

Calculations of the thermal response of aluminium in fire are presented in BS EN 1991-1-2. These follow the same methodologies as for structural steel in the treatment of heat transfer to unprotected and protected members and structural members located external to the building façade.

Apart from inputting different thermal properties into the calculations, one slight difference is the calculation rules for establishing the section factor for various profiles where there are grooves. They state that grooves less than 20 mm wide should be discounted in deriving the surface area exposed to fire.

10.7 Thermal response of glass

10.7.1 General

As a non-crystalline solid which is, in effect, a super-cooled liquid, glass cannot plastically deform. It makes the transition from elastic to plastic once the temperature has risen so that it becomes viscous.

Some glasses have a high coefficient of linear expansion which causes large thermal stresses to be generated and causes failure. For fire resistance, glasses of low thermal expansion or capable of resisting high thermal stresses are used. The latter can be achieved by a toughening/tempering heat treatment process. More recently a process of chemical strengthening has been developed in which sodium ions are replaced by potassium ions to create residual compressive stresses in the surface.

Some glasses, such as the ceramics, have almost zero expansion but cannot be used in construction without some form of laminating.

There is a wide range of glass products with properties, such as heat-resistance, impact-resistance, insulation, low expansion, corrosion resistance and fire performance. These can be monolithic or laminates of various types.

The structural use of glass can be broadly divided into two functions:

- vertical, e.g. façades and partitions;
- horizontal as loadbearing floors.

There are two types of façade constructions where glass forms the structural element:

a) façades of glass panels hung from a supporting structure; and
b) façades of glass where panes are fixed together without the use of frames.

Though designed to support loads from gravity and environmental influences, façade constructions are not considered loadbearing (see Clause **12**).

For horizontal loadbearing floors there are several forms of construction, including the following.

1) Single sheet of float glass 20 mm to 35 mm thick with a fire resistant intumescent glass pane and a laminated glass pane underneath. The thick float glass layer is the "wearing" layer which provides the required loadbearing capacity, thickness being dependent on span and the live load.
2) Toughened laminated glass sheets, each 6 mm thick, with a number of float glass layers beneath. Fire-resisting glass floor is constructed from toughened laminated glass sheets. The sheets are bonded together by means of a transparent plastic foil comprising a thin polyvinyl butyral (PVB) sheet. The PVB-foil is sandwiched between the glass layers and the composite is cured in an oven at temperatures up to ≈ 120 °C to finish the bonding process. The float glass layers are sacrificial and crack after the heating has started.

10.7.2 Empirical data based upon fire test results

Several glass manufacturers provide technical literature on fire resistance performance of loaded floor systems. Available calculations on the use of loadbearing glass flooring systems are primarily concerned with product specification rather than with thermal calculations.

10.8 Thermal response of plastics

10.8.1 General

Plastic composites or laminates are increasingly an option for use in structural applications that could be subject to fire. The information available is very specific to the type of composite, the reinforcement and its volume fraction.

The subject of composite plastics is often regarded as a specialist area in terms of selection of materials for their intended purpose. Selection should always be carried out in consultation with the manufacturer/supplier.

10.8.2 Reinforcement

Reinforcement can be broadly divided into two aspects:

a) The reinforcement, where fibres are used to provide structural stiffness and strength to the composite so the choice of fibre type and material is determined by the properties required. The reinforcement type has to be compatible with the matrix for adhesion and interface stability.
b) The matrix, which provides the medium that transfers load to the fibre reinforcement and maintains the shape and orientation of the fibres with respect to the applied loads.

Reinforcement types include:

1) rovings: multi strands in which tension can be applied to control orientation and consolidation;
2) mats:
   i) chopped strand mat – non-woven planar material in which the strands are chopped into short lengths, evenly distributed and randomly orientated;
   ii) continuous filament – non-woven material in which the fibres are continuous and randomly swirled;
   iii) woven rovings – bi-directional reinforcement;
3) fabrics: plain, satin, twill (woven fabrics interlacing warp and weft yarns to give a variety of pattern types);
4) non-crimp fabrics: unidirectional fibre tows laid parallel to each other or held at precise, predetermined orientations;
5) prepegs: fibre reinforcements with resins already infiltrated but not fully cured.

Reinforcing materials include:

- glass fibre:
  - E glass has the highest strength;
  - C glass has good chemical resistance but is not as strong as E glass;
  - ECR glass is boron-free glass with similar properties to E glass;
- carbon fibres: a wide range of properties in strength and stiffness;
- aramid: organic fibres that include Kevlar®.

10.8.3 Matrix resins

The selection of the polymer resins for use in structural composites depends upon a number of factors, primarily compatibility with the reinforcement and the service conditions, of which temperature is one of the major issues. The common resins used are as follows:

- polyester resin: a general purpose thermosetting orthophthalic resin, which has a good combination of mechanical properties and moderate elevated temperature performance:
  - isophthalic acid (IPA);
  - bisphenol-A (BPA);
  - chlorendic;
- vinyl ester resin: thermosetting resin derived from the components of polyester and urethane resins:
  - bisphenol-A;
  - vovalic;
- modified acrylic resins: thermosetting resins which can have good flammability characteristics;
- phenolic resins: most suitable where heat is a primary consideration, such as fire resistance;
- epoxy resins: can provide good mechanical strength at elevated temperatures.

The thermal properties of thermoset polymers are not strongly dependent on temperature so, for heat transfer analysis, an average constant value can be assumed. However, their application is limited by the temperature at which the resin suffers a loss in stiffness. Significant creep occurs if the temperature is close to the heat distortion temperature and this largely dominates their use.

10.8.4 Empirical data based upon fire test results

Data on the behaviour of fibre composites is held by individual manufacturers.

10.8.5 Simplistic calculations of the temperature response of loadbearing composites

In fire, as the decomposition reaction progresses through the material thickness, the transport properties (e.g. heat conduction, charring) vary dynamically according to the local state of the resin. By limiting the heat transfer to one dimension and assuming the plastic components are intimately mixed and orientated in a plane perpendicular to the through thickness direction, the transport properties can be treated as a function of the constituent volume fractions. The proportion of fibres to the matrix is typically up to 40% of the total system but can be as high as 70%.

The amount that a solid polymer expands or contracts when heated invariably depends upon the nature of the polymer and the temperature reached during the fire.

The specific heat capacity of the fibre-reinforced polymer is determined using a weighted average of the form:

$$C_{com} = (C_f \rho_f V_f + C_x \rho_x V_x) / (\rho_f V_f + \rho_x V_x) \quad (62)$$

where:

C_{com} is the specific heat of the fibre reinforced polymer (J/kgK);

C_x is the specific heat of the matrix (J/kgK);

C_f is the specific heat of the fibres (J/kgK);

ρ_f is the density of the fibres (kg/m^3);

ρ_x is the density of the matrix (kg/m^3);

V_f is the fibre volume fraction of the composite;

V_x is the matrix volume fraction of the composite.

The thermal conductivity of a fibre-reinforced polymer in the through thickness direction is derived from the conductivity of the fibre and matrix polymer components using the following rule of mixtures approach:

$$1/k_{com} = V_f / k_f + V_x / k_x \quad (63)$$

11 Behaviour of separating elements in fire

11.1 General

Separating elements take two forms: those designed to provide fire separation for a particular duration and those not designed to do so. Non-fire-rated elements provide fire-separation for a limited period while they remain imperforate, as they are not normally subjected to furnace testing. This contribution should not be ignored as the reduction in the enclosure volume can sometimes produce a more rapid flashover. Where a number of such elements are present in a fire compartment their performance should be taken into account (see **11.5**).

Fire-separating barriers designed to resist fire obviously have to prevent fire spread for the duration specified in the strategy. With the exceptions of elements of cast *in situ* concrete walls and floors, most separating elements are of composite construction and their performance is therefore the sum of the interactions between the components. Even a blockwork wall is a composite of blocks and mortar, and the strength of the bond influences its fire behaviour.

Apart from the consideration of how structural members should be jointed, this Published Document has so far considered the fire behaviour of homogeneous elements of a single material. Such elements are relatively easy to model in order to predict their performance. Where the fire performance is based upon the interaction between components, e.g. framing members, boards, insulation and fixings, the behaviour is more difficult to predict. Empirically derived data, generated by the appropriate standard fire resistance test, are often the only available indicator of the likely performance of a composite construction. Depending upon the fire scenario chosen this might or might not be acceptable (see **11.4**).

The ability of a fire-resisting separating element to prevent the spread of fire is measured in terms of the duration for which it satisfies the loadbearing capacity (R), integrity (E) and insulation (I) criteria. In terms of the standard fire tests, these criteria are given fixed pass/fail performance levels to remove any subjectivity in the reporting of the results.

However, in a fire engineered solution plan, the risk and the appropriateness of these levels should be reviewed. These issues are discussed in Clause **8** and the analysis of the loadbearing capacity of structural elements is covered in Clause **12**. This clause primarily considers integrity and insulation performance, but takes into account fire-induced loads.

There are few validated computer models for predicting the integrity (E) of separating elements, or the insulation (I) in complex elements constructed from multiple components. Therefore, there is a greater reliance on standard fire resistance test data for predicting the performance of separating elements. However, fire resistance test generated data are not directly applicable to many fire engineered applications for reasons of size (most elements being larger in practice than in the test), or exposure conditions, which generally differ from those used in the standard furnace test. In the absence of valid models to predict the probable performance, an estimate can be achieved using the extended application process (see Annex D) as follows:

a) identify the critical exposure and constructional parameters for an element in a particular application;

b) on a parameter-by-parameter basis, establish the influence of a change in the size or a variation in the exposure on the performance, as appropriate.

This is not often conclusive and engineering judgement should still be made to "weight" the influences, some of which are conflicting, in order to make a reasonably accurate prediction of the fire response of the actual construction. This methodology should avoid any excessive under- or over-design of the elements, though there is always a margin of uncertainty.

Any fire strategy should demonstrate, in a written statement, that the issues raised in **11.1** have been addressed.

11.2 Behaviour of fire-resisting separating elements

11.2.1 General

The failure of an element to contain a fire is generally the result of one of two mechanisms.

a) Direct: flames and/or hot gases penetrate through the barrier to the protected face and provide the potential for ignition of the structure or contents on the protected side, resulting in a loss of tenability due to rapid temperature rise and actual flaming (integrity failure). The direct method invariably involves the development of gaps or fissures, through which fire gases can pass, and can develop as a result of differential movement, distortion, shrinkage, erosion or burn-through of the separating element. In the case of a separating element that incorporates combustible materials, the element itself can ignite on the unexposed face if the surface temperature exceeds spontaneous or pilot ignition temperature. This direct method of spread is the hardest mechanism to model because it invariably occurs as a result of significant composite action.

b) Indirect: the surface temperature of the protected face reaches a level that can induce ignition (or smouldering) of items in contact with the element by conduction (insulation failure), or of items in the vicinity by radiation, resulting in a slower loss of tenability. The indirect method is the mechanism most readily predictable as it occurs primarily as a result of heat conduction through the element under consideration. However, with some hollow constructions there is a convective heat transfer within the element. In order to establish the potential for ignition of items in contact with the elements it is sufficient to compute the surface temperature. For materials separated from the surface in which ignition occurs by radiation, both the emissivity and surface temperature of the elements need to be known.

Most organic-based or hygroscopic materials shrink when heated, causing a general loss of volume and, eventually, erosion. Where the material is thick it shrinks on the exposed face relative to the unexposed face, thereby inducing concave distortions. In contrast, metals, glasses and other non-hygroscopic materials expand and, if contained, produce buckling in the weaker plane and generally distort in a convex manner toward the fire.

During a fire most organic materials gradually erode resulting in a temperature increase on the unexposed face which is more directly proportional to the reduction in thickness. However, metals, glasses and ceramic based materials do not generally erode and the rise in temperature is a function of the density, mass and thickness.

Predicting the behaviour of some materials, e.g. glass (see **12.3.11**) and polymers (see Clause **10**) is made more difficult by their change in state, normally into a "plastic" condition. Many of these materials have well documented transition temperatures, but when only exposed on one face, the time at which they change state is related to the heat loss from the unexposed face, the mass and thickness. Physical testing is normally the only method by which the behaviour of such products can be characterized.

If the element is designed to resist fire spread, its ability to do so should be determined by one of the methods in Clause **10**. Most of the evidence readily available to support the performance of a common form of construction is determined by the standard tests. Much of this evidence is only available from private industry, as most of the linings, and many of the structural studs and joists are of proprietary construction. This evidence should be obtained from the relevant manufacturers or suppliers.

A problem associated with the direct use of this evidence relates to the fire resistance rating having been determined using a relatively small element (walls 3 m × 3 m, floors 4 m × 3 m) which is frequently far less than the size incorporated in the building. The use of such evidence in a prescriptively driven fire safety strategy (e.g. building regulations or code compliant case) presents few problems to the designer because the prescription takes the size change into account. However, if, in a fire engineered strategy, there is a need to consider the performance in larger and/or less restrained applications, the applicability of fire resistance test evidence to the as-built condition is not clear and is often overlooked.

Crucially, the designer should consider the impact of distortion in one element on adjacent elements. Distortion due to thermal restraint and other mechanical forces can produce gaps, particularly at three-dimensional junctions. These cannot be tested and can induce loads on adjacent non-loadbearing elements to the extent of seriously impairing their function. Loadbearing elements can be deemed to have satisfied their performance requirements with regard to protection for a particular period in the horizontal condition, even if they have deflected L/30. Deflections of this order can be extremely damaging to separating barriers when beams, for example, are located directly above or pass through these types of non-loadbearing elements. If these situations do arise it might be necessary to limit distortion to lower levels than the limits of loadbearing capacity permit [36].

Almost all separating elements are penetrated in practice for the purpose of access, light transmission, vision or the provision of services. It is important that the manner by which these are closed-off does not compromise the fire resistance that the solid structure provides.

11.2.2 Elements primarily composed of concrete or masonry

11.2.2.1 General

Concrete is used either in the form of cast *in situ* or blocks and slabs to form floors and walls.

Cast *in situ* construction can be either reinforcement, for both floors and walls, or pre-stressed in the case of floors. Some walls that are sufficiently thick might be un-reinforced, although this is unusual.

Stairs and balconies can be pre-cast but these are not considered in this Published Document.

11.2.2.2 General consideration of response of concrete elements

The initial response of concrete to heating is expansion. Until the temperatures are reached at which the cement bond starts to break down, the aggregate expands causing the exposed face to expand relative to the cooler main bulk of the material. This can induce the unusual effect of horizontal concrete separating elements "hogging" during the early stages of heating. The density of the concrete dictates the amount of heat that is conducted into the core of the material, hence the rate at which erosion takes place. The nature of the aggregate generally determines the density of concrete, with light-weight clay aggregate at the lower end of the range and gravel aggregates at the upper end. However, foamed concrete can be produced by the injection of air/gas to create artificially low densities. Quite weak, low-density concrete has very low thermal conductivity and generally exhibits high resistance to erosion during heating.

The influence that the thermal conductivity of concrete has on its mechanical response depends upon the nature of the construction. If the concrete element is used horizontally it normally incorporates reinforcing steel to compensate for its inherent weakness in tension. The ability of the reinforcement to carry either the self-weight or any applied loading is determined by its temperature, and this in turn is influenced by the amount of concrete cover between the strands or mesh and the fire.

Where the overall concrete thickness or cover to the reinforcement is insufficient to meet the required fire resistance, such deficiency can be compensated for by the use of non-combustible insulation such as fibrous and cementitious sprays, insulating boards, lightweight mortars and gypsum plasters. The required thickness is calculated from high temperature, non-steady state thermal conductivity data supported by evidence of "stickability" in respect of a concrete substrate.

Unreinforced elements, such as mass concrete walls, should remain stable until the concrete strength has deteriorated to a point where the loadbearing capacity has effectively been lost (see Table 12).

Table 12 **Recommended fire protection thickness to compensate for deficiencies in concrete thickness/reinforcement cover**

Protection product	Compensating thickness
Mortar/gypsum plaster	0.6 × concrete deficiency
Lightweight plaster	1.0 × concrete deficiency for up to 2 h
Sprayed lightweight insulation	2.0 × concrete deficiency for more than 2 h
Vermiculite slabs	1.0 × concrete deficiency for up to 2 h
Stonewall insulation slab	1.5 × concrete deficiency for more than 2 h

11.2.2.3 Integrity

Concrete has a high natural integrity, being a relatively homogenous substance. This is enhanced by the aggregate normally being held together by steel reinforcement. Being of significant thickness, a solid slab is unlikely to fissure during fire exposure unless it experiences large deformations. The behaviour of a solid concrete element normally differs to the elements abutting it so that the primary cause of an integrity failure could be differential movement between the concrete element and any adjacent or incorporated elements.

It is normal practice to recommend the thicknesses of concrete cover to the steel reinforcement, as in BS EN 1992-1-2, although this is related to the loadbearing capacity of the element rather than calculating the performance of "integrity only" elements. When using this as a measure of performance in predicting integrity behaviour, loadbearing capacity should be satisfied even when large distortions are achieved, up to L/30 for floors. This level of deflection could compromise integrity at junctions between elements, or even initiate the collapse of separating elements beneath. As with steel beams, if deflection is to be controlled, then higher levels of cover/protection are required.

Where apertures are formed in cast *in situ* constructions to permit the incorporation of stairways, doorways or windows, the exposed edges are prone to spalling (see **10.2** and Annex A). There are no traceable tests on concrete so formed, representing the fully developed fire or even the standard time/temperature curve. Since the changes in the differential furnace pressure regime within the fire resistance test, introduced in the early 1970s, no test data have been placed in the public domain. Spalling is likely to be more of a problem in high-strength concrete rather than in normal or low-strength/low-density concrete. The major indication of this is a loss of fixings for any installed element, e.g. a window or door. Consequently, all fixings should be made away from any concrete edge(s) where spalling could occur. All lintel reinforcement should be used such that the stresses are to be redistributed around the opening. To reduce the risk of spalling BS EN 1992-1-2 can be used for the depth of concrete cover to the steelwork in relation to all the heated faces of the wall, including the reveal of any aperture. Excessive moisture content can lead to an increased risk of spalling.

11.2.2.4 Insulation (I)

It should be relatively easy to predict the unexposed surface temperatures of concrete by modelling. When using test data in support of predictive calculations, the free moisture content at the time of measurement affects the temperature profile. For moisture levels other than those tested, ASTM E110 – 09c [37] provides a method for making corrections.

11.2.2.5 Cast *in situ* elements

Almost all cast *in situ* constructions are loadbearing and any evaluation of the loadbearing capacity should be in accordance with Clause **12**.

In situ concrete is unlikely to produce integrity failures due to fissuring or erosion, particularly when reinforcement is incorporated or when deflections do not exceed the accepted limits specified in standard test procedures. However, integrity failures between concrete elements and any adjacent construction are possible, e.g. in stairway enclosures, glazed screens and door assemblies, unless deflection is accommodated.

The thickness of cast *in situ* construction is normally substantial enough to support the design loads, so satisfying the insulation criteria is easy for low periods of fire resistance.

As *in situ* concrete is homogeneous, it lends itself to computer analysis more readily than most composite forms of construction. The majority of structures are designed in accordance with codes so that the methods given in Annex D are not likely to be appropriate for cast *in situ* concrete elements.

Composite metal deck floor systems are discussed in Clause **12** in respect of establishing the loadbearing capacity. As these have a continuous steel soffit they normally provide high levels of resistance to integrity losses and compliance with the insulation criterion can be thermally modelled.

11.2.2.6 Masonry and brick walls

Blockwork walls can be loadbearing or non-loadbearing.

Walls constructed using masonry and blockwork experience a temperature gradient between their exposed and unexposed faces. This induces significant thermal bowing (lateral displacement) which, if unrestrained, results in excessive movement. If the head is unrestrained the bowing occurs at the top, but, if both the top and bottom are fixed, maximum bowing occurs at mid-height. The insulation properties of the brick or blockwork directly influence the extent of bowing, with the greatest displacements occurring with materials having lower heat transfer properties. Bowing can restrict conventional thickness brick or blockwork walls to a maximum height of 3 m to 4 m, assuming the base is adequately fixed, beyond which instability occurs. The magnitude of this movement might be as expected from a one-dimensional cantilever. A non-loadbearing wall is more likely to be unrestrained at the head than a wall supporting an imposed load.

A wall expands as it is heated. If insufficient allowance is made to accommodate this movement, a load is imposed upon the construction assembly even though it might have been designed as non-loadbearing. This occurs when it abuts against part of the structure that is relatively stiff, i.e. the underside of a beam or floor slab. Any loading of this type normally starts off as being eccentric, as only the exposed face is heated and therefore expands. However, with prolonged heating the temperature differential between the hot and cold faces reduces and the loading becomes more concentric. If an allowance is to be made to prevent distortion due to restrained expansion, the gap should be sealed with a proven linear gap sealing system, although performance can only have been established under standard heating conditions. Guidance on linear gap sealing can be found in the Intumescent Fire Seals Association (IFSA) Code of Practice [38]. Additional information is given in the ASFP publication on fire stopping and penetration seals [39]. An inherently fire-resisting beam or a beam that is fully fire protected could still deflect significantly and yet continue to carry its design load. Consequently, any non-loadbearing wall or partition fixed beneath such a beam could still experience applied loads at the fire limit state (FLS).

The ability of an unrestrained blockwork wall to tolerate out-of-plane distortion also depends upon the quality of the mortar bond between the blocks. The choice of mortar is therefore critical to obtaining the correct balance between strength and thermal properties.

When analysing the likely performance of a non-loadbearing blockwork wall using engineering judgement within the methodology given in Annex D, the following constructional parameters should be considered:

- height of wall (R, E);
- thickness of block (R, E, I);
- thermal conductivity of block (R, I);
- end fixity (R, E);

- expansion allowance (if necessary) (R, E);
- moisture content (I).

NOTE For each parameter the criteria likely to be influenced are identified in parenthesis, using the codes R (loadbearing capacity), E (integrity) and I (insulation).

The list excludes exposure and mechanical parameters that should be considered if non-standard heating conditions or a modified loading exists. Such exposure parameters might identify acceptable limits to the heating conditions. Changes in the load should also be reviewed.

11.2.2.7 Concrete beam and block floors

It is assumed that all floors are loadbearing and that ceiling membranes (see BS 476-22) are unlikely to be constructed from lay-in concrete components. Floors such as these are constructed from primary beams and in-filled with lay-in proprietary secondary floor slabs. BS EN 1992-1-2 can be used to evaluate the ability of the floor slabs to contain a fire and, as these are proprietary, test evidence against standardized test conditions is also available. The main beams can be evaluated (see Clause **12**), but the usual methodology for establishing the conformity with BS EN 1992-1-2 might not be appropriate to non-standardized heating conditions. The lay-in slabs are simply supported and covered with a non-structural concrete screed.

If a "fire-rated" suspended ceiling is fixed beneath a lay-in concrete floor it only contributes to the fire resistance if it has been tested in accordance with BS 476-21, with a floor having a similar thermal/mechanical response.

NOTE A fire resistance test to BS 476-23 is not appropriate as its scope does not cover such constructions and there are differences in furnace pressures and failure criteria between tests to BS 476-21 and BS 476-23.

Excluding fire exposure conditions or mechanical parameters which cover all aspects of fire behaviour, the following parameters of a lay-in concrete floor construction influence the performance in respect of the criteria identified in parenthesis:

- span of slabs (R, E);
- bearing (R, E);
- thickness of slab (R, E, I);
- position and dimensions of any reinforcement (R);
- concrete strength (R, E);
- thermal conductivity/density (I);
- expansion allowance, if necessary (R, E);
- moisture content (I);
- gaps (E);
- presence and properties of any screed (E, I).

All fire resistance tests should be performed with the moisture content in equilibrium with the laboratory environment to represent conditions encountered by the element in practice. As with *in situ* concrete (**11.2.2.5**), where the moisture content at the time of test is not in equilibrium with the in-use condition, the ability to meet the insulation criterion of 180 °C maximum temperature rise, or 140 °C mean temperature rise, differ and correction factors should be applied (see **11.2.2.4**).

11.2.3 Elements primarily composed of metal

11.2.3.1 General

Aluminium is typically used for roofing and cladding as part of a composite system. Generally, steel is used in any application where fire resistance is a primary requirement.

Steel is rarely used on its own to construct a separating element, except as a simple form of non-insulating cavity barrier as permitted in some prescriptive design codes. However, it is frequently used as the primary component in the construction of a number of "closures", such as:

- hinged or pivoted doors;
- sliding doors;
- roller shutters (vertical/horizontal); and
- dampers.

Steel and aluminium are used as major components in the construction of sandwich panels (see **11.2.5.1**):

a) as a facing to some forms of proprietary lightweight partitioning systems; or

b) as suspended ceiling panels, the majority of which are not designed to provide fire protection.

For external applications, steel and aluminium can both be used as part of a cladding system for insulating walls and roofs. Walls might need to be fire resistant when close to a boundary, as designated by prescriptive regulations and codes or the fire strategy. Roof systems should prevent fire penetration close to junctions with compartment walls or adjacent to escape routes by means of a protected zone, where the resistance to the penetration and spread of fire is controlled [36]. Roofing systems are not usually designed to contain fire, only to resist inward penetration. However, some applications might require fire resistance from within, such as in the protection of high level escape routes. External cladding might require fire resistance from inside or outside depending upon the fire strategy approach taken.

11.2.3.2 Integrity (E)

The initial influence of heat on any metal is that of expansion but this is followed by phase changes, resulting in a, usually detrimental, alteration in the physical properties. In the case of aluminium and its alloys, melting occurs at temperatures of around 590 °C to 650 °C. Steel does not melt until significantly higher temperatures are reached and it is extremely rare for a fire of such intensity to occur. However, after prolonged exposure at high temperatures of, typically, 1 000 °C, severe oxidation of carbon steel occurs. Although sheet steel does not normally fissure, there is a minimum thickness gauge that can be tolerated to satisfy the gap criteria of integrity.

Unless steel is heavily insulated the cotton pad method for evaluating integrity is not generally appropriate as the radiation from the surface itself could cause the pad to ignite. However, due to the resistance of steel to fissuring, it can provide high levels of integrity resistance regardless of whether one of the mechanisms is unsuitable. Measuring joints between panels are the exception and such gaps can only be measured by gap gauges.

11.2.3.3 Insulation (I)

Metals are unlikely to satisfy the criterion of insulation for any significant duration without the aid of applied insulation. In comparison with other building materials, metal has a high level of conductivity. The rise in temperature of the unexposed face of, e.g. a simple steel element can be predicted by means of 3D transient state thermal analysis models. These models can make predictions when steel is used in conjunction with simple forms of insulation, but when the construction is more complex and the number of interfaces between materials increases, the accuracy of a thermal model is reduced. Test evidence, including that generated by the standard exposure conditions, might be the only way of determining the unexposed face temperature, or at least correlating any calculated temperature.

Connections between exposed and unexposed facings create local heat paths capable of producing "hot spots", but this is unlikely to result in a significant fire spread risk, although they could cause a test failure. The face of an insulating metal door, both at the edges of the door leaves and the adjacent door frame, are unlikely to satisfy the insulation criteria. Historically this has not been regarded as a hazard as materials are not likely to be "stacked" in contact with the unexposed face of a door. Consequently, many doors in prescriptive regulations are exempt from the need to satisfy the insulation criterion. This should be reviewed if the door ever becomes redundant. Uninsulated doors radiate significantly with the associated life safety, ignition, and fire spread risks, and they should only be used in the construction of refuges after due consideration.

11.2.3.4 Radiation

Metals, particularly steel, reach high temperatures when exposed to fire and radiate heat readily into any protected enclosure (see Clause **10**).

11.2.3.5 Hinged or pivoted door assemblies

Except for a few exceptions where aluminium is used in the construction of proprietary, low fire resistance duration doors, most fire doors are made of steel. Hinged and pivoted doors can take the form of a single thick sheet of steel, possibly with edge thickening for stiffness, or can be constructed from two steel skins separated by spacers, which might or might not incorporate insulation. These doors are unlikely to suffer a loss of integrity due to penetration owing to the imperforate nature of the steel (see **11.2.3.2**). They are, however, likely to suffer a loss of integrity at the frame/leaf edge junction due to thermally induced distortion.

Some doors containing an insulating material in the core can be classed as insulating. However, the majority of these are not insulated at the leaf edges because of the difficulty in providing strength to the leaf and fixing points for building hardware, e.g. hinges and locks. As these edge zones heat up during a fire, they are exempted from the insulation criterion by fire resistance testing and classification procedures. Therefore the edges are only adjudged by compliance with critical maximum gap dimensions. In terms of fire safety, there is a greater likelihood of hot gases passing into the protected enclosure in which gaps have been monitored solely by the gap criterion. The conditions within the unexposed space remain tenable for longer when the door has satisfied the cotton pad procedure in the fire resistance test. A partial solution to the problem is to fit a heat-activated seal which extends the duration for the protected enclosure to remain tenable, even if the gap criterion is used.

Thermally induced distortion can produce a loss in integrity by allowing the leaf to bow out of the confines of the frame and cause gaps to develop between the leaf edge and the frame. Fully restrained doors with multi-point latching and several hinges, generally resist distortion, thereby maintaining an imperforate barrier within the door opening. Where levels of restraint are lower, i.e. a single point or an unlatched door, the ability of the door to remain in place is controlled by design so that the door expands across its height and width. This then allows the door to "lock" into the frame before the thermally induced distortion causes the leaf to bow outside of the plane of the frame.

During the early stages of heating of uninsulated hollow doors, the air gap between the two faces results in increased distortion as a result of the temperature differential, hence expansion between the two skins. However, later in the fire when radiation becomes the primary method of heat transfer between the two faces, the temperature differential reduces significantly, as does bowing due to expansion. In the case of insulated doors, temperature differentials can be established early. These remain throughout the period for which the door satisfies the insulation criterion, indicating that thermally induced distortion influences bowing throughout the period of exposure.

The expansion of the door leaf in any direction can be characterized by:

$$\Delta L = 1.4 \times 10^{-5} h_{\text{door}} T_{\text{door}} \tag{64}$$

For steel doors and roller shutters, expansion is often beneficial as a method of locking these components in place.

Where the differential temperatures cause the door to bow, it might be possible to estimate the extent of movement, and, therefore, the likelihood of a gap developing between the leaf edge and the frame.

The movement at the head of the door (out of plane) can be approximated by an arc having radius, r, where:

$$r = \frac{d_{\text{door}}}{T_{\text{door}} \times 1.4 \times 10^{-5}} \tag{65}$$

The movement of the door head can be calculated approximately by:

$$\Delta_{\text{head}} = r - \sqrt{r^2 - \left(\frac{h_{\text{door}}}{2}\right)^2} \tag{66}$$

This technique is only as accurate as the temperature predictions, which are in turn related to the sophistication of the modelling used. The only quantifiable evidence is generated against the standard fire-resisting test conditions.

A more accurate way of establishing predictive behaviour is through a series of designed experiments. The objective is to establish coefficients for the influence that a range of factors has on the behaviour of the leaf, or leaves. Typical factors might be the height, width, thickness, position of lock, number of restraint points or exposed face temperature etc. Unfortunately, this requires multiple testing on a prototype before the "model" can be established. The principle has been developed for timber doors (see **11.4** and BS ISO/TR 12470).

The deflection at the meeting stile of a pair of doors, or the deflection of the closing edge for a single door, can be calculated from the curvature of the door. Maximum distortion occurs when the differential temperature between the two facings is highest which is in the early stages (the first 20 min) of a furnace test exposure. Steel doors, uninsulated and insulated, hinged to open away from the fire, are generally less able to resist fire than those opening inwards.

When using the parameter/factor influence method (see **11.4** and Annex D) the number of parameters forming a design are extensive. As a result, it is a very complex method, more suited to use on simple assemblies if a high degree of accuracy is to be achieved. However, when the fire behaviour of an untestable construction has to be predicted, it is one of the few methods that allows for an estimate of the performance. The following constructional parameters should be considered for hinged and pivoted doors (the criteria that the changes influence are given in parenthesis):

- leaf construction, including insulation (E, I);
- variation in thickness, cross-section or number of stiffeners (E, I);
- cross-section of steel frame and any infill (E, I);
- smoke seals; non-insulated doors (E);
- smoke seals/heat-activated seals; insulated doors (E, I);
- ironmongery/hardware;
- hinges/pivots and welding details if appropriate (E);
- latches/locks (E, I);
- closing devices (E);
- size and shape of glazing angle/beads (E, I);
- glazing material/system (E, I);
- decorative facings (if any) (E, I).

These constructional parameters should be generated for every type of design. However, they do not consider the thermal and mechanical parameters resulting from a change between the exposure conditions in practice and those used in the standard test. One such parameter is the position of the neutral pressure axis in the fire and it could result in a door experiencing only negative pressure in some applications and not the passive pressure required by the test standard.

Having established the methods by which the leaf can lock into the frame, the distortion of the frame itself could continue to be a problem. Strong fixings into the supporting construction either side of the centre line of the frame, intended to provide enhanced resistance to bending, might reduce the magnitude of the distortion. An alternative solution is to in-fill the frame with a mortar, or similar material, which both stiffens and increases the thermal mass of the frame. However, the fixing method is only effective if the primary structure into which the fixings are made has adequate strength and the fixings are stable at high temperatures. Modern forms of construction, such as studwork walls (steel or timber) or composite panels, are unlikely to provide adequate support and restraint against bowing unless suitably reinforced and stabilized.

11.2.3.6 Sliding door assemblies

Sliding doors are less prone to distortion than hinged and pivoted doors as they are typically restrained at the top and bottom and fixed in the plane transverse to the opening by the track and suspension mechanism. They are difficult to seal satisfactorily to prevent the flow of hot gases and smoke and, since the edges are generally only measured by the gap criterion, they can be susceptible to leaks. Sliding door design is quite simple and the bulk of the leaf area can be modelled for heat flow by means of a suitable 3D transient state thermal analysis model (see **11.2.3.3**). In order to quantify the level of protection that a sliding door is able to provide, only evidence generated by the standard test is available. A sliding door is likely to be too large to fit in the furnace to be tested. Therefore, the quantified performance relies heavily on extrapolation techniques, such as those given in Annex D. When using these techniques the following parameters should be considered (the criteria that the changes influence are given in parenthesis):

- insulation if fitted [type and thickness (E, I)];
- edge seals [smoke or heat (E)];
- suspension system (E);
- latches/locks (E);
- self-closing devices [including gravity assisted suspensions/supports (E)].

For any particular design the number of parameters used are extensive. The list does not include the thermal and mechanical parameters.

11.2.3.7 Roller shutters (vertical and horizontal)

Typically installed shutters are much larger than those tested, whether in terms of height for industrial buildings or width for retail buildings. There are no published methods for extrapolating the size of these assemblies but a number of certification authorities have developed "in-house" models of the standard exposure conditions. If used in a fire engineering context, validation of the model should be sought.

It is not viable to construct shutters that satisfy the insulation criterion for prolonged durations, therefore high temperatures and consequently high levels of radiation on the unexposed face should be anticipated. To estimate the risk of fire spread, it can be assumed that the unexposed face temperature of the shutter is roughly 100 °C lower than the exposed face temperature after the first 20 min, and the radiation from the shutter should be calculated accordingly. Consequently, there should be a zone around the roller shutter that is free from combustible material if fire spread by radiation is to be avoided [40]. Some prescriptive codes restrict the amount of wall that can consist of shutter. Some shutter arrangements, such as tandem shutters, might provide reduced levels of radiation.

As the steel lathes or slats in a shutter are solid the temperature of the unexposed face can be determined fairly accurately but conservatively, as the calculations cannot incorporate a component that covers hot gas leakage between the lathes. This temperature then allows the radiation to be calculated more accurately.

These calculations of fire spread resulting from temperature and radiation do not measure the risk to life resulting from smoke and hot gas leakage. Both the lathes and the guides are difficult to seal against the passage of high temperature gases and smoke. As for hinged and pivoted steel doors, whilst a shutter could be given a prolonged integrity rating, gaps do exist through which smoke and hot gases could pass, which can seriously compromise the protected enclosure.

If the analysis of fire protection is to be established in accordance with Annex D, the following critical parameters should be considered, together with the criteria they influence:

- width/span of shutter curtain (E);
- height/length of shutter curtain (E);
- orientation (E);
- cross-sectional geometry of lathe (slat) (E);
- material of lathe (slat) (E, I);
- thickness of lathe (slat) (E);
- material of guide (E);
- thickness of material used for guide (E);
- cross-sectional geometry of guide (E);
- length of barrel (E);
- diameter of barrel (E);
- thickness of barrel wall (E); and
- restraint to ends of the barrel (E).

This list does not include the thermal and mechanical parameters. Modern forms of construction, such as studwork walls and composite panels, are unlikely to provide restraint against bowing unless supported and stabilized.

11.2.3.8 Fire-resisting metal dampers

Dampers are a form of closure used to prevent the spread of fire and smoke via a ductwork system. Some systems on the market are all-steel constructions consisting of a number of moving parts activated by a thermal link or detector. In the closed condition they typically form a single sheet of steel with overlapping zones where the "blades" meet. The dampers can be sited at the entrance/exit to the ductwork or within the duct at any point where it passes through a fire-resisting element.

It is possible for the dampers to consist of an insulated construction but, as prescriptive codes rarely specify an insulation requirement, most dampers are non-insulating. During a fire, these become hot on the unexposed face and radiate accordingly. This does not directly influence fire spread as the inside of the ductwork receives the heat flux from the damper. If the ductwork is insulating then the impact on the unexposed surface temperatures of the duct is minimal.

In most applications, however, the ductwork is uninsulated and therefore heats up on the unexposed face due to the heat from the damper radiating heat on the duct walls. Although this gradually heats the duct, the temperature it reaches is the sum of many factors, including the leakage rate and the flow rate. However, test evidence generated against the standard test procedure is typically the only way to quantify the fire separation provided by such devices.

Neither the gap gauges nor the cotton pad are used to establish the integrity rating because of restricted access. The integrity of a damper is expressed in terms of leakage rate measured at various pressure differentials across the damper. As with temperature rise, any failure to meet the insulation criteria only has an indirect influence on life safety or property protection.

Of greater importance when considering the ability of dampers to prevent fire spread from a compartment into an adjacent space via an unseen route is ensuring that they operate when required. It is often difficult to visually audit their performance and implement remedial action in the event of a fire. Operational testing is vital to ensure reliability and this is more likely to occur if the damper is motorized. Motorized dampers, operated by a signal from a smoke/heat detector, are less prone to delay than a thermal trigger which might be slow to activate due to laminar flow in the duct.

The most likely source of failure in dampers is via the seal between the wall and the damper frame. The installation of any fire stopping or linear gap sealing should be as tested, particularly where there is the possibility of significant movement due to changes in the cross-section or shape of the damper housing.

When carrying out an extended application exercise to establish the performance of a damper in accordance with Annex D, the following parameters and the criteria they influence should be considered:

- change in pressure (E);
- increase in width (E);
- increase in height (E);
- change in method of fixing/installing (E);
- number of blades/irises (E);
- insulation on blades or adjacent duct (E, I).

The list does not include the thermal and mechanical parameters.

11.2.3.9 Metal studs or joists in separating elements

11.2.3.9.1 Metal studs

Light gauge metal studs are often used to form the structural frame for non-loadbearing fire separating walls. These typically consist of steel channels, or “C” sections, which are used to construct a framework to support a variety of linings. Depending upon the fire rating required to meet the strategy, insulation could be incorporated within the cavity. For greater fire resistance an insulation which protects the unexposed face lining is often fitted. Similarly, a number of ceiling membranes incorporate steel joists.

Metal expands when subjected to heat, and a lined metal stud frame can move in different directions in the x, y and z planes during a fire. In the early stages of a fire, one flange is at a higher temperature than the other and the temperature differential between them can cause the stud frame to bow into the fire. Eventually, after the exposed face linings have been lost due to erosion, been burnt away or have fallen out, the metal frame closely follows the fire’s temperature. At temperatures greater than 800 °C the metal has either melted, in the case of aluminium, or has little strength remaining, and integrity failure is likely. To obtain high levels of fire separation, suitable board linings should be used that protect the metal from direct exposure to the fire for as long as possible. Metal stud constructions can be lined with a variety of linings. Non-timber based linings are considered below and timber based linings in **11.2.4.6**.

a) *Calcium silicate boards*. Calcium silicate boards are manufactured from ground materials including lime, cement, silica and fire protective fillers in combination with cellulose fibre. The finished boards come in a range of types offering hot strength and possibly fire-resisting properties and stability in fire.

   The boards are categorized as non-combustible when tested in accordance with BS 476-4 and designated as a class "0" building material, as defined in Approved Document B to the Building Regulations [1].

b) *Cement-based boards*. Cement-based boards are smooth, flat and durable and are manufactured using a mixture of cement and binding or reinforcing materials, such as engineered wood filaments. They have some loadbearing capabilities as well as sound insulation, fire performance and moisture resistance. These boards are categorized non-combustible when tested in accordance with BS 476-4 and designated as a class "0" building material.

c) *Gypsum plasterboard variations*. Gypsum plasterboard consists of a gypsum core encased in, and firmly bonded to, strong paper liners. Gypsum is non-combustible and plasterboard contributes to the fire resistance of the structure in which it is attached, either as a wall or ceiling lining, or as directly applied for protection. Several grades of plasterboard are available, defined by the quality and density of the gypsum.

   Where extra fire protection is required, fire resistant plasterboard should be used. It has increased resistance to fire and incorporates glass fibres and vermiculite additives which maintain its strength when hot. Boards of this type are categorized as limited combustibility and are designated as a class "0" building material.

As there are a number of proprietary boards available, the manufacturers' information should be sought in respect of their fire performance [41].

There are no design guides for non-loadbearing partitions. Unlike the standards and codes that exist for conventional structural materials, predictions on the fire performance of a specific partition design should be made using the extended application process (see Annex D). Excluding the exposure and mechanical parameters, the constructional parameters that should be considered and the criteria they influence are:

- characteristic strength, gauge and dimensions of materials forming any stud (E, I);
- stud centres (E, I);
- the thermal behaviour and thickness of the exposed and unexposed linings (E, I);
- type, position and fixing of any insulation (E, I);
- the method of fixing linings (E);
- the method of jointing linings (E, I);
- penetration by services, e.g. switches, and the method of sealing them (E).

NOTE It is increasingly common to fit proprietary sealing systems to protect such penetrations, especially for longer durations or where the insulation makes no contribution to the performance. Most product claims for board materials are made without penetrations.

11.2.3.9.2 Metal joists

Numerous lightweight metal joist systems or wood/steel composites exist which, though loadbearing, have very low levels of inherent fire resistance (see Clause **12**). Like stud walls, their fire performance is heavily dependent upon the contribution of the linings. Boards used for partitions are also suitable for floors. However, gravity can cause the protection they provide to be less than in vertical elements. When evaluating the capacity of these floors to provide a separating function, the following construction factors are relevant, ignoring the influence of changes in exposure and restraint:

- characteristic strength, density and dimensions of materials forming any joist (E, I);
- joist centres (E, I);
- type, position and fixing of any insulation (E, I);
- the thermal behaviour and thickness of the exposed and unexposed linings (E, I); and
- penetrations for services e.g. concealed lights (I).

As with linings for partitions, various proprietary systems are available for protecting penetrations.

11.2.4 Elements primarily composed of timber

11.2.4.1 General

Timber is used extensively in the construction of floors and walls as loadbearing members. While Clause **10** describes how to calculate their fire resistance, **11.2.4.6** addresses the protection provided by linings used to protect these constructions, or which themselves form the barrier.

Untreated timber-based products are rarely used to form a fire-resisting barrier due to surface spread of flame. However, when suitably treated, they can be used as linings in many separating applications.

Timber or wood-based board materials are used primarily for the following separating applications:

- web(s) of composite timber (I) beams or box beams;
- timber-based linings applied to stud or joisted elements; and
- fire-resisting door assemblies.

For guidance on the use of timber in the construction of glazed joinery screens, see **11.2.6.9**.

11.2.4.2 General fire behaviour of wood-based products

When timber surfaces are exposed to heat they lose moisture, leading to caramelization of the cellulose, a precursor to carbonization or charring. While thermal degradation of the fibres takes place at low temperatures, carbonization or charring often occurs following the ignition in short term, post-flashover fire behaviour.

There are two forms of ignition, spontaneous and pilot ignition. Spontaneous ignition typically results from radiating heat sources where there is no direct flame. Pilot ignition occurs in the presence of flames capable of igniting the volatile gases produced during heating. Fire spread within an enclosure is typically the result of a series of pilot ignitions, whereas spontaneous ignition is often the cause of fire spread outside of the enclosure, from barriers providing integrity but not insulation. The temperature at which ignition occurs is not fixed but is related to the level of heat flux received, moisture content, the magnitude of any convective cooling and many other related actions. A conservative approach is to assume that pilot ignition and the onset of charring and reduction, occurs when surface temperatures attain 350 °C. Spontaneous ignition can be expected at incident heat fluxes in excess of 25 kW/m^2 [34].

A timber component exposed to heat experiences differential movement, with the exposed face bowing away from the fire due to shrinkage on that face. Unburnt timber distorts if it does not have sufficient cross-section dimensions to resist this movement. Surface charcoal is an excellent insulator with respect to both convective and conductive heat transfer, as long as the outer layer is not crushed by any of the other components. This char layer is diathermanous and, as radiation becomes the dominant heat transfer mode, pyrolysis starts to occur at the interface with the solid timber. Beyond the charred interface, there is a boundary layer at a temperature just above ambient in which the normal cold state physical properties apply. During short durations of exposure to high temperatures the heat affected zone is narrow and can effectively be ignored. Long durations of fire exposure result in a change in physical properties [42].

11.2.4.3 Integrity of timber (E)

Solid timber is unlikely to have an integrity failure until it has almost completely charred, at which stage fissuring of the thermally degraded residual timber can lead to a gap through which hot gases can pass. Composite timber-based boards, e.g. particle board, plywood or medium density fibreboard, are less likely to fissure until the board has almost been consumed, due to the more random orientation of fibres. Although not documented for vertical applications, integrity is expected for 90% of the time to consumption and the same for composite boards such as chipboard, assuming that all board joints are made over studs and the fixings are adequate. For boards used in the horizontal plane, i.e. transverse to the studs and not backed up by noggins/duanges (cross-timbers), the duration before integrity failure occurs is less than vertical components. This is dependent upon the quality and frequency of fixings and the self-weight of the board. Evidence of performance, even from standard tests should be used to quantify the protection provided.

NOTE For guidance on the anticipated rate of charring, see Annex A.

Timber is a hygroscopic material and contains between 1% and 20% water. Heat exposure drives this water off, causing shrinkage. In the case of solid timber, the shrinkage typically occurs transverse to the grain. In board materials, shrinkage is typically equal in all directions as there is no dominant grain direction. The introduction of resins, such as those used in chipboard, modifies the rate of shrinkage.

Shrinkage can cause premature loss of integrity at junctions between timber and timber-based products, or between these materials and any adjacent construction. Solid form or gunned intumescent mastics are used to compensate for shrinkage and maintain a tight gap until the material is consumed. They function almost regardless of the exposure conditions, albeit higher temperatures can exhaust the material quicker.

The rate of consumption or charring of timber is not significantly influenced by increases in gas temperature over 400 °C to 900 °C. Above this range, where the radiative component of the heat flux is much higher, the rate of charring increases and this should be considered when calculating the consumption rate and associated fissuring time for non-standard heating conditions.

11.2.4.4 Insulation (I) of timber

Timber has a very low thermal conductivity (see Annex A) and is therefore an excellent insulator. Wood-based board materials are generally denser than natural wood and contain significant percentages of resins or other chemicals so their thermal conductivity is higher. As timber does not exhibit critical changes of state when heated, there are no 3D transient state thermal analysis models available to evaluate timber constructions. Timber and wood-based materials satisfy the insulation criteria of the standard test until nominally 5 mm of material remains. The material ignites before it reaches surface temperatures that could generate ignition of adjacent cellulose.

11.2.4.5 Timber joists or studs in separating elements

Timber joists and studs alone do not provide integrity and insulation. However, they are often used as the main constructional component in separating walls and floors and, as such, make a major contribution to maintaining the integrity of linings that do provide the separation function. The greater the stability of the stud or joist, the greater the likelihood that the board(s) remains in place, particularly those not initially involved in the fire, i.e. those protecting the protected enclosure. As many of the studs/joists are protected by, or support, proprietary boards, the evidence in support of their use in a fire strategy is often only the board producer/supplier's test reports. The majority of this evidence was generated by standard fire resistance tests and similar performances should not be assumed for non-standard exposure conditions. Evidence generated against the hydrocarbon test conditions, if available, could provide reasonable data for protection against heavy exposure conditions.

11.2.4.6 Timber and timber based linings to studs and joists

Timber-based linings are able to make a significant contribution to the fire separating capabilities of stud and joist framed separating constructions, regardless of whether the studs are timber or metal.

The rate at which timber is converted into charcoal is discussed in Annex A. However, these figures relate only to solid timber sections of a minimum section size of 70 mm. They do not apply to composite board materials, such as chipboard or medium density fibreboard. For calculation purposes it can be assumed that composite boards with a dry density greater than 700 kg/m^3 burn away at a rate of 1 mm/min. The introduction of flame retardant salts into these materials by impregnation or during manufacture to improve their surface spread of flame rating, rarely reduce their burning rate and could cause the material to be eroded faster.

When determining the contribution of tongued and grooved, non-backed up joints, the integrity calculation should only take into account the thickness of material up to the outer, or non-exposed surface of the tongue.

In many cases, the integrity of a wall or floor lined with timber facings on one or both sides is enhanced by infilling with insulation, e.g. mineral rock fibre for acoustic or fire protection purposes. However, not all insulation contributes to fire-resistance and manufacturers' technical information should be consulted. The influence of variations of fire resistance provided by stud or joist constructions in respect of the identified criteria can be established in accordance with Annex D. The following construction parameters should be considered.

a) Studded and board clad constructions:
    1) characteristic strength, density and dimensions of materials forming any stud (E, I);
    2) stud centres (E, I);
    3) type, position and fixing of any insulation (E, I);
    4) thermal behaviour and thickness of the exposed and unexposed lining (E, I);
    5) method of jointing linings (E, I);
    6) penetration by services e.g. switches (E).
b) Joisted and board clad constructions:
    1) characteristic strength, density and dimensions of materials forming any joist (E, I);
    2) joist centres (E, I);
    3) type, position and fixing of any insulation (E, I);
    4) thermal behaviour and thickness of the exposed and unexposed lining (E, I);
    5) method of jointing linings (E, I);
    6) penetration by services e.g. concealed lights (E).

NOTE As with linings for partitions, various proprietary systems are available for protecting penetrations.

These do not take into account thermal or mechanical parameters which can vary significantly in a fire engineered analysis.

11.2.4.7 Fire-resisting timber hinged and pivoted doorsets

Timber and wood-based products are frequently used in the manufacture of fire-resisting doorsets, which play an important role in maintaining an enclosure. Whilst these doors traditionally use solid timbers, modern flush door leaves are more likely to consist of a timber frame around a composite board with wood based facings. Joinery doors are still constructed from solid timber, containing either glazing or solid panels as appropriate.

Most doors fail to provide their potential resistance to fire as a result of distortion, generated primarily by asymmetric heating, irrespective of whether they have been designed as fire doors or not. The factors that influence the ability of the door to resist distortion and stay within the frame while maintaining the seal around its perimeter to prevent the spread of smoke and flames are:

- method of construction (jointing, glues, etc.);
- size of the leaf: thickness, height and width;
- quality, density, species and section size for the timber used in stiles, rails and muntins;
- degree of restraint provided to the leaf or leaves from hardware (e.g. hinges, latches, closers) and/or certain types of intumescent seals (as fitted);
- size of mortices and morticed components (latches, closers, etc);
- size of the perimeter gaps;
- type and design of any glazing, including retaining bead details and fixings.

The choice of timber can also influence the distortion pattern. Timbers that burn slower leave a larger unburnt section and are better able to resist the shrinkage of the exposed face. Some timbers are inherently more stable because of the straightness of their grain and freedom from knots, etc. The tendency to shrink when exposed to fire means that hinged doors opening towards the heat are less able to resist fire.

The size of the leaf has a large influence on the amount of distortion that can be accommodated. Control of the distortion of doors under fire exposure is normally achieved by means of restraints applied through builders' hardware, for example, locks, latches and edge bolts. The choice of configuration, i.e. whether the door is single action, affects how much restraint can be applied by such devices. Intumescent edge seals can also maintain the integrity of gaps between the leaf and the frame by controlling distortion through generating of friction forces from the expanding material. Intumescent seals made from sodium silicate or intercalated graphite are more likely to produce the necessary restraining pressures than other forms of intumescent material, such as monammonium phosphate.

The type of builders' hardware used can influence the localized burn-through of the door. Hardware items can conduct heat and offer potential paths for integrity failure and should always be specified as part of the door assembly. Care should be taken when hardware is different from that evaluated in the proving test for the door. As a consequence, the integrity of a timber door is more likely to be lost at the perimeter leaf/frame interface than in the body of the leaf, except where hardware, glazing or a reduction in thickness compromises the performance.

Designing timber doors from first principles without expert assistance is not recommended and proprietary products, tested in the mode and configuration required, should be used. However, these doorsets are only tested against the standard exposure conditions and some allowance might be made if the predicted conditions vary significantly, resulting in changes in the exposure parameters, either positively or negatively. The door leaf is, however, dependent upon the frame to provide compatible support.

Timber door frames do not exhibit the very large distortions associated with metal door frames. The slight shrinkage-induced bow that they exhibit when heated is normally contained by the fixings into the supporting construction. The number of such fixings should conform to the recommendations of the test documentation or any subsequent field of application assessment. Unless specifically permitted, the fixings should not incorporate any low melting point components.

Metal door frames are not compatible with timber door leaves due to the difference in the degree of the distortion and thermal conductivity of the materials. The recommendations of **11.2.3.5** for reducing the distortion of metal frames increase the compatibility, but care should be taken before specifying these combinations. Aluminium frames, whilst normally used only for low durations of fire resistance, are more compatible than steel frames.

Many fire doors require vision panels for safety or cosmetic reasons. The removal of material from a leaf often affects the distortion pattern of the leaf due to a reduced stiffness in these areas. If the protection is not properly reinstated by correct installation of the glazing, this can lead to a total reduction in the protection that the leaf is able to provide. It is vital that the sizes and positions of apertures are approved and that the glazing system is compatible with the core type and leaf construction of the door. Many of the fire glasses that can be used (see **11.2.6**) do not provide any significant insulation performance. As a consequence, there is a greater flow of heat through the vision panels, which might compromise the level of protection the door would otherwise provide. This is important in refuge areas or where there could be a risk of fire spread by radiation. Glasses that do not radiate are more suitable for these applications.

Further information on the critical design aspects of timber-based fire doors can be found in BS 8214.

There are no numerical models that can predict the performance of a fire-resisting doorset, even against the standard test conditions. Since timber is a natural product it has a variable structure exhibiting different physical properties and characteristics, not just between species, but also within a species. As a consequence, it is difficult to characterize the material for modelling purposes with any degree of confidence.

When using the recommendations in Annex D, the numbers of parameters which form part of the design process are even more extensive than for metal doors. As a result, it is difficult to analyse all the parameters and, initially, is most suited to the less complex configurations if a high degree of accuracy is desired. However, when the fire behaviour of an untestable construction is to be predicted, it is one of the only methods that allow an estimate of the performance without resorting to test. The following primary construction parameters of hinged and pivoted timber-based doorsets and the criteria they influence should be considered for every construction or leaf design:

- size of opening components – leaf (E);
- number of leaves (E);
- mode of operation, e.g. single swing or double swing (E);
- leaf construction including adhesives (E, I);
- frame type (E):
    - timber; type and density (E);
    - metal; type, section size and filling (E, I);
- edge seals, particularly intumescent, quantity and type (E);

- ironmongery/building hardware:
  - hinges/pivots (E);
  - latches/locks (E, I);
  - closing devices (E);
- position and size of any glazed openings (E, I);
- glazing system, glass type (E, I);
- applied finishes/facings (E, I).

11.2.5 Elements constructed from composite panels

11.2.5.1 General

Composite panels, consisting of a structural core with metal faces on each side, are an increasingly common building component due to the large spans and heights they can accommodate, coupled with fast erection methods. Sandwich panel constructions are used as separating elements in:

- internal walls (fire-rated and non-fire-rated) forming part of an internal envelope or cellular layout;
- horizontal ceilings (fire-rated and non-fire-rated) as a membrane ceiling (including walk-on ceilings);
- external walls in "boundary" situations.

There are a number of alternative core materials in regular use, e.g. foamed polymerics, mineral rock fibre and other foamed insulating materials. Data on the thermal properties are given in Annex B. A high temperature tolerant core is needed if the element is to provide fire resistance. Although panels constructed with combustible cores could still restrict fire spread, this usually involves incorporating steel cover plates and additional fixings to both sides over and around joints. The use of high temperature tolerant cores does not automatically indicate fire resistance, especially at the heights and spans used in buildings. These sizes are generally in excess of the sizes tested and the use of composite panels at these sizes should be supported by an extended application analysis. Some insurers, however, do test the fire performance of composite sandwich panels for inside corner configurations at a height of 15 m (50 feet) as part of routine certification, although not typically for fire resistance.

Guidelines on the correct use and design of enclosures constructed from composite panels can be found in the Guide published by the International Association for Cold Storage Construction (IACSC) [43].

11.2.5.2 Integrity of composite panels

Typically the facings are steel, providing the structural strength to support the panel's self-weight. These facings are capable of resisting fire penetration for a significant duration through the main body of the panel. If the panel facings are aluminium they melt at fairly modest temperatures, resulting in a loss of integrity unless the insulation is of such a type, and fixed in such a manner, that it can protect the unexposed lining. Plain carbon steel experiences severe oxidation at temperatures in excess of 1 000 °C and, if the steel gauge is too small, this could lead to fissuring, particularly when significant distortion causes the oxidized surface to delaminate. However, until these temperatures are reached steel can satisfy the gap criteria of the standard test. As stated in **11.2.3.2**, the cotton pad is not a suitable method for establishing the integrity of steel linings unless they remain well insulated by the core material.

Steel readily expands and, if restrained, could cause significant distortion. If the joints are not adequately designed and/or constructed this can result in an integrity failure, most likely occurring at joints between panels rather than through the body of the panel. Analysis of such tests has shown that the "free edge", usually incorporated in a test specimen as a requirement of the test standard, can create an artificial mode of failure. The need to use a free edge when testing metal faced sandwich panels has been questioned in ISO/TR 834-3.

Gaps in panels containing cores that do not melt or erode significantly are easier to seal and retain integrity better than those that melt or erode.

11.2.5.3 Insulation of composite panels

As steel has a high thermal conductivity it transmits heat readily, so the temperature rise on the unexposed face of a composite panel is primarily influenced by the nature of the core. If the core is metal then the heat from the exposed metal sheet is rapidly transmitted to the unexposed face, so raising its temperature. If, however, the insulation remains intact then the rate of temperature rise of the unexposed metal face is reduced depending upon the thermal conductivity of the infill material. In practice, either the insulation melts or is eroded, or its characteristics changed as a result of, for example, the loss of binders/fibres. The rate of temperature rise of the unexposed face is therefore significantly higher than might be anticipated using the cold state insulation values.

The temperature rise of the unexposed face is higher when the unexposed face is adjacent to openings and through-joints due to heat paths generated through the assembly. Thickness in metal connections, for the purpose of strength or stiffness, can result in "hot-spots", but in a fire engineering strategy these localized increases might not compromise the fire safety objectives.

11.2.5.4 Radiation of composite panels

The amount of radiation emitted from the unexposed face of a composite panel construction depends upon the insulation remaining in place and reducing the temperature of the unexposed face lining. Metals, when they reach high temperatures, radiate heat readily into the protected enclosure. Even without the benefit of effective insulation, if both linings can remain in place, a significant reduction in the radiation from the unexposed face can be achieved.

11.2.5.5 Composite fire-resisting walls forming part of an internal envelope or cellular layout

Walls constructed from composite sandwich panels are used for the construction of industrial and commercial buildings and, as such, are much taller than normal walls. Taller walls, especially those constructed from materials with a high coefficient of thermal expansion like metal, can exhibit higher levels of deflection than walls made from inert materials. If there is an adequate composite action between the facing and the core, the deflection is modified by the stiffness of the panel. However, if the composite action is destroyed in the fire process, the deflection could result in the facing delaminating and the panel losing much of its structural benefit.

If the core is combustible, the loss or buckling of the facing means large areas of fuel are more readily available for the fire. The conditions within the enclosure are made less tenable, but this might not directly affect the immediate ability of the structure to contain the fire.

Relative to the in-use sizes, the standard fire resistance test is carried out on a small specimen, i.e. not more than 3 m wide × 3 m high. Since the extent of distortion is proportional to the height of the wall (or longest dimension) care should be taken in applying the result of a standard fire test to a wall designed to be installed against a fire engineered strategy. The increased distortion could seriously affect the integrity of panel joints, both vertical and horizontal, between walls and ceilings. Panels with combustible cores lose their integrity if they span across fire-resisting barriers without being suitably fire stopped.

Large-scale fire tests to examine reaction to fire behaviour also provide information on the behaviour of panel joints (see ISO 13784-2). The LPC Design Guide [40] includes Construction Design Sheets that indicate generic performance in the Loss Prevention Standard (LPS) 1181 test, featuring panel assemblies 3.0 m high × 4.5 m wide × 20 m long.

For information on how to use sandwich panels and enhance their fire stability see the IACSC guidelines [43].

Since it is not easy for the building user/operator or fire-fighters to know what core is used in the construction of the panels, and as the choice of core influences the panel's use and performance, the fire strategy should incorporate a requirement for clear labelling of this form of construction.

When considering the influence of increased size and alternative exposure conditions by means of the methods in Annex D, the following constructional parameters should be considered:

- height of panel (E);
- thickness of panel (E, I);
- core material (E, I);
- adhesives (E, I); and
- panel joints (E, I).

As with lined stud construction walls, composite panel walls do not provide significant resistance to resist thermal bowing of metal framed, hinged or pivoted doors, metal roller shutters or sliding door assemblies. Adequate support should be given to the composite panels to both support and restrain the wall where it interfaces with doors and shutters. Door assemblies tested in conjunction with composite panel walls are more likely to be successful, although the height and length of the walls containing the door assemblies should be considered when judging their suitability to support metal framed door assemblies.

11.2.5.6 Fire separating composite panel ceiling membranes (including walk-on ceilings)

The use of composite sandwich panels for the construction of horizontal ceiling membranes is not dissimilar from their use as free-standing vertical barriers. The main differences are that any tendency to delaminate when heated is made worse by the action of gravity and hot gases collecting reasonably uniformly under the soffit of the panels.

When heated from below, the rise in temperature on the unexposed face is influenced by the choice of insulant and its ability to remain in place. The integrity of the joints is the product of their design and the amount of distortion, particularly differential distortion, between edge and centre panels where different levels of edge restraint exist.

Of particular concern with horizontal panels is the additional risk of collapse should fire occur in the enclosure of the roof space above a ceiling. If the upper face is exposed to high temperatures adhesion could break down which, regardless of the core's potential contribution to fire resistance, could cause delamination and a loss of the ceiling's fire separating capabilities from above. There is no standard method of test for fire from above as, for most constructions, exposure of the compression zone is not critical. However, this is not the case for stressed skin constructions and, in these cases, it should therefore be considered. Failure by this mechanism is not unique to any particular core and the detrimental effect can only be negated by the design and installation. The designer should consider developing a catenary action between panels to prevent such failures.

Whilst it is common to allow light foot traffic on some composite ceiling panels, excessive loads and/or use could have a negative influence on the adhesion of the upper skin exposed to fire and this should be considered in the design. Either a ceiling panel should be used which has proven stability and integrity to meet the required fire resistance, or, if the preferred panel is known to be vulnerable to degradation due to frequency of loading, dedicated independently-supported walkways should be installed.

When considering the influence of increased size and alternative exposure conditions in accordance with Annex D, the following constructional parameters should be considered:

- span of panel (E);
- fixity and restraint at panel edges (E);
- thickness of panel (E, I);
- core material (E, I);
- adhesives (E, I); and
- panel joints (E, I).

11.2.6 Elements primarily composed of glass

11.2.6.1 General

With the exception of glass bricks and some proprietary forms of frameless glazing, fire-resisting glass is used in combination with timber, steel or concrete framing. Glass is used in the construction of the following fire separating elements:

- vertical glazed internal fire screens;
- curtain walling; and
- horizontal glazed membrane ceilings.

Conventional soda/lime window glass used for normal glazing applications, even in its toughened state, should not be relied upon for any significant level of integrity unless it has been prepared and installed in a manner designed to provide fire resistance.

If fire resistance is required, the glass should be specifically designed to provide separation during the late pre-flashover or post-flashover conditions (see **11.5**).

11.2.6.2 General consideration of the response of glass elements

There are many different types of fire-resisting glass, which broadly fall into three major categories:

- monolithic glasses, wired and unwired, which do not provide any significant level of insulation;
- composite glasses consisting of glass in combination with intumescent/ablative materials, which control temperature rise and provide protection from radiation; and
- coated monolithic glasses which restrict radiative heat flow.

As these products provide very different levels of fire protection they should be glazed by methods appropriate to their behaviour at high temperatures. Even the individual monolithic glasses need different glazing methods for consistent performance. Coated glasses are directional in their performance and fail prematurely if the uncoated side is attacked by the fire.

Since the average temperature of the glass determines whether the product is rigid or starting to flow, the ambient conditions on the unexposed face are critical to its mechanical response.

11.2.6.3 Mode of failure of monolithic glasses

Cracking induced by differential temperatures on its exposed surface is the primary mode of failure for a conventional glass (monolithic clear glass of a soda-lime silica composition). Glass has a high coefficient of thermal expansion which causes significant cracking of the exposed surfaces. If glass is to act as a barrier to fire, it should be retained in position by a robust glazing system that prevents the edges from becoming hot. As a consequence, large strains are generated between the exposed and protected glazed surfaces and this can lead to thermally-induced cracking. Glass that incorporates integral steel wires can retain the fragmented glass sections together as one unit. This type of glass can satisfy the integrity criterion even when cracked. If a clear, unwired monolithic glass is intended to provide fire resistance, it should be arranged such that the strain developed between the exposed and unexposed areas does not generate excessive levels of stress.

There are several ways that this can be achieved. Toughening the glass by heat-treatment methods improves its strength, thereby making it capable of resisting the development of high stress levels, although a change in composition might also be needed. In isolation this is unlikely to be adequate and, therefore, the thermally-induced strain should also be reduced by, for example:

- using glasses with lower coefficients of thermal expansion;
- reducing the edge cover to the glass pane to a minimum; or
- using glazing systems that have improved thermal conductivity.

Typically, edge cover is restricted to a maximum of less than 10 mm when using toughened unwired soda/lime glasses to prevent unacceptable stresses being generated.

If monolithic unwired glass is to be used, glass with a lower coefficient of expansion, such as borosilicate, is the preferred option. This type of glass can accommodate significantly higher levels of edge cover (up to say 25 mm) before failing as a result of differential temperatures. Clear ceramics exhibit zero expansion characteristics and work almost independently of the glazing system, but these cannot be made into safety glasses without laminating, and care should be exercised in this process because of the flammability of polyvinyl butyl interlayers.

11.2.6.4 Mode of failure of laminated glasses

With insulated glasses, the initial reaction to heat and the resulting mode of failure is very different. There are three types of products:

a) multi-laminated glass with rigid interlayers of clear sodium silicate-based intumescent product between panes of soda/lime glass;

b) gel glasses with a void between two panes of soda/lime composition glass, filled with a gel that sets rigid and becomes opaque when subject to heat; and

c) "sandwich" glasses with a rigid clear cast core between two soda/lime glasses.

The first of these achieves its performance by the activation of the intumescent interlayers that progressively expand and erode throughout the test. The exposed pane of glass cracks early but is retained on the surface by the sticky activated intumescent layer which turns opaque and acts as a barrier to heat transfer by radiation. The action of the intumescent layer protects the next pane of glass from direct exposure to heat but, in the process, the pressure generated can crack the glass. Eventually all interlayers become activated and the exposed material is eroded along with any of the remaining intumescent product. The unexposed face surface temperature can be kept below the insulation criteria of the standard fire resistance test for periods up to 80 min, depending upon the thickness and the number of interlayers. Integrity can invariably be maintained for a period significantly beyond the duration for which insulation is satisfied, typically 30 min or more, depending upon the pane size.

Failure of intumescent laminated glasses usually occurs as a result of a localized burn-through created by coincidental glass cracks, or by the pane becoming so weak due to cracking/fragmentation of the glass that it is unable to support its own weight.

The gel or rigid core-filled glasses tend to exhibit similar integrity and insulation ratings, with failure usually resulting from a complete loss of infill core due to erosion. The weight of these glasses puts undue pressure onto the fixings and, if the glazing framing is being eroded, then the failure of any fixings could contribute to the loss of the entire glass area.

11.2.6.5 Consideration of other factors

Currently, the only method of establishing the contribution of the fire glasses to the containment of a fully developed fire is by an engineering analysis that utilizes test evidence from fire resistance tests. Glass is a temperature-sensitive material in terms of differential surface temperatures, rate of heating and mean temperatures. Consequently, evidence of performance against the standard test conditions should not be assumed to apply to a wide range of exposure conditions, especially if fluctuations are likely. If the fire engineering analysis indicates that temperatures higher than the standard curve are likely to exist, then the onset of slumping could occur earlier. Monolithic glasses do not lend themselves to prediction using time-equivalent methods. Wired glass, borosilicate composition glass and the insulated glasses can be considered as more robust to variations in temperature than other types of glass. The influence of thermal shock from fixed water suppression systems should also be considered for non-wired glasses.

Attempts have been made to model the behaviour of glass in fires, but this has been aimed at conventional annealed soda/lime composition glass in developing fires. They do not reproduce the critical characteristics of special composition fire-resisting glasses.

11.2.6.6 Integrity of glass

Monolithic clear glasses can produce an integrity failure as a result of cracking (see **11.2.6.3**). They can also fail as a result of the glass slumping, i.e. losing its inherent stiffness and self-supporting ability, thereby pulling out from the glazing system at or near the top of the individual panes. This produces a gap through which hot gases and flames can escape. Laminated insulating glasses can experience an integrity failure as a result of cracks in the glass accompanied by erosion of the interlayer local to the cracks, or by the total collapse of the pane as a result of failure of the fixings.

The performance of all types of monolithic glass can be improved by creating an insulated, cool frame around the perimeter that resists the tendency of the glass to "slump". The application of pressure to the glass edge can also help to resist slumping. This can be achieved by clamping the edges, generating a uniform pressure by utilizing a restrained pressure-producing intumescent product which might also help insulate the glass edge.

These measures are incompatible with the restricted edge cover permitted for clear, toughened soda/lime glasses so these glasses are unlikely to reliably go beyond their softening temperature. This is typically reached between 40 min and 45 min for 6 mm thick glass under the standard test conditions. Even with deep glazing systems and edge pressure, the tendency to slump can make it difficult for large panes of wired soda/lime glass to resist slumping for one hour or more under standard heating conditions. To reduce slumping for both clear soda/lime glasses with their restricted edge cover, and large panes of wired soda/lime glass, high temperature adhesives are often used in the glazing rebates on the upper edges. Little is known of the long-term reliability of high temperature adhesives and their use in any subsequent replacement should feature prominently in the fire safety manuals for the building.

For any monolithic, unwired glass, stress concentrations at the glass edge should be avoided. Glass edge preparation is critical. If any damage to these edges occurs during installation it is likely to cause a severe reduction in the integrity performance of the glass and could reduce integrity to less than five minutes.

When using monolithic, unwired soda/lime glasses, only the approved and validated glazing systems should be used. Failure of glazing due to excessive thermal stresses usually occurs in the first five minutes of heating, as the toughened glass falls wholly from the frame, breaching the fire separation.

The designer should be aware that the method of determining conformity to the integrity criterion varies significantly between the uninsulating, monolithic glasses and the insulating glasses. Due to the levels of radiation emitted from monolithic glass, the cotton pad technique (see BS 476-20:1987) is deemed inappropriate for evaluating the amount of gas leaking through a gap, as this would ignite solely as a result of radiative transmitted heat. This indicates the level of risk that monolithic glasses represent. For such glasses, only the gap criteria (using the recommended gap gauges) should be used.

The drenching of ordinary soda/lime glass with water is a method for keeping the exposed face of the glass cool, thereby reducing the edge cover temperature differential. However, unless the water curtain totally covers the surface of the glass there is a risk of the surface temperature increasing in any unwetted area, creating thermally-induced strains that cannot be accommodated by the glass. This can occur where surface deposits, e.g. grease, have accumulated on the glass, or where the flow of the water within the curtain is interrupted by shielding or air currents. The use of drenchers can only be accepted if the water curtain is homogeneous and the glass surface condition is kept even.

The sensitivity of glass to temperature from the rate of heating, the surface differentials or its magnitude, makes it unsuitable for use in conjunction with time/equivalent techniques.

11.2.6.7 Insulation of glass

Conventional soda/lime composition annealed glass cannot satisfy the integrity criteria for more than a few minutes (see **11.2.6.1**) and is unable to provide any significant insulation. Monolithic fire glasses can satisfy integrity but are highly conductive and relatively thin and, therefore, unlikely to provide more than a few minutes insulation. However, for thicker panes of glass, the unexposed face surface can take longer to reach temperatures which could ignite components on the protected side.

Heat transfer from a sheet of monolithic fire glass to the enclosure has two components: transmissive heat and emitted heat. Glass permits some heat to pass straight through, being thermally transparent until it attains a mean temperature of approximately 600 °C, at which point it becomes almost completely thermally opaque. Once glass has reached this temperature the heat transfer mechanism is primarily by emission and glass has an emissivity of 0.8 at a surface temperature of approximately 600 °C.

11.2.6.8 Radiation of glass

When glazing systems including sandwich intumescent interlayers or ablative gels are exposed to fire they rapidly become opaque and heat transfer through the glass is only by conduction. However, the conductivity constantly changes as the interlayers or gels react and degrade, making it impossible to model. Information on the anticipated unexposed face temperature and the resultant radiation can only be generated by physical testing, usually against the standard fire test. Under these conditions, the glasses continue to satisfy the integrity criteria for periods well in excess of that required and often into the next classification period. They do, however, remain substantially opaque with regard to transmissive radiation, and heat-flow into the compartment can occur as a result of convection from the unexposed face of the glass. Radiative heat can also be emitted from the hot surface rather than from the fire itself. Test data should be used for both surface temperatures and heat flux data.

Radiation-control glasses consist of monolithic unwired glasses where a metallic-based coating is applied to one surface of the glass. This coating can reflect radiation within certain wavelengths, thereby reducing the amount of heat that is absorbed by the glass, its average temperature, and the associated risk of slumping and lowers the heat emission from the unexposed face. It is critical that only the treated side faces the fire because, if the radiation penetrates the non-coated side, it can bounce back through the glass and increase the temperature of the glass itself.

11.2.6.9 Fire-resisting vertical glazed internal fire screens

The two most common forms of internal glazed screens are based upon either a tubular metal frame typically manufactured in steel, or a solid timber framework put together to create the required openings for glazing. The glazing installed can be uninsulating monolithic glass, either wired or clear, or multi-layered glass which satisfy the insulation criterion and meet the fire safety objectives.

Unless a thermal break can be introduced, steel/metal sections are normally conductive and do not satisfy the insulation criterion, whereas timber is naturally insulating. The framing around a glazed fire-screen represents a very small proportion of most fire-screen designs. Therefore, in a glazed screen containing insulating glass, the lack of insulation systems is not likely to represent a significant fire spread hazard, unless flammable materials are in intimate contact with the protected face. Similarly, life safety should not be compromised by the presence of hot members located between panes of insulated glass, unless personnel make contact with the unexposed face while escaping late in the fire, i.e. during a crowded and prolonged evacuation. If non-insulated members are used, the influence on the glass edge and pane behaviour might require a special glazing system.

Where there is a risk to safety from hot framing, either a thermal break should be incorporated, or the outer exposed faces should be clad with an insulating board to reduce the temperature of the unexposed face. This should also minimize distortion and reduce the risk of fracture due to bending.

The critical aspect in the design of a glazed fire screen is the selection of a compatible glazing system. Glazing systems designed for use with a timber frame which compensate for erosion of the substrate are not suitable for use in a metal frame where the ignition of the system could be induced by heat transfer to the unexposed face. The framing system should therefore be compatible with the glass/framing material combination.

Unless uninsulated glass is used, a timber framework is unlikely to cause a direct integrity loss, as the glass is more likely to fail first. When this occurs it is a result of radiation and convection attacking the arrises of the aperture framing members, e.g. glazing beads. Good design of these components can eliminate the problem by reducing the exposure of these surfaces to both forms of heat transfer.

As with many forms of non-loadbearing construction, no models can predict the performance of internal glazed screens. The behaviour of the framing members can be modelled (see Clause **12**) but, invariably, the designer relies on evidence generated in standard fire resistance tests. The influence of any change in the construction parameters on the fire resistance of the glazed element can be considered in accordance with Annex D. The relevant construction parameters are as follows:

- physical and thermal properties of framing materials (E, I);
- height/length of any framing members (E);
- section modules of framing members (E, I);
- centres of framing members (E);
- method of joining framing members (E);
- glass type (E, I);
- glass area (E);
- aspect ratio of pane (only important for some glass types and some durations, e.g. greater than 30 min) (E); and
- glazing system, including glazing beads and especially glass edge cover for some clear soda/lime composition glasses (E).

11.2.6.10 Fire-resisting curtain walling

Many applications require the external element to be resistant to fire penetration. This could be because of its position on a boundary, to prevent leap-frogging from floor-to-floor, to resist spread across an internal corner between two elements positioned either side of a compartment line, or to protect an external stairway. Where the façade is primarily glass, the glazing is typically retained in a curtain-walling system. This is a specialist construction where the glass is clamped in place with an external strip which is bolted into the main tubular framing members. The retaining pressure is generated by means of spacers and gaskets.

When considering the fire resistance of this form of construction a number of issues should be considered.

- The fire-resisting glass should be part of a double-glazed unit (DGU) for energy conservation/comfort reasons.
- The fire exposure is different from the outside and the inside, and the criteria of acceptance are different for each direction.

- The heat relaxes the clamping system, either as a result of expansion in the clamping bolts or thermal damage to the gasket.
- Weather-tightness is as important as integrity and fire resistance, which restricts the glazing materials that can be used.

The major problem associated with double-glazed units is the nature and performance of the secondary glass. If this is a non-fire-rated glass and is the pane first exposed to the fire it could shatter and fall away leaving the fire-resisting glass to perform its function. If, however, the fire-resisting glass is the first pane to be exposed it protects the non-fire-resisting glass, allowing it to remain in place for a prolonged period, particularly if it is laminated.

An added complication occurs when the non-fire-rated glass fails early as this loss could destroy the integrity of the glazing pocket. Therefore, detailing and choice of sealants is critical to the performance of the system.

It should not be assumed that a fire-resisting glass can provide the same performance as part of a double-glazed unit as it does when single glazed.

For external exposure conditions the standard time/temperature and pressure conditions are not appropriate. Fire attack consists primarily of radiation from the plume escaping from an adjacent building or openings within the same building, and this occurs when the pressure is atmospheric. Although there are a number of possible scenarios that represent external exposure conditions, there is always an absence of a positive pressure differential when considering the integrity of external envelopes. Similarly, when there are no combustibles in direct contact with the external system, the environment can accept higher temperatures and heat flux without risk.

Currently, there are no models capable of predicting the loss of clamping pressure, although knowledge of the temperatures could permit a rational analysis of what might happen.

Intumescent materials are used extensively as glazing systems for fire-rated glasses as they produce a cool edge and can generate clamping pressures. However, some of them have a propensity to leach or absorb water and they should therefore be specified with care and properly installed when used in the external envelope. For further guidance see the relevant trade association literature and the LPC Design Guide [40].

When predicting performance with respect to the identified criteria in accordance with Annex D, the following constructional parameters should be considered:

- physical and thermal properties of the curtain wall framing (E, I);
- section modules of framing members (E, I);
- centres of mullions (E);
- method of construction (E);
- glass type;
- fire glass (E, I);
- secondary glass (E);

- glazing sealant (E);
- air gap (E, I);
- aspect ratio of pane (only important for some glass types and some durations, e.g. greater than 30 min) (E);
- glazing gaskets (E); and
- edge cover (E).

This excludes any changes in the thermal or mechanical parameters which should be considered separately when a fire-engineered strategy is in place.

11.2.6.11 Fire-resisting glazed horizontal ceiling membranes

Many of the considerations in **11.2.6.9** relating to vertical glazed fire screens also apply to horizontal glazed membranes, or ceiling membranes. The unique aspect of horizontal monolithic fire-glass is its mode of failure, where the pane becomes plastic, loses stiffness and slumps out of the frame. When the pane is horizontal, or even sloping away from the vertical, the action of gravity causes the slumping to occur earlier and much more dramatically. Even if the glazing system is enhanced to provide additional clamping forces at the edges, the pane can continue to slump in the centre, resulting in a thinning of the glass, tearing and loss in integrity.

As there is a risk of accidental impact from falling debris it is customary to use a secondary safety glass in conjunction with the fire glass. This can deny cooling of the fire glass, as with double-glazed units (see **11.2.6.10**), depending upon which side of the fire-resisting glass it is installed.

Some insulating glasses are not suitable for horizontal applications and failure can occur earlier as a result of delamination of the interlayers due to gravity.

It should not be assumed that a glass which has demonstrated its fire resistance vertically, can provide a similar performance horizontally and evidence of performance should be sought, even if it is restricted to tests carried out under standard conditions. There are no models to predict the integrity or insulation performance of horizontal glazing systems.

Should the horizontal glazing element need to support any weight, then thick, loadbearing secondary glass is typically used. This can drastically influence the cooling of the fire-rated glass. As before, the performance of the system depends upon which pane of glass is initially exposed to the fire and there are no models for loadbearing capacity.

When considering the fire behaviour of horizontal glazing using the method in Annex D, the exposure and constructional parameters should be considered separately. In addition, the following constructional parameters should be taken into account:

- side to which any secondary glass is applied (E);
- angle of glazing with respect to the vertical (E); and
- magnitude of any load (R,E).

11.3 Maintaining the separating capability of elements or constructions

11.3.1 Fire stopping and linear gap sealing between separating elements

11.3.1.1 General

There are two types of seals:

a) void seals which seal gaps resulting from faulty assembly; and

b) linear gap seals which seal gaps incorporated for functional purposes.

Fire stopping covers gaps that occur in construction at the junction between elements and components, usually as a result of the tolerances needed to ensure ease of construction. Linear gap seals cover functional discontinuities created by the need to accommodate expansion or movement, or to reduce transmitted sound or heat, etc. Methods for fire stopping only need to address the maintenance of fire-resistance, whilst the linear gaps should meet these requirements and perform the function for which they have been introduced.

11.3.1.2 General consideration of the behaviour of gaps

Any sealant applied around imperfections of fit or functional discontinuity should provide the same level of performance in containing a fire as the structure or elements to which it is applied. The seal should provide the same level of integrity, or resistance to the passage of flame, smoke or hot gases but, in a fire engineering context, the need to satisfy the insulation criterion in the standard test methods might not be so important. This depends upon whether the area of the gap or the potential for materials to be stored in direct contact with the seal is a likely cause of fire-spread. There should be a limiting temperature rise on the seal as radiation ultimately causes fire-spread if no controls are applied. However, the level of insulation needed can be established by a risk assessment carried out on the protected enclosure.

Currently, there are no British Standard tests for evaluating linear gap or penetration seals. Most of the products have been evaluated against the time/temperature and pressure conditions of BS 476-20:1987 without control over the test arrangements and, therefore, the only evidence available for the fire-safety engineer could lack comparability. It is important when selecting products for sealing applications that the product is capable of accommodating the anticipated movement under normal environmental conditions as well as during a fire. The seal can be:

a) static in service and static in fire;

b) static in service and able to accommodate movement in fire; or

c) able to accommodate movement in service and in fire.

Furthermore, the seal should be capable of providing the required level of fire resistance in any of the positions that might be found in practice, i.e. fully compressed and fully relaxed. The extreme conditions should be established in respect of magnitude and direction of the movement and it might require tests at both extremes of use, unless one of the extremes can be demonstrated to represent the worst case. Furthermore, this might be deemed necessary to pre-cycle the seal prior to test.

11.3.1.3 Integrity of gaps

Many materials meet the performance requirements at ambient temperature and can maintain integrity when small samples are tested under standard fire test conditions. However, their suitability should be reviewed in the light of the deflections anticipated for the size of element to which they are applied. Cementitious materials are rarely suitable if large deflections in the hot state are to be met. Materials should be able to compensate for erosion of one or both of the opposing substrates. Fibrous materials, such as mineral rock fibre, can accommodate differential movements between elements, but they might not be able to compensate for any enlargement of the aperture. In these circumstances, intumescent-based materials are more suitable. A combination of the two materials is suitable for many applications [39].

The orientation of the seal is critical. Non-engineered seals could drop out of horizontal gaps with vertical entry or from vertical gaps with vertical entry. Evidence of performance should be examined to show that it has the correct characteristics, which can include stickability. Any specification should clearly identify the orientation of the joint.

11.3.1.4 Insulation of gaps

Depending upon the risk analysis, it might not be necessary to fully conform to the conventional criteria for insulation. However, mineral fibre or intumescent-based seals are more likely to satisfy the criterion than cementitious materials. The thermal properties of some of the materials that can be used as a component of fire stopping or linear gap sealing are given in Annex B.

11.3.1.5 Curtain wall-to-floor slab seals

Curtain wall-to-floor slab seals present a special problem as the external curtain walling element is not required to be fire-resisting, unless called upon to satisfy boundary requirements or means of escape. The seal is usually provided to prevent premature internal fire spread. Since the curtain walling is not fire-resisting in these circumstances, it should not be relied upon to retain the seal in position.

Successful fire-sealing of voids behind curtain walling systems depends upon the nature of the curtain walling system and its reaction to any fire attack. Systems incorporating glazing, aluminium and, to a lesser extent, steel-faced composite panels filled with combustible insulation, can shatter, melt or buckle. This allows the passage of fire to by-pass the seal, unless the design and installation of the system has taken these possibilities into account.

When the external components are not fire-rated they are likely to exhibit large deflections. A resilient seal should be used if its contribution is to be maximized until the failure of the façade.

11.3.2 Service penetrations passing through elements or protection systems

11.3.2.1 General

Fire containment can be influenced by penetration of services through separating elements or any protection system applied to structural elements. These can include:

a) pipes (metal and plastic);

b) cables (metal and fibre optic cored); and

c) metal ducts for heating and ventilation (H & V), air conditioning and smoke and heat exhaust (SHEV);

Reinstatement of the fire separating element should be achieved by means of appropriate construction materials designed for the intended application and capable of withstanding the anticipated thermal exposure conditions.

11.3.2.2 General consideration of the behaviour of services

Inadequate fire sealing of services has been shown to be a major potential cause of breaches in compartmentation, and fire spread has occasionally been made worse by the fire exploiting voids and holes. For this reason all service penetrations should be properly fire-stopped. While various ad hoc materials have been used for fire stopping or penetration sealing, the potential for such systems to be compromised or deteriorate during the life of the building should be recognized and measures should be introduced to prevent this. This can be accomplished by:

a) sealing the penetration with products designed for fire stopping which can be removed to accommodate additional services with minimum levels of re-instatement; or

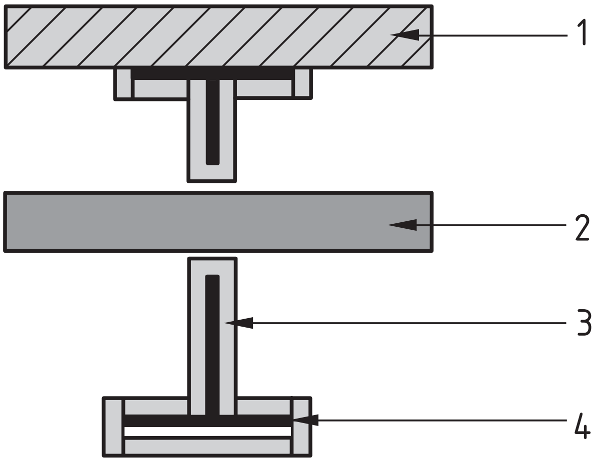
b) building a strict inspection and maintenance regime into the management section of the fire manual to prevent unprotected apertures remaining unsealed.

All penetration seals should be able to cope with the anticipated cold loads, hot loads and fire-induced movement in either the element or the penetrating service(s) ([37] and [39]).

Apart from penetrating separating elements, services sometimes have to penetrate structural members, such as steel and timber beams, as well as any fire protection associated with them. For metal pipes, conduits or large diameter cables that penetrate fire protection cladding installed around a structural member, it is insufficient to seal the service into the protection. This is because heat is conducted along the service into the space created to protect the beam, and possibly into the beam itself. It is therefore necessary to protect the element using the type of detailing shown in Figure 22.

The influence of distortion of the service on the fire protection material should be considered and any additional restraint specified as necessary. Equally, the influence of distortion of the element on the service should be accommodated.

Figure 22 **Typical detail showing protection to a floor beam with a service penetration**



Key

1	Slab or floor	3	Fire protection
2	Service	4	Steel beam

11.3.2.3 Integrity of services

The integrity of an unsealed penetration is zero, hence the need to introduce sealing. Integrity failure of a sealed aperture occurs if:

a) the service melts or falls out;

b) the sealing material falls out due to the action of the fire or movement in the service or the substrate;

c) the sealing material falls out due to erosion of the substrate, or possibly the service;

d) the sealing material is eroded or consumed by the fire due to the heat conducted along the service;

e) the sealing material on the unexposed face ignites as a result of conduction;

f) the service, or any covering applied to the service, ignites due to heat conduction or heat flux.

The selected service sealing system should prevent or accommodate any of these events as appropriate to the particular services/substrates.

Whilst smoke containment is not a conventional criterion, the ability to restrict the spread of smoke is important. Impermeable seals provide enhanced fire separation at all stages of a fire compared to permeable materials.

11.3.2.4 Insulation of services

Any service penetrating a fire barrier, whether it is a pipe, cable or duct, can conduct heat to the non-fire exposed face. Depending upon the nature of the service or the criticality of the environment on the protected face, this could provide an ignition source or present a hazard in other ways. The quantity of heat conducted to the unexposed face is related to:

a) the conductivity of the service, any supporting frame and possibly the supporting/associated construction;

b) the thermal inertia of the services;

c) the thickness of the element and thermal capacity of the sealing material;

d) the contents of the pipe, if a fluid and full;

e) high temperature behaviour of any applied insulation on the non-fire exposed face of cables or pipes; and/or

f) the conductivity and erosion resistance of the seal.

The acceptable temperature rise on the unexposed face of a separating element should be established by a risk assessment and take into account how current and future uses of the services might influence heat flow along the systems.

For relatively simple penetrations, some thermal models can provide a fairly accurate estimate of the unexposed face service temperature.

11.3.2.5 Pipes and associated seals

There are no models available for predicting the complex interaction between the service, the sealing system and the associated construction in respect of the time for which it should satisfy integrity criteria. If plastic pipes or heavily insulated metal pipes are installed, some form of intumescent closure device should be incorporated to close off any void left by the degradation of the pipe or insulation. Test evidence should establish the suitability of any selected system. However, this might need to be reviewed with respect to variation in the thermal exposure conditions using the extended application approach. The various insulating materials have different degradation or softening/melting temperatures and the evidence should relate to the particular materials being considered. Orientation is also significant.

For metal pipes, the sealing system should accommodate any thermally-induced longitudinal or rotational movement that might occur and this should feature in the analysis. Where multiple penetration services exist, the difficulty of sealing small gaps between adjacent services or the service and the aperture should also be considered in the selection of suitable sealing systems. There are two situations where apertures might need to be sealed:

- where the apertures are small and can be sealed by a single sealing material (a small penetration); or
- where the aperture is large and a primary bulkhead seal is used in conjunction with sealants (a large penetration).

A method suitable to seal the aperture size and service combination should be selected [39].

When using the extended application approach to establish the suitability of a sealing system for a service penetration, the relevant constructional parameters and the criteria they influence are:

- orientation (E);
- size and shape of penetration (E);

- properties of construction being penetrated (E, I);
- thickness of construction being penetrated (E, I);
- pipe diameter(s) (E, I);
- pipe material(s), including plastics (E);
- pipe end condition (open/closed) (E, I);
- pipe contents (liquid/gas) (E, I);
- mix and distribution of pipes (E, I); and
- characteristic properties of any service supporting racks/trays (E).

Exposure and mechanical parameters should be considered in addition to the constructional parameters specified.

11.3.2.6 Cables and associated seals

The issues associated with the penetration of cables are similar to those of pipes (see **11.3.2.5**), except for the added complication of the insulation melting onto the cable and igniting on the unexposed face. Metal conductors rarely melt unless they are short circuited. However, optical fibre data cables can melt-out and the sealing system should take this into account.

In the absence of modelling, which may provide an estimate of the unexposed face temperatures, test evidence should be obtained as the primary mechanism for evaluating the suitability of the system, adjusted as necessary to take into account any change in the thermal exposure conditions. This evidence should relate to cables and cable insulation of a similar size and with similar thermal characteristics to those proposed for use [39].

When establishing the criteria in accordance with Annex D, the relevant constructional parameters and the criteria they influence are:

- orientation (E);
- size and shape of penetration (E);
- properties and thickness of construction being penetrated (E,I);
- cable diameter(s) (E,I);
- conductor diameter(s) (E,I);
- nature of insulation (E,I);
- conductor material (E);
- number of cables (E,I);
- mix and distribution of cables (E,I);
- characteristics/properties of supporting ladder or tray (E).

Exposure and mechanical parameters should be evaluated in addition to the constructional parameters identified.

11.3.2.7 Metal ducts and associated seals

Metal ductwork can compromise fire separation by:

- providing a route through which hot gases can circulate;
- penetrating the element and leaving a gap; and
- conducting heat along the body of the duct.

The first of these can be solved by installing dampers in ducts at any point where they traverse a separating element (see BS 9999). Dampers are operated by a fusible thermal link or by an actuator mechanism. As there is always a risk of a damper failing due to a lack of operational checks, a motorized damper is preferable as it enables automated checking. Also, if the risk assessment shows a benefit in having the damper closed-off by means of the detection system, a motorized damper enables an earlier response.

If the duct is part of a heat and smoke ventilating system (SHEV) any damper should be motorized as the duct is intended to exhaust hot gases. The hot gases flowing through the duct cause the metal temperature to rise to the point at which the gases reach the limiting design temperature, set by the fan design. These temperatures are usually above the criterion specified in BS 476-24, ISO 6944 and are typically a maximum of 200 °C. The designer should perform a risk assessment, focusing on the environment surrounding the duct, to establish whether the smoke design temperature could cause ignition on the protected side of any separating element. If this is possible, the duct should be insulated accordingly and either the adjacent area kept free from combustible materials or the rise in smoke temperature restricted to a safe level.

When a non-insulated metal blade damper is used, the damper radiates significantly, with the radiation falling onto the internal surfaces of the duct and raising the temperature. In such cases, the duct should be insulated for a sufficient distance beyond the barrier so that the metal duct temperature does not become a hazard. Alternatively, a non-metallic damper should be used or a metal damper insulated.

Service ducts that penetrate a fire wall should not be fire stopped using typical methods. Filling a void around a thin steel walled duct raises its temperature and causes the distortion to occur within the confines of the wall or floor. This leads to an even greater gap developing due to the change in cross-sectional area of the duct, negating the fire stopping unless it has been selected to respond to this. Fire stopping around ductwork should only make use of materials that have been tested as duct seals and have been quantified as suitable through the extended application methodology form of assessment (see **11.4**).

The approved fire stopping method should take into account probable thermal movement in the element and/or the duct. For example, a system that is suitable for installing in a blockwork wall should not be assumed suitable for a steel stud wall.

A metal duct can conduct heat from the hot to the cold side of the separating element. Depending upon the fire scenario and the resultant heating, the ductwork should be insulated for a distance proportional to the risk on both, or either side of the barrier. This can cause local overheating of the duct and should be taken into account. Additional stiffeners or thicker metal might be required to prevent the duct cross-section from collapsing.

There is no way of modelling these phenomena and the maintenance of the fire separation can only be quantified using the critical parameter, extended application methodology. In this case, some of the critical constructional parameters are:

- thickness and characteristic strength of the metal used for the duct (E, I);
- the cross-sectional area of the duct (E);
- the nature of any damper (E, I);
- the orientation of the duct (E, I);
- the nature of the seal at the interface with any separating element (E,I);
- the temperature and flow of any hot gases transported by the duct (SHEV only) (E,I);
- the physical characteristics and thermal behaviour of the separating element (E);
- the support/restraint on the duct (E, I).

This list does not include the thermal and mechanical parameters that evolve from an analysis of the actual exposure conditions.

11.4 The extended application process

When establishing the application of the element it is necessary to determine the variations in the exposure parameters (i.e. thermal and mechanical) and the constructional parameters between what was fire tested and what is to be installed. These parameters should be considered in turn to see whether the variation has a beneficial or negative effect on the previously established performance. The three methods by which the extended application can be devised are (see BS ISO/TR 12470):

a) the application of rules (if they exist);

b) fire engineering calculations; and

c) engineering judgements.

Rules typically address the parameter of size and rarely consider exposure or mechanical parameters other than in cases where the variation is unequivocally beneficial. On the other hand, engineering calculations can address the influence of a change in the thermal or mechanical parameters, although usually only for simple, homogeneous elements. However, for composite constructions, they rarely address the issue of size or constructional variations. Engineering judgement and analysis can address any influence, including the effect of multiple influences, but are rarely able to quantify the output of the process, although every attempt should be made to do. It is important that the judgemental analysis is made by a competent person(s) with the relevant materials knowledge, and an understanding of the fire exposure conditions and anticipated responses. This still leaves the problem of consistency, even between expert judgements made by equally competent person(s). The methodology in Annex D should bring uniformity to the process by specifying that:

- all thermal and mechanical parameters that might vary as a result of the proposed change, if any, should be identified;
- the components of a construction which might vary either directly or indirectly as a result of the proposed change should be identified;
- all constructional parameters that might change as a result of a change in that component should be identified;

- for each parameter, the factors that might change should be identified;
- for each factor, the factor influences on the relevant criteria should be determined.

The common variations in the thermal and mechanical conditions and the constructional parameters that are likely to be encountered are as follows:

a) thermal and mechanical parameters, including:
   1) the load on loadbearing elements;
   2) the boundary conditions applied to the element at its ends or edges;
   3) the thermal action, e.g. BS EN 1363-2, a parametric fire curve or a change in the number of faces exposed;
   4) the pressure differential due to height or environment;
   5) the mechanical impact (if appropriate).

The fire engineering analysis should establish where these parameters vary from those used in the standard test methods, based upon ISO/TR 834.

b) constructional parameters, including:
   1) the construction of the element (e.g. thickness), the method or the materials used;
   2) any change in the dimensions of the element (normally larger) from that tested to that under consideration;
   3) the introduction of, or any variations to, an aperture in a separating element;
   4) the orientation of an asymmetric element in respect of the fire exposure;
   5) the orientation of an element with respect to the fire, e.g. a change from vertical to sloping.

In due course, extended application standards will be available for all of the individual elements under consideration and these should be used to identify the relevant parameters.

All the individual elements under consideration can be assessed in accordance with Annex D. As each construction differs, it is impossible for all factors to be pre-identified and the designer should draw up a relevant list.

The obvious influences should also be listed, but it is unlikely that all influences are contained in Annex D, particularly those that occur due to the exposure conditions following a fire engineering analysis.

For each influence, if there is evidence available to quantify the variation, by secondary test evidence, historical data, ad hoc tests or small-scale tests, calculation should be considered or a qualitative analysis should be made by expert judgement.

The expert analysis of the influence on the result should be performed:

- initially on a factor by factor basis; and
- on a global basis where the interaction between the influences of factors is taken into account, i.e. do they complement or contradict each other?

11.5 Behaviour of non-fire-resisting separating elements

Where enclosure boundaries are imperforate under ambient conditions but have no readily determinate resistance to fire spread, i.e. are unproven by way of engineering analysis or standard tests, they can be allocated a notional ability to restrict fire spread. It can be assumed that all well-installed enclosing surfaces can delay the spread of a growing fire. However, once flashover occurs (see PD 7974-2), the boundaries should be assumed to resist the fire and remain in place, satisfying the integrity criteria of the standard fire resistance test specified in BS 476-20 for the notional periods in Table 13.

Table 13 **Notional period of fire endurance for which imperforate condition can be assumed for unproven elements subject to fire exposure**

Boundary type	Notional period of fire endurance min
Gypsum or calcium silicate board dry lined, steel or timber stud partition	15
Gypsum or calcium silicate board under drawn, timber joisted ceiling	15
Lathe and plaster ceiling on timber joists	5
Suspended lay-in ceiling	5
Annealed or toughened soda/lime unwired glass in a fixed partition or window of timber or metal	0 [A)]
Integral wired soda/lime glass in fixed partition or window of timber or metal	10
Non-integral, resin bonded laminated wired glass in window of timber or metal	5
Timber or metal doorset glazed with annealed or toughened soda/lime glass	0 [A)]
Timber or metal doorset glazed with integral wired soda/lime glass	10
Flush timber doorset, hollow core	5 [B)]
Flush timber doorset, solid core	10 [B)]
Panelled timber doorset >35 mm thickness with panels >10 mm thick	10
Panelled timber doorset <35 mm thickness with panels <10 mm thick	5
Hinged or pivoted flush steel doorset with insulated core	10
Hinged or pivoted flush steel doorset with hollow core	5

A) The guaranteed protection is less than 5 min, but is unlikely to be as bad as an unprotected opening.
B) Only when closed.

In the case of doorsets, the notional durations should only be applied to assemblies that are normally closed in use, either automatically or as part of a strategy, or where management control ensures their closure in the event of a fire.

For any unproven boundary element, including those in Table 13, it might be possible for competent persons to quantify the time of penetration by fire more accurately using numerical engineering tools.

12 Analysis of mechanical response of loadbearing structural elements and frames

12.1 Concepts

12.1.1 General

The properties of a material change with respect to temperature and, as a result, the behaviour of structural elements also changes with temperature. Typically, several or all of the following can occur at fire limit state:

- materials lose stiffness and strength with increasing temperature;
- materials expand as their temperature increases;

  NOTE Restrained thermal expansion can lead to induced thermal stresses and non-uniform temperatures within an element resulting in induced thermal curvature.

- for some forms of construction, structural material can be lost during the heating process (for example, due to spalling in the case of concrete and charring in the case of timber).

All of these can lead to reduced structural performance and/or failure. Clause **12** provides guidance on how to assess the performance of structural elements and assemblies at elevated temperatures.

12.1.2 Fire loads

Fires in buildings are relatively rare so, for design purposes, they can be viewed as a form of accidental loading. Therefore, in order to make the likelihood of failure due to fire similar to the likelihood of ultimate limit state behaviour, the construction industry approach is to associate partial safety factors with the various dead loads (G_k) and imposed loads (Q_k) on a building to calculate the overall design load ($E_{d,t}$):

$$E_{d,t} = \gamma_G G_k + \psi_{1,1} Q_{k,1} + \sum \psi_{2,i} Q_{k,i} + \sum F_d(t) \qquad (67)$$

The coincidental application of accidental loads is very unlikely and, accordingly, the fire design case attracts differing partial safety factors. Table 14 shows the values that are included in equation 67. In the absence of detailed information, it can be safely assumed that only 70% of the ambient design load is active in the fire situation. Loads in fire conditions should not be confused with "fire load", which is the combustible contents within an enclosure (see PD 7974-0).

However, an alternative (and more robust) approach is to assess the consequences of a fire under different appropriate load conditions and assess whether the risk is acceptable. This approach might be preferable in situations where the consequences of a combined event could be particularly high or the likelihood of combined events is not sufficiently low to be negligible, for example wind and fire in high-rise buildings, or fire and snow in countries where annual snow fall is high. In the case of a high rise building, due to the combination of the number of floors and the height of the building, the likelihood of a fire occurring at the same time as high winds is much greater than for low rise buildings, so the consequences of failure are much greater. In such instances it might be appropriate to assess the risk of a fire under both low and high wind conditions to ensure that the risk associated with both events is acceptable.

Table 14 **Partial safety factors for loads (illustrative)**

Loads	ψ_1	ψ_2
Dead loads	1.0	—
Imposed loads on:		
permanent items, e.g. fixed partitions, plant	1.0	1.0
Live loads (BS EN 1991) on:		
escape stairs and lobbies	1.0	1.0
residential	0.5	0.3
offices	0.5	0.3
assembly	0.7	0.6
commercial	0.7	0.6
industrial and storage	0.9	0.8
roofs	0.0	0.0
Live loads on:		
escape stairs and lobbies	1.0	1.0
all other areas	0.8	0.8
Snow loads	0	0
Wind loads on:		
buildings less than 8 m high	0	0
buildings taller than 8 m	0.33	0.33

12.1.3 Increased loads

In evaluating the mechanical response of boundary enclosures to fire, increased loads should be considered, including:

- loads due to fire-created pressures, e.g. a wall bounding a fire enclosure is likely to be exposed to a pressure of approximately eight Pascals per metre of height on the exposed face;
- impact loads from collapsing fire affected elements, including service plant;
- impact of fire-fighting hose streams;
- loads applied due to restrained thermal expansion;
- loads applied due to shrinkage or thermal contraction;
- loads applied due to the deflection of boundary elements creating load paths where previously there were none, e.g. deflection of beams onto non-loadbearing partitions which might also need to be considered in terms of its effect on separation;
- loads applied due to failure of building contents to remain self-supporting, e.g. ducts and services.

12.1.4 Reduced loads

In evaluating the mechanical response of boundary enclosures to fire, reduced loads should be considered including loads removed from timber floors by collapse of the ceiling or its components.

Detailed guidance on quantitative techniques for determining the response of loadbearing elements of various materials to fire exposure is available [44].

12.2 Acceptance criteria

12.2.1 Stability

During a fire, the elements of structure might be required to maintain overall stability and/or contain the fire. Therefore, in some instances it is sufficient to consider stability only, but in others stability, integrity and insulation should be considered.

Limit state design can be applied to determine the mechanical response of structures to fire, on the basis that time to failure is the time where the following condition is no longer satisfied:

$$R_{d,t} \geq E_{d,t} \tag{68}$$

12.2.2 Integrity for acceptance criteria

Thermal actions can cause significant deformations in structural elements and, because the fire limit state is an accidental load case and limit state design can be applied, much larger deflections/deformations are allowable compared to those under serviceability limit state design. However, whilst such large deformations might not cause a stability failure, they can lead to integrity failures of structural elements such as floors or walls.

BS 476-20 provides acceptance criteria for integrity failures based on whether collapse or sustained flaming occurs on the unexposed face of the separating element. However, most methods for the assessment of structural performance do not include a means of determining whether cracks might develop that are large enough to allow sustained flaming on the unexposed surface. Therefore, it is often necessary to consider alternative acceptance criteria to protect against integrity failures. Typically, these include deflection, strain or curvature limits. Whatever criterion is adopted, it should be appropriate for the element in question and justifiable.

12.2.3 Insulation for acceptance criteria

The need to meet insulation requirements is usually addressed under structural response as a separating element as opposed to structural response as a structural element. However, the performance of the structure could have an impact on its insulation characteristics. For example, excessive deformations in the structure could lead to material degradation or a change in the dimensions of a separating element, which in turn could lead to an insulation failure. In the case of a concrete floor slab, deformation could lead to concrete cracking on the lower surface which can reduce the effective thickness of the slab even if the cracks are not full depth cracks.

12.2.4 Compatibility

There are situations where the response of structural elements in fire impacts on the performance of non-structural elements. For example a deflecting floor slab can cause failures in walls or partitions above or below the slab. In such circumstances it is important to ensure that:

- reliance is not placed on the performance of the non-structural element in fire conditions;
- the non-structural element can accommodate the deflections or forces generated by the structural element; or
- appropriate acceptance criteria are imposed to insure that the response of the structural elements do not adversely affect the performance of the non-structural element.

12.3 Methods for determining structural response

12.3.1 General

There are three recognized methods for determining the structural response of elements exposed to fire:

- testing;
- analysis of the structure under the assumption that it can be treated as a series of isolated elements; and
- analysis of the structure as a structural frame or sub-frame.

Many aspects of these approaches are material independent. It should be recognized that many of the methods and considerations given in **12.3.2** below are material specific.

With all methods, careful consideration should be given to the fire performance of connections/joints and the impact of any unusual construction details.

12.3.2 Empirical data from testing

12.3.2.1 Approach

Historically, the fire performance of structures has been determined by fire tests defined and controlled through adopting the procedures described in the test standards but, in some cases, the tests are bespoke. The data taken from testing are used to compile prescriptive requirements for use in design to ensure that specific temperatures within the structural member are not exceeded. Some methods allow the designer to account for the utilization of the member, but others assume that the member is loaded to its full design capacity.

12.3.2.2 Validity

When using data from testing, the designer should ensure the tests are representative of the built structure in terms of the fire conditions, structural assembly and sometimes the environment.

The fire performance of some forms of construction is particularly sensitive to the fire conditions. For example, the temperature of unprotected steelwork follows the fire temperature much more closely than protected steelwork or concrete, and spalling in concrete is more likely to occur in fires with high-temperature release rates. Therefore, before using design guidance derived from tests, it should be ensured that either the form of construction is not sensitive to the fire conditions, the test fire conditions are suitably representative of the real fire conditions, or the design guidance is modified to account for the real fire conditions.

Great care should be taken when modifying design guidance.

It is not practical to test a structural element in all of its potential configurations. Similarly, due to the dimensional constraints of test furnaces, it is not possible to test the performance of large structural members. Therefore, it is sometimes necessary to extrapolate test results for use in design. When using the results of test data in design, it should be ensured that the test configuration is suitably representative of the real building configuration and that any extrapolation is appropriate (see Annex D).

12.3.2.3 Considerations

Most design methods derived from testing give simple pass/fail criteria. This can lead to the perception that, provided the recommendations are followed, fire spread or structural failure is prevented. However, all design methods based on testing have implicit acceptance criteria which deliver a corresponding performance standard. For example, BS 476-20 stipulates a maximum deflection of span/20 for loadbearing beams, which means that beams designed in accordance with the prescriptive requirements of BS 476-20 could reach deflections of up to span/20 under certain fire conditions. Similarly, conforming with BS 476-20 for insulation does not mean that fire spread is prevented, but that the average temperature rise of the non-fire side of the separating element has not to exceed 140 °C (or +180 °C max) when exposed to the standard fire curve. Therefore, when using design methods based on testing in the context of BS 7974, the acceptance criteria and their implications on design should be clearly understood.

Care should also be taken when interpreting fire resistance standards in relation to stability, integrity and insulation. For example, if it is stated that a steel beam is protected to a 60-minute standard, this is likely to be in terms of stability only. Therefore, if this beam is also to be used as part of a compartment wall, additional protection might be required to achieve the insulation requirements.

12.3.3 Simplistic calculation methods

12.3.3.1 Approach

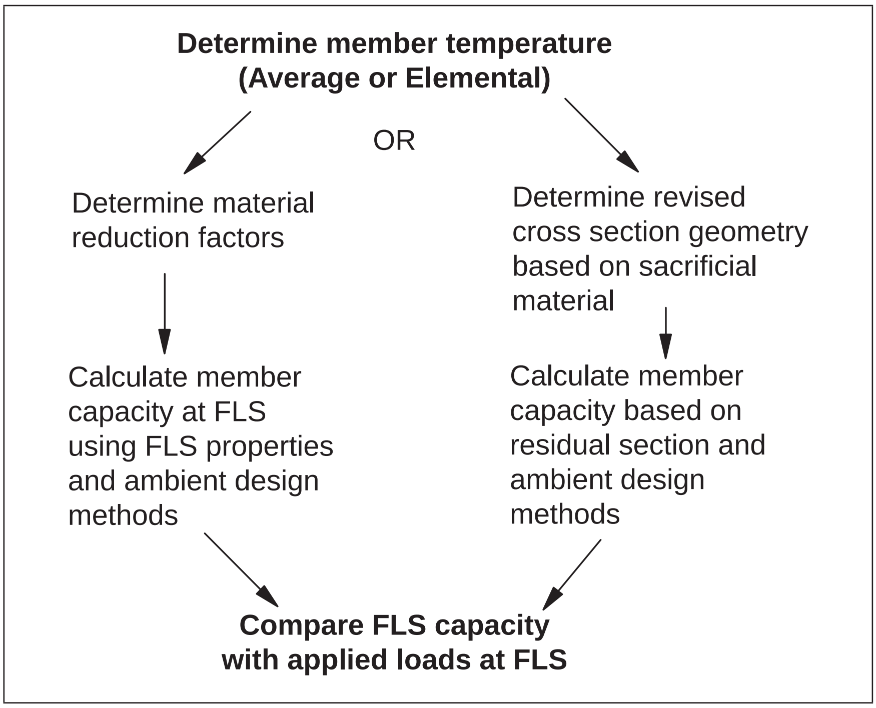
As with design for ambient conditions, many structural assemblies can be considered as a series of individual, isolated members at the fire limit state. This is usually the simplest approach and, in most cases, adopting such an approach provides conservative solutions. The analysis methods vary between codes, standards and forms of construction, but the majority are based on assessing the member's ability to support the applied loads at fire limit state.

Typically, a designer should follow one of the design paths illustrated in Figure 23.

There are some cases where the approach is slightly different and these are discussed within the material-specific sections.

Most methods for determining the fire performance of isolated elements are strength-based and do not consider thermal expansion or creep. These are acceptable assumptions as, typically, isolated members are unrestrained or the expansion is only restrained by the applied load, and fire durations are sufficiently short to ignore creep.

Figure 23 **General approach to structural fire safety design**



12.3.3.2 Validity

All methods that accurately determine the member's fire-limit state properties over the cross-section when subjected to realistic fire exposure conditions are valid. However, some methods are based on the standard fire curve and are only valid when this is representative of real fire conditions.

12.3.3.3 Considerations

Treating structures as isolated elements tends to deliver conservative results as this approach ignores the alternative load paths and beneficial contributions from adjacent members that are often present in structural assemblies. However, there are occasions when treating a structural assembly as a series of isolated elements does not yield conservative results. A typical example would be where failure is induced at relatively low temperatures as a result of restrained thermal expansion, inducing high compressive forces in a member. This might occur is in a steel truss or space frame where a member is subjected to localized heating.

12.3.4 Advanced calculation methods

12.3.4.1 General

In many instances, the assumptions made within the simplistic calculation methods are not sufficiently accurate and analysing the structural frame as a series of individual members is not sufficiently representative of the real behaviour of the structure at fire limit state. The structural performance can be shown more accurately by considering:

- the assembly's ability to redistribute loads via alternative load paths (for example columns acting in tension);
- continuity; and/or
- alternative structural modes (for example tensile membrane action).

However, it is also possible that the performance of a structural element is reduced when included as part of a structural assembly. The deformation of one structural element could cause failure of another, for example, deflections of a floor could cause failure of an adjacent wall, or restrained thermal expansion or contraction in the cooling phase could induce premature failure of a structural element or its connections.

12.3.4.2 Approach

Advanced methods based on fundamental physical behaviour and structural mechanics provide an alternative to the simplistic calculation methods often included in design codes and standards. Advanced methods can treat structures as isolated elements, connected elements, continuous members, 2D or 3D sub-frames or entire buildings.

NOTE For some frames it is possible to analyse the entire structure, but often this is not practical due to the size and complexity of the structure and, therefore, it is necessary to analyse a representative sub-frame or series of sub-frames.

Whichever of the structural forms is adopted, the analytical methods should address the following.

- Advanced calculation methods for mechanical response should be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature.
- The effects of thermally-induced strains and stresses due to temperature rise and temperature differentials should be considered.
- The model for mechanical response should also take account of:
  - the combined effects of mechanical actions, geometrical imperfections and thermal actions;
  - the temperature-dependent mechanical properties of the material;
  - geometrical non-linear effects;
  - the effects of non-linear material properties, including the unfavourable effects of loading and unloading on the structural stiffness.
- Attention should be given to whether thermal creep is explicitly considered.
- Attention should be given to whether material-specific phenomena, such as concrete spalling or charring of timber are explicitly considered.
- In the analysis of individual members or sub-assemblies, the boundary conditions should be checked and detailed to be representative of the restraint that would be provided by the surrounding structure.

The process for conducting advanced calculations typically consists of:

- selecting/defining a representative frame/sub-frame;
- selecting the analysis method;
- ensuring that the method is appropriate and validated for its intended use;
- carrying out the analysis;
- conducting a sensitivity study.

12.3.4.3 Finite element analysis

There are some analytical methods for analysing simple structural frames, but for most structural frames the complex interactions necessitate the use of finite element or finite difference analysis. These methods require that the structure is defined as an assembly of discrete elements. Typically, beams and columns are represented as a series of line elements and shell elements are used to define slabs and walls. However, beams and columns can also be represented as an assembly of shell elements, which is useful if predicting local behaviour is important. There are two types of finite element or finite difference software, each with its own merits:

a) General purpose software is written to have an extremely broad application, both structural and non-structural. Most general purpose software packages include elements developed for common forms of construction, but they also include the ability to develop user-defined elements. The primary advantages of general purpose software are that they are capable of modelling all forms of construction (providing the appropriate elements have been developed), validation is often extensive, and they are typically well supported by their commercial developers. However, the process of describing the structure can be complex and time consuming, analysis times can be long and the software is not always validated for its intended use.

b) Purpose-written software packages are developed specifically for their intended use. They work in the same way as general purpose software with the exception that the various element types are defined for a specific application. Therefore, well written, bespoke software of this nature should be simple and quick to use with more economic run times, and any validation is appropriate for the intended use. The primary disadvantage is that the range of application is often limited.

The general principles of finite element models are described in the Institution of Structural Engineers Guide to the advanced fire safety engineering of structures [45].

12.3.4.4 Validity

All advanced calculation methods (including finite element analysis) are still an approximation of real building behaviour. For finite element analysis, the accuracy of the representation is a function of:

- the sophistication of the element formulations;
- the adequacy of the selected sub-frame to represent overall building behaviour;
- the finite element mesh density; and
- any boundary conditions that are applied.

One of the most important considerations in the validation of finite element analysis is to ensure that the software has been validated for its intended use. Validation is usually demonstrated by comparing the results generated from the software against test data. Therefore, it should be ensured that the test was sufficiently representative of the scenario and that any differences are justifiable.

12.3.4.5 Considerations

Finite element analysis is a complicated procedure and it is important that the designer/modeller has sufficient knowledge and experience to develop the finite element model, conduct the analyses and interpret the results. The modeller should have a complete understanding of the capability of the software including any embedded assumptions or approximations.

Most finite element analysis software does not include predefined acceptance criteria. Therefore, careful consideration should be given to the selection of appropriate acceptance criteria. Typical acceptance criteria would include maximum allowable strains, maximum allowable deflections or maximum allowable curvature.

The mechanical response of a structure can be very sensitive to thermal distribution within the structure. Therefore, it is important to ensure that the temperature distributions within the structure are sufficiently accurate for all of the elements for the duration of the analysis. The fire should therefore be defined accurately and the heat transfer and thermal analyses should be accurate. This is particularly important to structures that are likely to perform differently under different fire conditions.

It is difficult to model localized behaviour, including reinforcement fracture and connection behaviour, in finite element analysis. Therefore care should be taken that:

- the model is sufficiently sophisticated to predict local behaviour;
- appropriate acceptance criteria (such as deflection limits) are applied so that local failures do not occur; or
- local failures do not a significantly impact on the overall performance of the structure in terms its functional requirements.

12.3.5 Concrete

12.3.5.1 General

Concrete loses stiffness and strength as its temperature increases. However, concrete has a relatively low thermal conductivity (up to 50 times lower than steel), and heat transfer through concrete elements is relatively slow compared to typical building fire durations. Therefore, only those parts of the element which are near to the exposed surface(s) of concrete sections lose significant stiffness and strength. The depth of loss of material stiffness and strength depends on the exposure period.

As with its ambient material properties, the variability of the temperature-dependent material properties of concrete can differ greatly. This is reflected by applying partial safety factors to the ambient characteristic strength which is generally acceptable for fire limit state analysis, but it might be appropriate to consider additional sensitivity studies to ensure a reliable solution. The material properties at fire limit state are a function of the concrete type (normal weight or lightweight), the characteristic strength, the moisture content and the type of aggregate.

In addition to the variability of concrete material properties, concrete can be susceptible to spalling. Therefore, it should either be shown that the likelihood of spalling is negligible or the analysis method adopted should account for spalling directly or through appropriate sensitivity studies. Many parameters determine a concrete member's susceptibility to spalling, including the rate of imposed heating, moisture content, applied load levels, degree and type of restraint, concrete strength and permeability. Particular care should be taken in the extrapolation of standard fire test results to applications where the design fire conditions could be more severe. The likelihood and consequences of spalling should be carefully considered for high strength concretes, i.e. cube strengths in excess of 60 MPa. A summary of spalling is given in Connolly [46].

Since concrete has a good inherent fire resistance, the design of concrete buildings is not traditionally governed by fire limit state design. The most popular method of design is to use tabulated data based upon tests. The tables provide minimum cross-sectional dimensions and depth of cover to reinforcement. However, for high fire resistance periods or structures where the depth of cover is small, advanced methods can deliver more economic solutions.

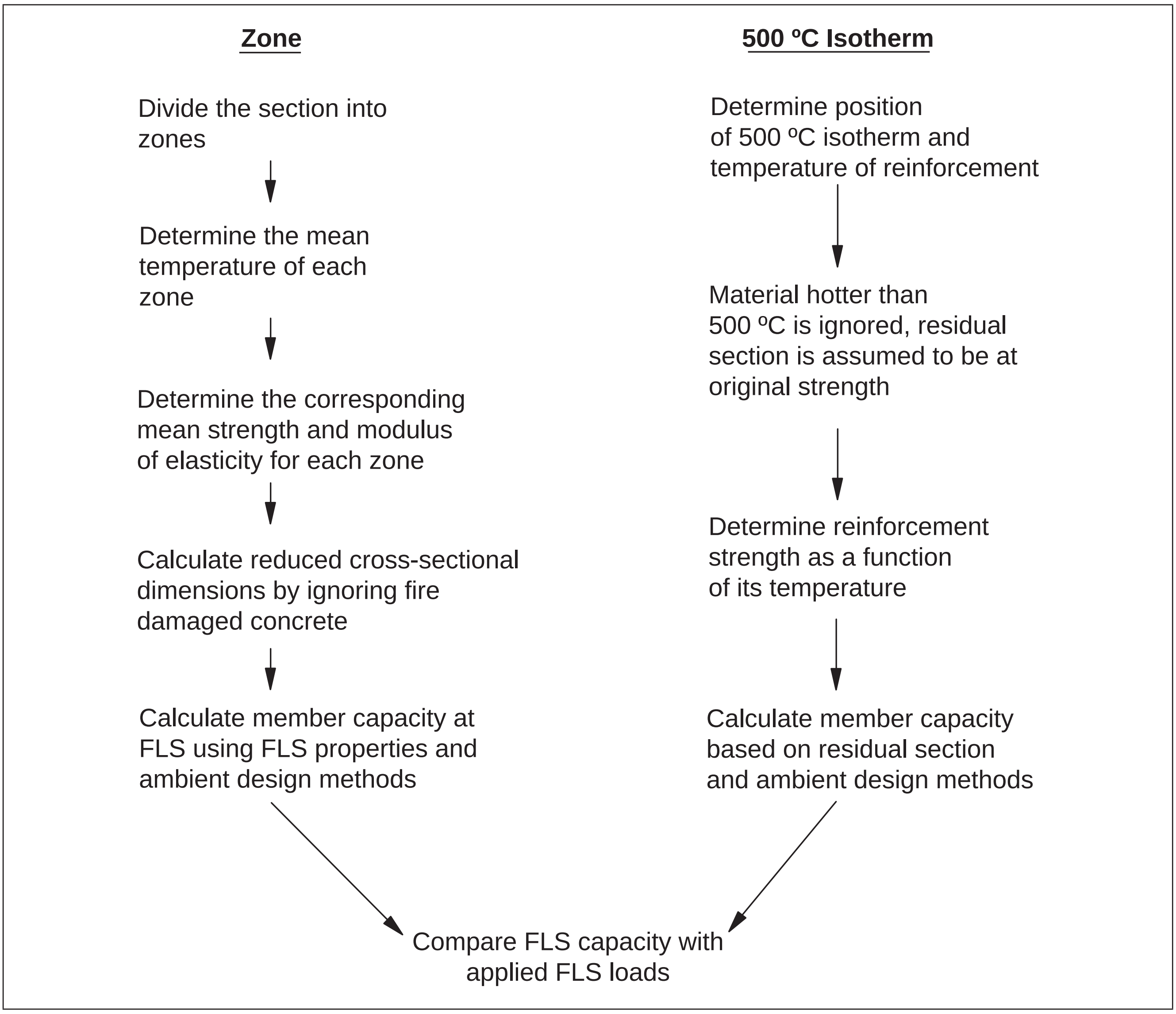
12.3.5.2 Empirical data from testing

The capacity of a concrete member to resist the effects of fire can be governed by the dimensions of the member in question and the depth of cover to its steel reinforcement, as specified in the relevant design codes. As the prescriptive requirements have been derived from testing in accordance with BS 476-20, they are only appropriate for situations where the standard fire is sufficiently onerous or representative of the real fire conditions. However, they are inappropriate for more severe fire scenarios, e.g. hydrocarbon fire exposure in which the propensity to spalling is much greater.

12.3.5.3 Simplistic calculation methods

Simplistic design methods for the fire performance of concrete elements are typically based on ambient design methods. BS EN 1992-1-2 provides two alternative methods for calculating resistance bending moments and axial forces at fire limit state: the isotherm method 500 °C and the zone method. Both methods are based on ambient design methods, but they differ in their assessment of the residual cross-sectional dimensions and material properties at fire limit state, as described in Figure 24.

Figure 24 **Design methods for fire limit state (FLS) design adopted in BS EN 1992-1-2**



NOTE When calculating the capacity at the fire limit state, safety factors can be used.

Second order effects can be included in both models. The two methods are applicable to structures subjected to a standard fire exposure, and the isotherm method can be used for parametric fires. The zone method is recommended for use with small sections and slender columns and it is only valid for standard fire since the calculation of the fire damaged area is based on the assumption of standard fire exposure.

12.3.5.4 Advanced calculation methods

Advanced calculations methods are rarely used for the fire design of concrete structures as:

- the design of concrete structures is not seen to be governed by the fire limit state requirements, so there is no commercial benefit to adopting sophisticated analysis techniques for fire limit state design;
- it is difficult to reliably predict the behaviour of concrete at elevated temperature particularly in relation to spalling, so few

software packages have been adequately validated for use in concrete-framed construction;

- few tests have been conducted to determine the fire performance of concrete-framed structures.

However, providing they have been validated for use in concrete-frame structures, advanced methods and frame analysis methods can be used for concrete structures. Care should be taken to ensure that appropriate sensitivity studies are conducted to mitigate the unreliability of concrete properties at elevated temperature.

12.3.6 Steel and cast and wrought iron

12.3.6.1 General

There are many different types of steel, e.g. carbon steel, stainless steel and fire resistant steel, each with their own thermal properties. Steels generally begin to lose strength at approximately 300 °C. At 800 °C hot finished steels retain approximately 10% of their original strength. Therefore, the ability of a steel loadbearing element to sustain its design load on exposure to fire depends on:

- the temperature developed within the steelwork;
- the reduction in mechanical properties associated with the temperature rise; and
- the capacity of the element to sustain the imposed load given its reduced capacity.

A number of methods have been outlined in **10.3** for determining the temperatures developed in steelwork exposed to fire environments. The consequence of the temperature rise in terms of resistance to fire depends on the temperature differential, with temperature affecting thermal expansion, the stress-strain relationship and ultimate capacity.

The temperature-dependant properties of steels are well known and the performance of steel structures at elevated temperatures can be accurately predicted. Steel has a high thermal conductivity and heats up relatively quickly so the inherent fire resistance of steel is not as high as other forms of construction, such as concrete. Fully exposed structural steel members could require applied fire protection in order to achieve the required fire performance.

12.3.6.2 Empirical data from testing

Since the material properties of structural steelwork are well known at elevated temperatures, the fire performance of structural members can be determined using analytical methods. Therefore, empirical or prescriptive solutions are not necessary. The most obvious exception is for the performance of protected steelwork where fire protection thickness is commonly derived from testing. Manufacturers of fire protection material provide tabulated data prescribing the required fire protection thickness for a steel element as a function of its section factor (defined as the area of the heated surface of the steel divided by the volume of the heated steel). In many cases manufacturers' data are available for different steel limiting temperatures and assist the designer in accounting for the load ratio within the member.

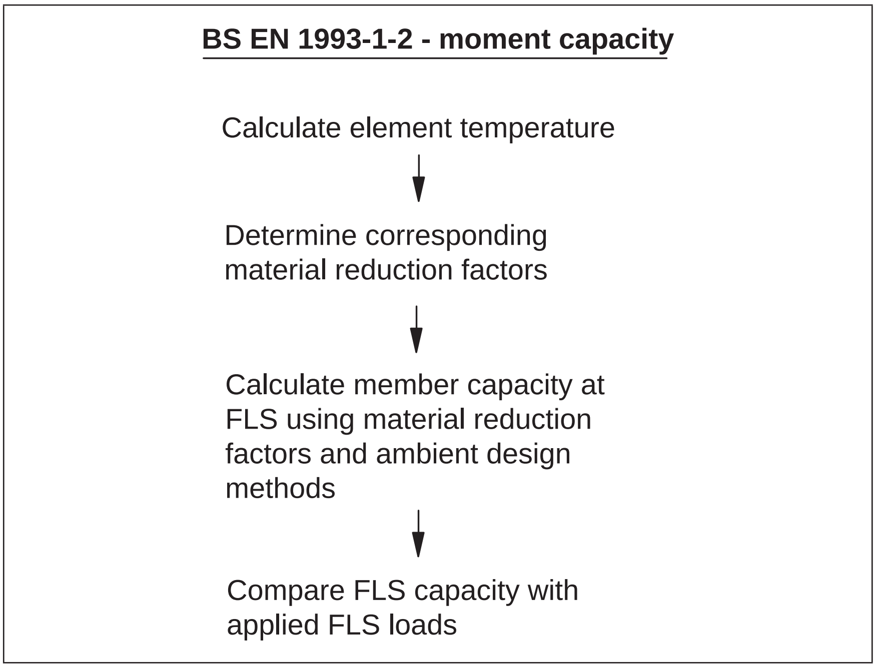
NOTE In most instances, the tabulated data are for standard fire exposure only and the data are product specific.

12.3.6.3 Simplistic calculation methods

Simplistic design methods for isolated members in bending, compression and tension are well established. These methods are based upon ambient engineering analyses, but the member capacity is based upon the elevated temperature properties for strength and stiffness and the applied loads at fire limit state. Some methods assume uniform temperature distributions within the member, but others can account for varying temperature distributions through the member and along its length.

BS EN 1993-1-2 provides simplistic calculation models for members in pure bending, compression and tension, and combined bending and compression. The designer should follow BS EN 1993-1-2:2005, **12.2.1** (see Figure 25).

Figure 25 **Principle design methodologies adopted in BS EN 1993-1-2**



NOTE Within BS EN 1993-1-2 it is possible to consider a uniform temperature across the cross-section or to divide the cross-section into zones and consider temperature distributions.

BS EN 1993-1-2 provides a calculation method for the buckling resistance as a function of fire limit state slenderness.

12.3.6.4 Compression members (BS EN 1993-1-2)

The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a class 1, class 2 or class 3 cross-section and a uniform temperature θ_a should be determined from:

$$N_{b,fi,t,Rd} = \chi_{fi} \, A \, k_{y,\theta} f_y \, / \, \gamma_{M,fi} \tag{69}$$

where:

- χ_{fi} is the reduction factor for flexural buckling in the fire design situation;
- $k_{y,\theta}$ is the reduction factor from BS EN 1993-1-2:2005, section 3 for the yield strength of steel at the steel temperature θ_a reached at time t.

The value of χ_{fi} should be taken as the lesser of the values of $\chi_{\text{y,fi}}$ and $\chi_{\text{z,fi}}$ determined according to:

$$\chi_{\text{fi}} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \bar{\lambda}_\theta^2}} \tag{70}$$

with:

$$\varphi_\theta = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right] \tag{71}$$

and:

$$\alpha = 0.65\sqrt{235/f_y} \tag{72}$$

The non-dimensional slenderness $\bar{\lambda}_\theta$ for the temperature θ_a, is given by:

$$\bar{\lambda}_\theta = \bar{\lambda}\left[k_{y,\theta}/k_{E,\theta}\right]^{0.5} \tag{73}$$

where:

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at the steel temperature θ_a reached at time t;

$k_{E,\theta}$ is the reduction factor for the slope of the linear elastic range at the steel temperature θ_a reached at time t.

12.3.6.5 Compression members

The ability of a steel member to sustain any given load, flexural or axial, is a function of its temperature. The limiting temperature appropriate to any given loading condition should be established and compared with the design temperature. This enables the steel member's capacity to sustain its function to be determined. The designer should note that the limiting temperature typically relates to some key component of the steel member, e.g. lower flange, and is not a mean cross-sectional value.

The limiting temperature is a function of the load ratio, the nature of the applied load (compression, tension or flexure), the temperature gradient and section dimensions.

Limiting temperatures for hot finished structural steel (strength grades 235, 275 and 355) range from 450 °C to 880 °C.

The limiting temperatures for cold formed steel are 50 °C to 100 °C below the equivalent values for hot rolled sections.

End restraint to columns is a beneficial effect in fire, helping to counteract the tendency for local buckling. Load sharing can also occur among members. These effects are taken into account for columns in walls by slightly increasing their compressive strength.

The designer should note that intumescent coating systems can be marketed based on their insulation capability to restrict the temperature rise of columns and floor beams to a maximum of 550 °C and 620 °C respectively. These design temperatures are often accepted through the manufacturer's specification of dry film thicknesses. However, these temperatures might not be always appropriate and should be examined by the designer for the project in hand.

The load ratio compares the load carried at the fire limit state to the load capacity of the section at ambient temperature (20 °C). The load applied during the fire can be significantly lower than the loads ordinarily considered during design and should be allocated partial safety factors.

The load ratio for columns exposed on up to four sides should be determined as follows.

a) For columns in simple construction designed in accordance with BS EN 1993-1-2:

$$R = \frac{F_f}{A_g p_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{p_y Z_y} \tag{74}$$

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 the moment resistance to lateral torsional buckling (Nm);

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

$$R = \frac{F_f}{A_g p_y} + \frac{M_{fx}}{M_{cx}} + \frac{M_{fy}}{M_{cy}} \text{ or} \tag{75}$$

$$R = \frac{F_f}{A_g p_c} + \frac{mM_{fx}}{M_b} + \frac{mM_{fy}}{p_y Z_y} \tag{76}$$

where:

M_{cx} is the moment capacity about the major axis;

M_{cy} is the moment capacity about the minor axis;

m is the equivalent uniform moment factor.

When evaluating members within frameworks with uncertain end conditions, the load ratio can be conservatively assigned a value of 0.67.

BS EN 1993-1-2 gives a method for comparing capacity with applied load for, for example, a typical compression member with differing slenderness.

12.3.6.6 Advanced calculation methods

The effects of thermally-induced strains and stresses due to temperature rise and temperature differentials should be considered.

The model for mechanical response should also take account of:

- the combined effects of mechanical actions, geometrical imperfections and thermal actions;
- the temperature-dependent mechanical properties of the material (see Annex A);

- geometrical non-linear effects; and
- the effects of non-linear material properties, including the unfavourable effects of loading and unloading on the structural stiffness.

Provided that the stress-strain relationships given in Annex A are used, the effects of transient thermal creep need not be given explicit consideration.

The deformations at ultimate limit state implied by the calculation method should be limited to ensure that compatibility is maintained between all parts of the structure.

The design should take into account the ultimate limit state beyond which the calculated deformations of the structure would cause failure due to the loss of adequate support to one of the members.

For the analysis of isolated vertical members, a sinusoidal initial imperfection with a maximum value of $h/1\,000$ at mid-height should be used when not specified by the relevant product standards.

12.3.6.7 Finite element analysis

Finite element analysis is often used for steel structures:

- when it is important to consider local effects;
- to assess the impact of localized heating in frame structures; and
- for determining the impact of restrained thermal expansion.

12.3.7 Cast iron and wrought iron loadbearing elements

Cast iron elements are capable of retaining their design function at elevated temperatures under favourable conditions. However, the brittle nature of cast iron and its variable quality results in a reduced capacity to accommodate deflection. Cast iron has good compressive strength but low tensile and flexural strength. Where cast iron is used in bending, design loads are extremely low and restraint on thermal expansion can exaggerate bending during a fire in both beams and columns.

Cast iron members are particularly susceptible to damage from the less quantifiable effects of a fire. For example, local heating of parts of an element or rapid cooling from a hose stream can lead to sudden failure.

In contrast, wrought iron, although a variable material in terms of its quality, is considerably more ductile and better suited to bending. It does not react in the same way as cast iron to situations involving thermal shock.

Subject to the adoption of sufficiently conservative design assumptions, both cast iron and wrought iron structures can be evaluated in a manner similar to that used on mild steels in the context of determining performance under conditions of exposure to fire.

NOTE Additional information on iron and steel structures can be found in [47].

12.3.8 Aluminium alloy loadbearing elements

Guidance on the design of loadbearing elements is given in BS EN 1999-1-2.

Whilst the methods adopted in determining the mechanical response of aluminium structures to fire is similar to those of steel structures, the key difference is that the rate of loss in strength with increasing temperature for aluminium is significantly greater than that for steelwork. The loss in strength is also particularly sensitive to the exact composition and heat treatment condition.

In the range of temperatures encountered in building fires, aluminium alloys can be considered to be non-combustible. The variation in 0.2% proof stress with temperature is alloy specific but is particularly rapid between 150 °C and 350 °C, during which up to 80% of its strength is lost. At 550 °C, the strength of aluminium alloys is virtually zero.

Any material properties used in the analysis of the thermal or mechanical response of aluminium elements should be appropriate. If protective coatings are employed, care should be taken to ensure that the dry film thicknesses recommended by manufacturers are valid for aluminium and its critical temperature. More often, recommended dry film thicknesses are established in fire tests as suitable for maintaining steelwork temperatures in the range of 550 °C to 620 °C and, as such, are unsuited to aluminium alloys.

When considering aluminium, the validity of the fire conditions predicted within the enclosure should be confirmed. Aluminium is widely used in applications where hydrocarbon fire conditions should be assumed, e.g. off-shore drilling platforms.

12.3.9 Timber

12.3.9.1 General

Guidance on the design of loadbearing timber elements to resist fire is given in BS EN 1995-1-2.

As a combustible material, the surface of timber shrinks and burns to form a post carbonization char when exposed to fire. The charred material has little residual strength but it insulates unexposed areas of timber which are unaffected by the fire. The depth of char is predictable. Therefore, design methods are based around determining the depth of char, ignoring the charred material, and determining whether the residual section has sufficient capacity to support the applied loads at fire limit state. Whilst methods for predicting the depths and rates of charring are discussed in Annex A, attention is drawn to the potential local increase in charring resulting from:

- metal fasteners, e.g. nails, screws;
- metal plate connectors;
- increased heating at corners (arises); and
- joints in glue laminated members.

The coating of timber elements with intumescent paints or varnish is not effective in the same way as it is with protecting metals. Most coatings only provide a delay to ignition after which normal charring rates apply. Ad hoc testing can be used to establish the ignition delay.

12.3.9.2 Empirical data from testing

An empirical method is available for quick evaluation of the performance of timber beams exposed to the BS 476-20 fire resistance test. Under standard exposure heating conditions, the fire resistance of a beam subjected to three sided fire attack is given by:

$$t_{fr} = 0.1 f b_t \left(4 - \frac{b_t}{d_t} \right) \tag{77}$$

where:

- t_{fr} is the fire resistance time (min);
- f is the empirical factor (min/mm);
- b_t is the breadth of beam (mm);
- d_t is the depth of beam (mm).

Equation 77 assumes a charring rate of 0.6 mm per minute. The empirical factor, *f*, allows for over-design and is a function of the load ratio, i.e. the ratio between the imposed load and the load capacity of the beam. Conservatively, *f* can be assigned a value of unity.

Alternatively, for 30 and 60 minutes, fire resistance notional charring rates for calculating the residual section for loadbearing calculations are given in Table 15.

Table 15 **Notional char depths for various species after 30 min and 60 min in the standard furnace test (BS 476-20)**

Species	**Depth of char in 30 min**	**Depth of char in 60 min**
	mm	mm
(a) All structural species with a density greater than 420 kg/m^3 and not included in (b) and (c)	20	40
(b) Western red cedar	25	50
(c) Hardwoods having a nominal density not less than 650 kg/m^3 at 18% moisture content	15	30

12.3.9.3 Simplistic calculation methods

In BS EN 1995-1-2 the design mechanical resistance of timber for strength and stiffness is determined from the following:

$$f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma_{M,fi}} \tag{78}$$

$$S_{d,fi} = k_{mod,fi} \frac{S_{20}}{\gamma_{M,fi}} \tag{79}$$

where:

- $f_{d,fi}$ is the design strength in fire;
- $S_{d,fi}$ is the stiffness property (modulus of elasticity or shear modulus);
- f_{20} is the 20% fractile of a strength property at ambient temperature;
- S_{20} is the 20% fractile of a stiffness property (modulus of elasticity or shear modulus) at ambient temperature;
- $k_{mod,fi}$ is the modification factor in fire;
- $\gamma_{M,fi}$ is the partial safety factor in fire.

The modification factor $k_{mod,fi}$ is applied to the relevant component/system.

The design value $R_{d,fi}$ of a mechanical resistance (loadbearing capacity) is calculated as:

$$R_{d,fi} = \eta \frac{R_{20}}{\gamma_{M,fi}} \quad (80)$$

where:

- $R_{d,fi}$ is the design value of a mechanical resistance in the fire situation at time t;
- R_{20} is the 20% fractile value of a mechanical resistance at normal temperature without the effect of load duration and moisture;
- η is a conversion factor;
- $\gamma_{M,fi}$ is the partial safety factor for timber in fire.

The 20% fractile of a strength or stiffness property is calculated as:

$$f_{20} = k_{fi} f_k \quad (81)$$

$$S_{20} = k_{fi} S_{05} \quad (82)$$

where:

- f_{20} is the 20% fractile of a strength property at ambient temperature;
- f_k is characteristic strength;
- S_{20} is the 20% fractile of a stiffness property (modulus of elasticity or shear modulus) at ambient temperature;
- S_{05} is the 5% fractile of a stiffness property (modulus of elasticity or shear modulus) at ambient temperature;
- k_{fi} is given in Table 16 for different components/elements.

Table 16 **Values of k_{fi} for different components/elements**

Component/element	**k_{fi}**
Solid timber	1.25
Glued laminated timber	1.15
Wood based panels	1.15
LVL	1.1
Connections with fasteners in shear with side members of wood and wood based panels	1.15
Connection with fasteners in shear with side members in steel	1.05
Connections with axially loaded fasteners	1.05

The 20% fractile of a mechanical resistance R_{20} of a connection is calculated as:

$$R_{20} = k_{fi} R_k \quad (83)$$

where:

- k_{fi} is taken from Table 16;
- R_k is the characteristic mechanical resistance of a connection at ambient temperature without the effect of load duration and moisture (k_{mod} = 1).

In BS EN 1995-1-2, two methods are presented to take account of the cross-section properties at the fire limit state.

a) Reduced cross-section method

An effective cross-section is calculated by reducing the initial cross-section by the effective charring depth d_{ef} as given by the following equation (see Figure 26A):

$$d_{ef} = d_{char,n} + k_0 d_0 \tag{84}$$

where:

d_0 = 7 mm;

$d_{char,n}$ is the depth of char and is determined according to equation (A88);

k_0 is as follows:

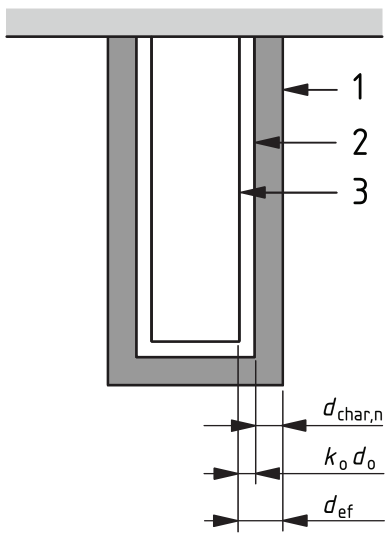
- for unprotected surfaces, k_0 is as given in Table 16A (see Figure 26B);
- for protected surfaces where time to start of charring (t_{ch}) >20 min, k_0 varies linearly from 0 to 1 during the time interval t = 0 to $t = t_{ch}$ (see Figure 26C);
- for protected surfaces where $t_{ch} \leq 20$ min, k_0 is as given in Table 16A (see Figure 26B):

where

t is time of fire exposure in minutes.

NOTE It is assumed that the material close to the char line in the layer of thickness $k_0 d_0$ has zero strength and stiffness, while the strength and stiffness properties of the remaining cross-section are assumed to be unchanged.

Figure 26A **Definition of residual cross-section and effective cross-section**



Key

1 Initial surface of member

2 Border of residual cross-section

3 Border of effective cross-section

Figure 26B **Relationship between k_0 and time of fire exposure for unprotected surfaces, and for protected surfaces where t_{ch} ≤20 min**

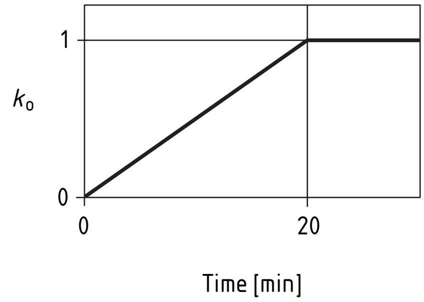


Figure 26C **Relationship between k_0 and time of fire exposure for protected surfaces where t_{ch} >20 min**

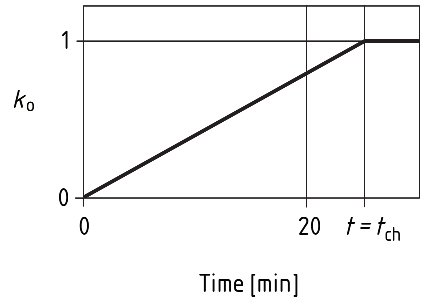


Table 16A **Determination of k_0**

t min	k_0
<20	t/20
≥20	1.0

The design strength and stiffness properties of the effective cross-section should be calculated with $k_{mod,fi}$ = 1.0.

b) Reduced properties method

For rectangular and round cross-sections exposed on three or four sides and fire resistance periods greater than 20 minutes, the factor $k_{mod,fi}$ is modified for the following parameters:

For strength in bending:

$$k_{mod,fi} = 1.0 - \frac{1}{200}\frac{p}{A_r} \quad (85)$$

For strength in compression:

$$k_{mod,fi} = 1.0 - \frac{1}{125}\frac{p}{A_r} \quad (86)$$

For strength in tension and elastic modulus:

$$k_{\mathrm{mod,fi}} = 1.0 - \frac{1}{330}\frac{p}{A_{\mathrm{r}}} \tag{87}$$

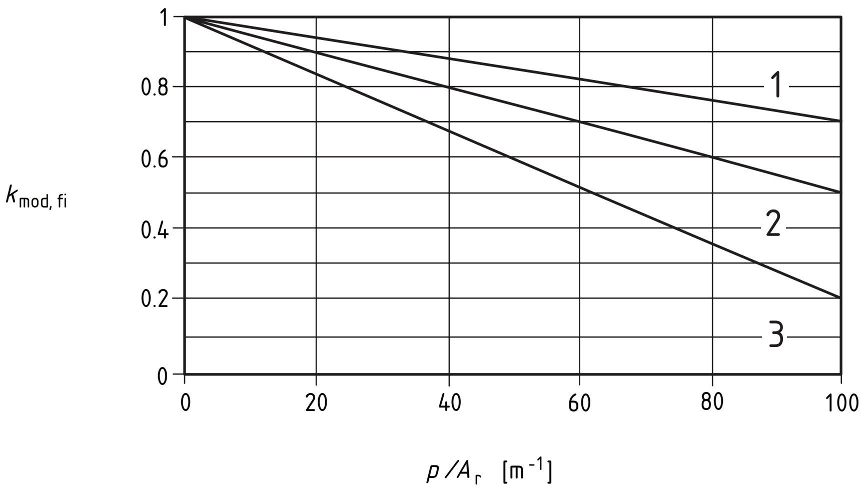
where:

p is the perimeter of the fire exposed residual cross section, m;

A_{r} is the area of the residual cross section, m^2.

Equations 85 to 87 are illustrated in Figure 27.

Figure 27 **Equations 85 to 87 illustrated**



Key

1 Tensile strength, elastic modulus

2 Bending strength

3 Compressive strength

12.3.9.4 Advanced calculations for thermal response

The deformation at ultimate limit state, as implied by the calculation methods, should be limited as necessary to ensure that compatibility is maintained between all parts of the structure. Where relevant, the mechanical response of the model should also take account of geometrical non-linear effects. In the analysis of individual members or sub-assemblies the boundary conditions should be checked and detailed in order to avoid failure due to the loss of adequate support for the members.

It should be verified that:

$$E_{\mathrm{fi,d}}(t) \le R_{\mathrm{fi,t,d}} \tag{88}$$

where:

$E_{\mathrm{fi,d}}$ is the design effect of actions for the fire situation, determined in accordance with BS EN 1991-1-2, including effects of thermal expansions and deformations;

$R_{\mathrm{fi,t,d}}$ is the corresponding design resistance in the fire situation;

t is the designed duration of fire impact.

In the calculation of loadbearing structures, the way in which the structure collapses in a fire, the temperature-dependant material properties, e.g. stiffness and the effect of thermal strain and deformation should be assessed.

12.3.10 Masonry

12.3.10.1 General

Like concrete structures, masonry walls have good inherent fire resistance and perform well under fire exposure. When structural failure does occur, it can generally be attributed to eccentric loading on the top of the wall, thermal bowing or imposed loads from other deflecting or collapsing structures.

The materials used in masonry walls tend to have a low thermal conductivity. In addition, masonry walls are often subjected to single-side exposures. Therefore, such walls are likely to be subjected to a high temperature gradient across the section and the corresponding thermal expansion can lead to thermal bowing towards the fire. Thermal bowing can be detrimental to the fire performance of the wall.

Guidance on determining the ability of loadbearing masonry to resist fire is given in BS EN 1996-1-2. The content of BS EN 1996-1-2 is largely descriptive, i.e. specifying minimum wall thicknesses for achieving designated fire resistance ratings when exposed to standard fire test conditions.

12.3.10.2 Empirical data from testing

Careful consideration should be given to the extrapolation of prescriptive design rules derived from tests (typically at a maximum size of 3 m × 3 m) to applications in buildings. Full-scale effects such as thermal bowing can lead to behaviour not exhibited in fire tests, e.g. the collapse and/or the imposition of lateral loads on buildings. It is recommended that scaling effects be fully investigated if the ratio between the wall's height and thickness exceeds 25.

a) Thermal bowing:

A rule of thumb is that if the wall deflection is less the wall thickness, the resulting eccentricity is unlikely to promote failure.

b) Fire resistance:

Prescriptive guidance provides tabulated data for the fire resistance of masonry wall construction. These data are based on test results from standard fire resistance tests extending over 50 years.

In general, the fire resistance of masonry walls is defined by a minimum wall thickness for a specified period of fire resistance, ranging from 30 minutes up to 6 hours. Table 17 shows an example of the tabulated data within BS EN 1996-1-2.

Table 17 **Minimum thickness requirements for dense and lightweight aggregate masonry, single-leaf, loadbearing walls (extracted from NA to BS EN 1996-1-2:2005, Table NA.3.2)**

Row number	Material properties: Gross density ρ (kg/m^3)	Minimum wall thickness (mm) for fire resistance classification REI for time (min) $t_{fi,d}$					
		30	**60**	**90**	**120**	**180**	**240**
1	**Group 1 units** Mortar: general purpose, thin layer, lightweight						
1.1	lightweight aggregate $400 \leq \rho \leq 1700$						
1.1.1 1.1.2	$\alpha \leq 1.0$	90 (90)	90 (90)	100 (90)	100 (90)	140 (100)	150 (100)
1.1.3 1.1.4	$\alpha \leq 0.6$	70 (60)	75 (60)	90 (75)	90 (75)	100 (90)	100 (90)
1.2	dense aggregate $1200 \leq \rho \leq 2400$						
1.2.1 1.2.2	$\alpha \leq 1.0$	90 (90)	90 (90)	90 (90)	100 (90)	140 (100)	150 (100)
1.2.3 1.2.4	$\alpha \leq 0.6$	75 (60)	75 (75)	90 (75)	90 (75)	100 (90)	140 (100)
2	**Group 2 units** Mortar: general purpose, thin layer, lightweight						
2.1	lightweight aggregate $240 \leq \rho \leq 1300$						
2.1.1 2.1.2	$\alpha \leq 1.0$	90 (90)	100 (90)	100 (90)	100 (100)	140 (140)	150 (140)
2.1.3 2.1.4	$\alpha \leq 0.6$	75 (75)	90 (75)	90 (75)	100 (90)	125 (100)	140 (125)
2.2	dense aggregate $720 \leq \rho \leq 1800$						
2.2.1 2.2.2	$\alpha \leq 1.0$	100 (90)	100 (100)	140 (100)	140 (140)	140 (140)	190 (150)
2.2.3 2.2.4	$\alpha \leq 0.6$	90 (75)	100 (90)	100 (90)	140 (125)	140 (125)	150 (140)

12.3.10.3 Simplistic calculation methods

12.3.10.3.1 Thermal bowing

A linear temperature gradient across a wall causes expansion of the face exposed to heat relative to the unexposed face. The differential expansions cause the wall to bow and the extent of the bowing depends on the fixing conditions for the wall. Where a wall is unrestrained it can be considered as a one dimensional cantilever. Movement at the head of the wall can be calculated by [48]:

$$\Delta_{head} = \frac{\alpha h^2 (T_{exp} - T_{unexp})}{2 d_w} \quad (89)$$

where:

α is 6 × 10^{-6}/K for masonry.

The deflection at the head of the wall can be significant. It can open gaps and permit fire spread, cause the wall to bear and transmit load to unsuitable paths or, ultimately, cause the wall to collapse. Consideration of the effects of thermal bowing should be project-specific. However, it is reasonable to assume, in the absence of detailed analysis, that once the bowing at the head of the wall moves the heated face outside the original plane of the unexposed face, collapse can occur.

Fixing the head of the wall reduces the extent of bowing. In such cases, maximum thermal bowing (lateral deflection) occurs at mid-height towards the fire and can be assumed to have a value of bow (mm) given by:

$$\Delta_{bow} = \frac{\alpha h^2 (T_{exp} - T_{unexp})}{8 d_w} \quad (90)$$

Equations 89 and 90 assume elastic behaviour and linear temperature gradients. The equations can also be applied to predict thermal bowing of steel columns built into walls, using appropriate temperature values and a coefficient of expansion of 1.4 × 10^{-5}. The fixity between the walls and the steel columns also influences their relative movement. Given the potential conflicting requirements to maintain integrity and stability, the design of fixings is important.

The fixing of walls into columns can also provide a means of reducing thermal bowing in the horizontal plane between columns. Equation 90 can be used to predict horizontal bowing with height, *h* being redefined as the column spacing. Load applied to the wall also reduces its propensity to bowing.

12.3.10.3.2 Fire resistance

To determine the fire resistance, the temperature profile of the cross-section is established to identify the structurally ineffective section and the residual cross-section. The loadbearing capacity at the ultimate limit state of the residual cross-section is calculated, and this is checked to ensure it is greater than that required with the relevant load combination of actions.

At the limit state for the fire situation, the design value of vertical load applied to a wall or column should be less than or equal to the design value of the vertical resistance of the wall or column such that:

$$N_{Ed} \leq N_{Rd,fi\theta 2} \quad (91)$$

The design value of the vertical resistance of the wall or column is given by:

$$N_{\mathrm{Rd,fi}\theta 2} = \Phi\left(f_{\mathrm{d}\theta 1}A_{\theta 1} + f_{\mathrm{d}\theta 2}A_{\theta 2}\right) \quad (92)$$

where:

- A is the total area of masonry;
- $A_{\theta 1}$ is the area of masonry up to θ_1;
- $A_{\theta 2}$ is the area of masonry between θ_1 and θ_2;
- θ_1 is the temperature up to which the cold strength of masonry can be used;
- θ_2 is the temperature above which the material has no residual strength;
- N_{Ed} is the design value of the vertical load;
- $N_{\mathrm{Rd,fi}\theta 2}$ is the design value of the resistance in fire;
- $f_{\mathrm{d}\theta 1}$ is the design compressive strength of masonry up to θ_1;
- $f_{\mathrm{d}\theta 2}$ is the design strength of masonry in compression between θ_1 and θ_2°C, taken as $cf_{\mathrm{d}\theta 1;}$
- Φ is the capacity reduction factor in the middle of the wall obtained from BS EN 1996-1-1:2005, **6.1.2.2** taking into account additionally the eccentricity $e_{\Delta\theta}$.

The temperature distribution across a masonry section, and the temperature at which the masonry becomes ineffective as a function of the time of fire exposure, should be obtained from the results of tests or from a database of test results. In the absence of test results or a database, BS EN 1996-1-2:2005, and Figure 28 and Table 18 of this Published Document, can be used. For autoclaved aerated concrete masonry, reference can be made to BS EN 12602.

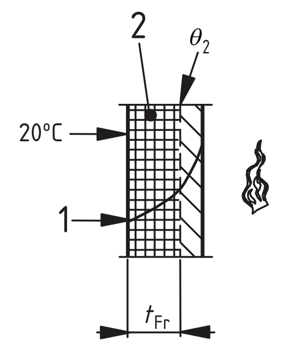
The eccentricity, $e_{\Delta\theta}$, due to the fire load, for use in this simplified calculation method can be obtained from test results or from equation 93, (see also BS EN 1996-1-2:2005, Figure 29).

$$e_{\Delta\theta} = \frac{1}{8}h_{\mathrm{ef}}^2\frac{\alpha_{\mathrm{t}}\left(\theta_2 - 20\right)}{t_{\mathrm{Fr}}} \leq h_{\mathrm{ef}}/20 \quad (93)$$

where:

- $e_{\Delta\theta}$ is the eccentricity due to variation of temperature across masonry;
- h_{ef} is the effective height of the wall;
- α_{t} is the coefficient of thermal expansion of masonry according to BS EN 1996-1-1:2005, **3.7.4**;
- 20 °C is the temperature assumed on the cold side;
- t_{Fr} is the thickness of the cross-section whose temperature does not exceed θ_2.

Figure 28 **Vertical section on masonry (adapted from BS EN 1996-1-2:2005, Figure C.2)**



Key

1 Temperature distribution from BS EN 1996-1-2:2005, Figure C.3a) to Figure C.3g).

2 Residual area of the cross-section with strength ($A_{\theta 1} + A_{\theta 2}$).

Table 18 **Values of constant, *c*, and temperatures θ_1 and θ_2 by masonry material – (extracted from BS EN 1996-1-2:2005, Figure C.2)**

Masonry units and mortar (surface unprotected)	Values of constant c	Temperature °C	
		θ_2	θ_1
Clay units with general purpose mortar	c_{cl}	600	100
Calcium silicate units with thin layer mortar	c_{cs}	500	100
Lightweight aggregate units (pumice) with general purpose mortar	c_{la}	400	100
Dense aggregate units with general purpose mortar	c_{da}	500	100
Autoclavated aerated units with thin layer mortar	c_{aac}	700	200

NOTE Values of c *are nationally determined parameters.*

12.3.10.4 Advanced calculation method

NOTE For autoclaved aerated concrete masonry, reference can be made to BS EN 12602. For other materials reference can be made to other authoritative publications.

The deformation at ultimate limit state implied by the calculation methods should be limited as necessary to ensure that compatibility is maintained between all parts of the structure. Where relevant, the mechanical response of the model should also take account of geometrical non-linear effects.

In the analysis of individual members or sub-assemblies, the boundary conditions should be checked and detailed in order to avoid failure due to the loss of adequate support for the members.

It should be verified that:

$$E_{\mathrm{fi,d}}(t) \le R_{\mathrm{fi,t,d}} \tag{94}$$

where:

$E_{\mathrm{fi,d}}$ is the design effect of actions for the fire situation, determined in accordance with BS EN 1991-1-2, including effects of thermal expansions and deformations;

$R_{\mathrm{fi,t,d}}$ is the corresponding design resistance in the fire situation;

t is the designed duration of fire impact.

In the calculation of loadbearing structures, the way in which the structure collapses under fire impact, temperature-dependant material properties including stiffness as well as the effect of thermal strain and deformation (indirect fire impact) should be assessed.

12.3.11 Glass

12.3.11.1 General

Glass has not traditionally been used as a loadbearing element, except in the external façade and roofs where it is expected to resist imposed wind loads. These applications rarely require fire resistance, and where they do, it is common to use glasses that can satisfy the fire resistance test and use limit state design principles by assuming that the maximum wind load and the fire do not coincide. Glass is, however, increasingly used for the provision of loadbearing horizontal floors within buildings. The loadbearing elements of these floors generally consist of:

- thick monolithic slabs of normal soda/lime composition glass; or
- laminated sheets of toughened glass bonded together by means of polyvinyl butyl or cold poured resin interlayers.

In the case of float glass there is no public domain information that identifies the reduction in strength of such glasses with temperature. Due to the thickness of the glass (usually 20 mm or greater) there is a certain thermal inertia to overcome and so the mean temperature of the glass rises relatively slowly, assuming that critical differential surface temperatures do not cause the glass to fracture earlier. Whilst the critical temperature differential values that cause fracture are known for float glass products of up to 10 mm thickness, there is no public domain information on the performance of thicker glass. Laminated float glasses are often glued together using either PVB interlayers or cold poured resins and it is the behaviour of these adhesives that dictates the hot strength of the glass. The PVB interlayers soften and start to boil at temperatures little over 100 °C and this process loses bond strength and actively causes glass layer separation. Cold poured resins do not soften like PVB, but they do char and produce smoke, both of which lead to failure.

As a consequence, it is customary for the loadbearing layers to be protected from fire by incorporating layers of insulating glass into a fixed ceiling mounted below the loadbearing layer, with an air gap between the protecting glass and the loadbearing glass. There are a variety of proprietary glasses that use intumescent materials or resin gel technology to create opaque insulating layers when hot, enabling the translucent loadbearing membrane above to remain cooler.

12.3.11.2 Empirical data from testing

At the time of writing, there are no empirical data for the fire performance of structural glass in the public domain.

12.3.11.3 Simplistic calculation method

At the time of writing, there are no known simplistic calculation methods for the fire performance of structural glass in the public domain.

12.3.11.4 Advanced calculation methods

The deformation at ultimate limit state implied by the calculation methods should be limited as necessary to ensure that compatibility is maintained between all parts of the structure.

Where relevant, the mechanical response of the model should also take account of geometrical non-linear effects. In the analysis of individual members or sub-assemblies, the boundary conditions should be checked and detailed in order to avoid failure due to the loss of adequate support for the members.

It should be verified that:

$$E_{\mathrm{fi,d}}(t) \leq R_{\mathrm{fi,t,d}} \tag{95}$$

where:

- $E_{\mathrm{fi,d}}$ is the design effect of actions for the fire situation, determined in accordance with BS EN 1991-1-2, including effects of thermal expansions and deformations;
- $R_{\mathrm{fi,t,d}}$ is the corresponding design resistance in the fire situation;
- t is the designed duration of fire impact.

In the calculation of loadbearing structures, the way in which the structure collapses under fire impact, the temperature-dependant material properties including stiffness and the effect of thermal strain and deformation (indirect fire impact) should be assessed.

12.3.12 Plastics

12.3.12.1 General

The elevated temperature response of composite plastics materials depends upon the behaviour of the fibre and matrix. The loss in strength of either one of these limits their elevated temperature loadbearing capacity. The matrix has a number of functions and, apart from providing protection (physical and environmental), its primary role is to provide shear, transverse tensile and compression properties to the composite, as well as transferring load between the fibres.

The fibres provide the strength and stiffness to the composite and this can be varied by the type of reinforcement-mat, e.g. strand, woven, as well as the volume fraction. A composite with fibres all aligned in one direction (uni-directional) is strong in that direction but weak in the transverse direction. Directionality is therefore an important consideration in the role of the composite.

The resins of composites are usually made from thermosetting materials and the process of manufacture helps to improve their properties at elevated temperatures. In fire, the chemical reaction in the matrix causes the polymer structure to break down through degradation in mechanical properties. For polyester resins, depending upon the particular formulation, softening occurs between 55 °C and 150 °C. The heat distortion temperature (temperature of deflection under load) provides a measure of the softening temperature and, at temperatures of 130 °C, apart from bisphenol polyesters, the strength of the matrix reduces by more than 50%. The phenolic resins can survive to higher temperatures in which the heat distortion temperature can be as high as 250 °C.

Resins, particularly epoxies, are often able to form a stable char in a similar manner to timber, and the char then provides some insulation to the remainder of the section with little loss in strength of the uncharred material. However, even this is limited and unprotected composites are unlikely to exceed 30 minutes in the fire resistance test without some form of additional passive protection.

12.3.12.2 Empirical data from testing

At the time of writing, there are no known empirical data for the fire performance of structural plastics in the public domain.

12.3.12.3 Simplistic calculation methods

The design of composites follow classic theory on laminate design and this is followed through at elevated temperatures.

The stiffness of laminates should be determined by experimental testing or obtained from manufacturer's data. However, in the absence of data, the stiffness properties can be calculated to determine the effective stiffness of the composite from the properties of the fibres and the matrix using the Halpin-Tsai relationship:

$$P = \frac{P_m\left[P_f + \zeta P_m + \zeta v_f\left(P_f - P_m\right)\right]}{\left[P_f + \zeta P_m - v_f\left(P_f - P_m\right)\right]} \tag{96}$$

where:

- P is the effective property of the composite (elastic and shear modulii);
- P_m is the effective property of the matrix (elastic and shear modulii);
- P_f is the effective property of the fibres (elastic and shear modulii);
- v_f is the volume fraction of fibres;
- ζ is the reinforcing efficiency parameter of the composite material indicating the extent to which the applied force is transmitted to the reinforcing phase.

Design charts derived from the Halpin-Tsai equations are used to derive the effective properties using material property data for the matrix and fibres.

12.3.12.4 Advanced calculation method

The deformation at ultimate limit state implied by the calculation methods should be limited as necessary to ensure compatibility is maintained between all parts of the structure. Where relevant, the mechanical response of the model should also take account of geometrical non-linear effects.

In the analysis of individual members or sub-assemblies, the boundary conditions should be checked and detailed to avoid failure due to the loss of adequate support for the members.

It should be verified that:

$$E_{fi,d}(t) \leq R_{fi,t,d} \qquad (97)$$

where:

$E_{fi,d}$ is the design effect of actions for the fire situation, determined in accordance with BS EN 1991-1-2, including effects of thermal expansions and deformations;

$R_{fi,t,d}$ is the corresponding design resistance in the fire situation;

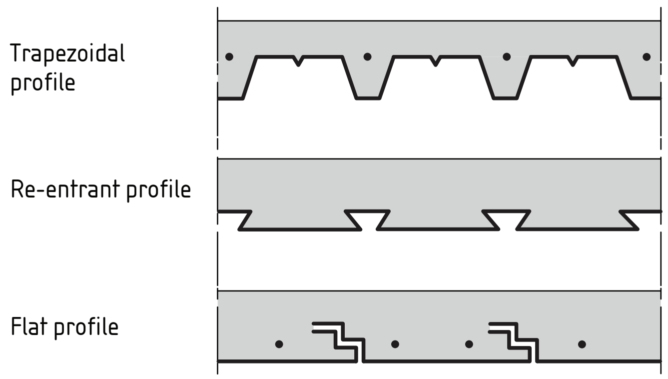
t is the designed duration of fire impact.

12.3.13 Concrete and steel composite floors, beams and columns

12.3.13.1 General

For structural design, the term composite means any structural element comprising two or more materials that have been connected mechanically such that they behave as a single, composite component. The methods described in **12.3.13.2**, **12.3.13.3** and **12.3.13.4** assume that the composite section comprises steel and concrete, but could be suitable to other materials (see Figure 29, Figure 30 and Figure 31).

Figure 29 **Typical examples of concrete floor slabs with profiled steel sheets with or without reinforcing bars (BS EN 1994-1-2)**



In all cases the sections should be designed to act compositely and, where necessary, the mechanical connection between the different materials should be maintained throughout the fire.

This section describes the performance of the composite section, but, for additional guidance, reference should be made to the material-specific sections of PD 7974-3.

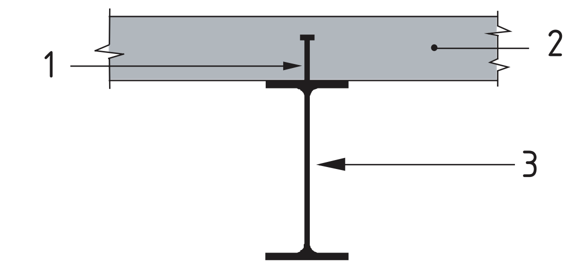
12.3.13.2 Empirical data from testing

As an initial approach, the design of composite members can be based on empirical data from testing. As with the design of concrete sections, the fire resistance of an element is governed by the dimensions of the member and the depth of cover to its steel reinforcement or the steel section. Several standards detail the necessary dimensions and cover depths for composite sections to achieve various fire resistance ratings.

As the prescriptive requirements are derived from testing in accordance with BS 476-20, they only apply to situations where the standard fire is sufficiently onerous or sufficiently representative of the real fire conditions. However, they do not apply to more severe fire scenarios, e.g. hydrocarbon fire exposure in which the propensity to spalling is much greater.

BS EN 1994-1-2 provides tabulated data for minimum depth of cover and minimum cross-sectional dimensions for many different composite sections. Sample tables for composite beams are provided in BS EN 1994-1-2:2005, Table 32 and Table 33 and reference should be made to BS EN 1994-1-2 when using these tables.

Figure 30 **Examples of composite floor beams (BS EN 1994-1-2)**



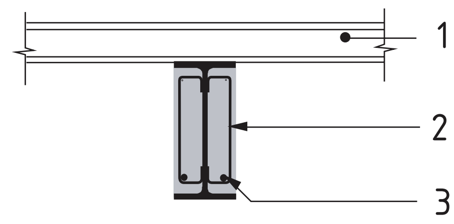
a) Composite beam comprising steel beam with no concrete encasement

Key

1 Shear connectors

2 Flat concrete slab or composite slab with profiled steel sheeting

3 Profiles with or without protection



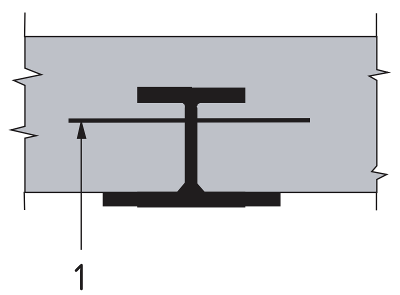
b) Steel beam with partial concrete encasement

Key

1 Optional

2 Stirrups welded to web of profile

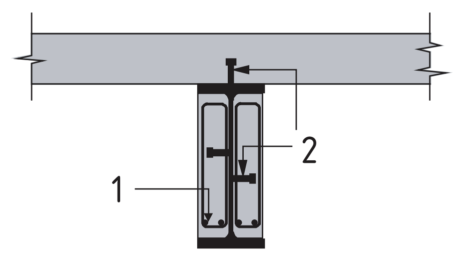
3 Reinforcing bar



c) Steel beam partially encased in slab

Key

1 Reinforcing bar



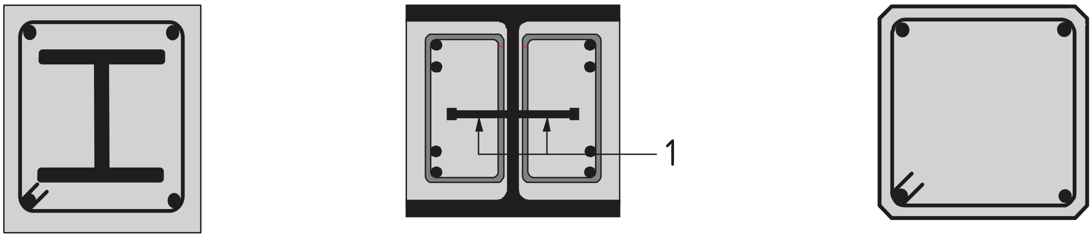
d) Composite beam comprising steel beam with partial concrete encasement

Key

1 Reinforcing bar

2 Shear connectors

Figure 31 **Examples of composite columns (BS EN 1994-1-2)**



a) Concrete encased profiles

b) Partially encased profiles

c) Concrete filled profiles

Key

1 Shear connectors welded to web of profile

12.3.13.3 Simplistic calculation methods

BS EN 1994-1-2 provides design methods for composite floors, beams and columns. The methods are either limiting temperature methods (where the steel section is assumed to have a uniform temperature) or capacity-based methods.

The capacity-based methods require the designer to:

- calculate the temperature of the various components of the cross-section;
- determine the resulting material strengths;
- calculate the capacity of the section based on the above strengths using ambient calculation methods;

  NOTE It is important ensure that any assumed longitudinal shear can be maintained at fire limit state.
- compare the member capacity with the applied load at fire limit state.

The constituent materials of composite sections have different rates of thermal expansion which are not considered within simplistic methods. Therefore, it should be ensured that either:

- the difference in rates of thermal expansion is negligible;
- different rates of thermal expansion do not have an adverse affect on the performance of the composite section; or
- the different rates of thermal expansion are accounted for within the calculation method (this might require the use of advanced calculation methods).

12.3.13.4 Advanced calculation methods

The mechanical response model should take account of:

- the combined effects of mechanical actions, geometrical imperfections and thermal actions;
- the temperature dependent mechanical properties of the materials;
- geometrical non-linear effects;

- the effects of non-linear material properties, including the effects of unloading on the structural stiffness; and
- the effects of thermally induced strains and stresses, due to temperature rise and temperature differentials.

Provided that the stress-strain relationships given in Annex A are used, the effect of high temperature creep need not be explicitly considered.

The deformations at ultimate limit state given by the calculation model should be limited as necessary to ensure that compatibility is maintained between all parts of the structure.

12.3.14 Composite floors systems – sub-frames

12.3.14.1 General

This subclause relates to composite steel-framed buildings consisting of steel beams acting in combination with a composite concrete and metal deck floor system.

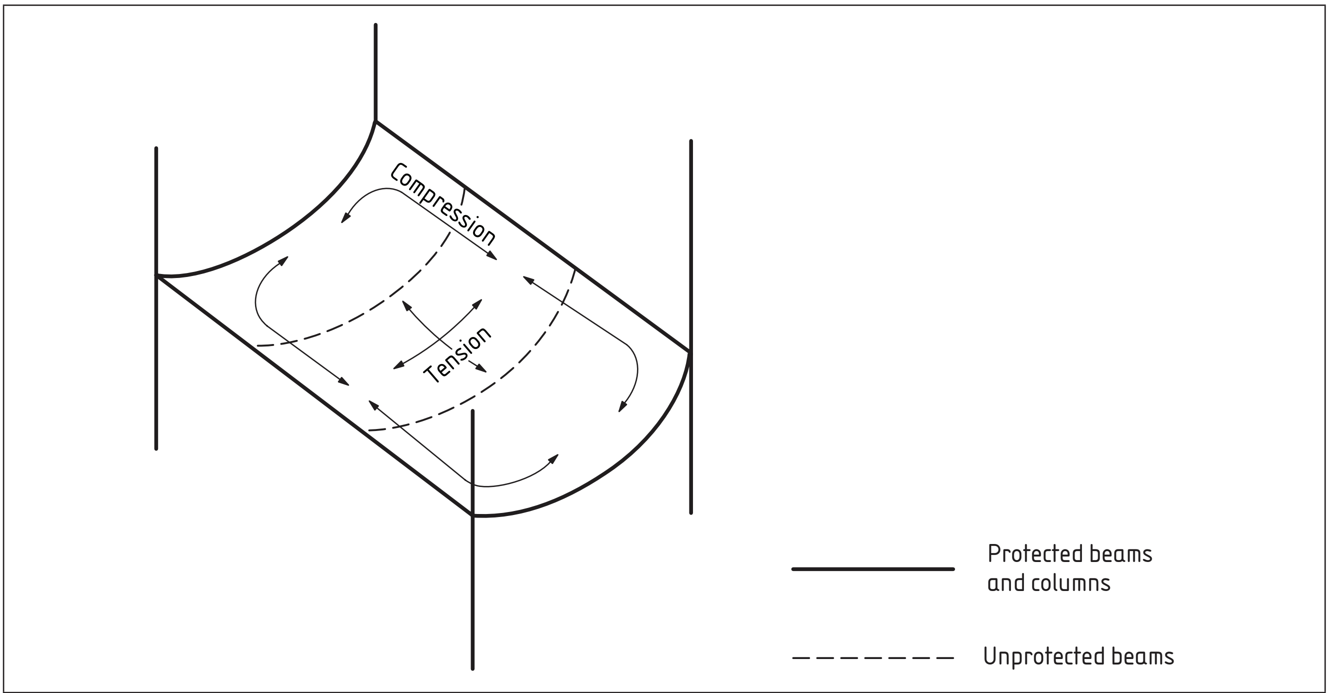
In 1990 a serious fire occurred in the Broadgate Phase 8 Building in London. The building was still under construction and the fireproofing had not yet been applied to the steel frame. The fire spread over a significant area of the floors, yet the structure remained stable and repairs were made. Traditional approaches suggested that the unprotected structure that was exposed to the fire conditions ought to have failed. This alerted the industry to the fact that composite steel-framed buildings have a greater inherent fire resistance than previously assumed. Therefore, six full-scale tests, as opposed to tests on individual members, were conducted on a purpose-built structure to investigate behaviour of real structures.

The test building was an eight-storey composite steel-framed structure built in the Building Research Establishment's (BRE) test facilities at Cardington. The tests were conducted by British Steel (Corus) and BRE. The frame was designed as a typical office building and contained no special features which might favourably affect its response during the fire tests.

An important fact to consider with the Cardington tests is that no protection was provided to the beams whilst atmosphere temperatures of up to 1 200 °C and steel temperatures of up to 1 100 °C were experienced. It had previously been assumed that steel beams would fail at a temperature of approximately 620 °C. At two hours the furnace temperature in a standard furnace test is approximately 1 050 °C, 150 °C lower than the atmosphere temperatures that were experienced during some of the Cardington tests.

It was evident from the Cardington tests that the performances of the concrete slab and its reinforcement are crucial to the overall survival of the floor system, due to tensile membrane action. This is an alternative structural mode and increases the distance that a slab can span at fire limit state. At the high deflections in fire conditions, the concrete slab supports the majority of the gravity loads. In order to mobilize tensile membrane action, the floor should be considered as a series of rectangular design zones. The edges of each design zone are supported vertically by protected columns and/or beams, whilst the beams within the floor zones can remain unprotected. This results in a tension zone in the central portions of the floor design zone (enabled by the tensile capacity of the anti-crack mesh within the slab). If the edges of the floor design zone are simply supported, the supports do not anchor the tensile action and a compression ring forms around the edges of the floor design zone (see Figure 32).

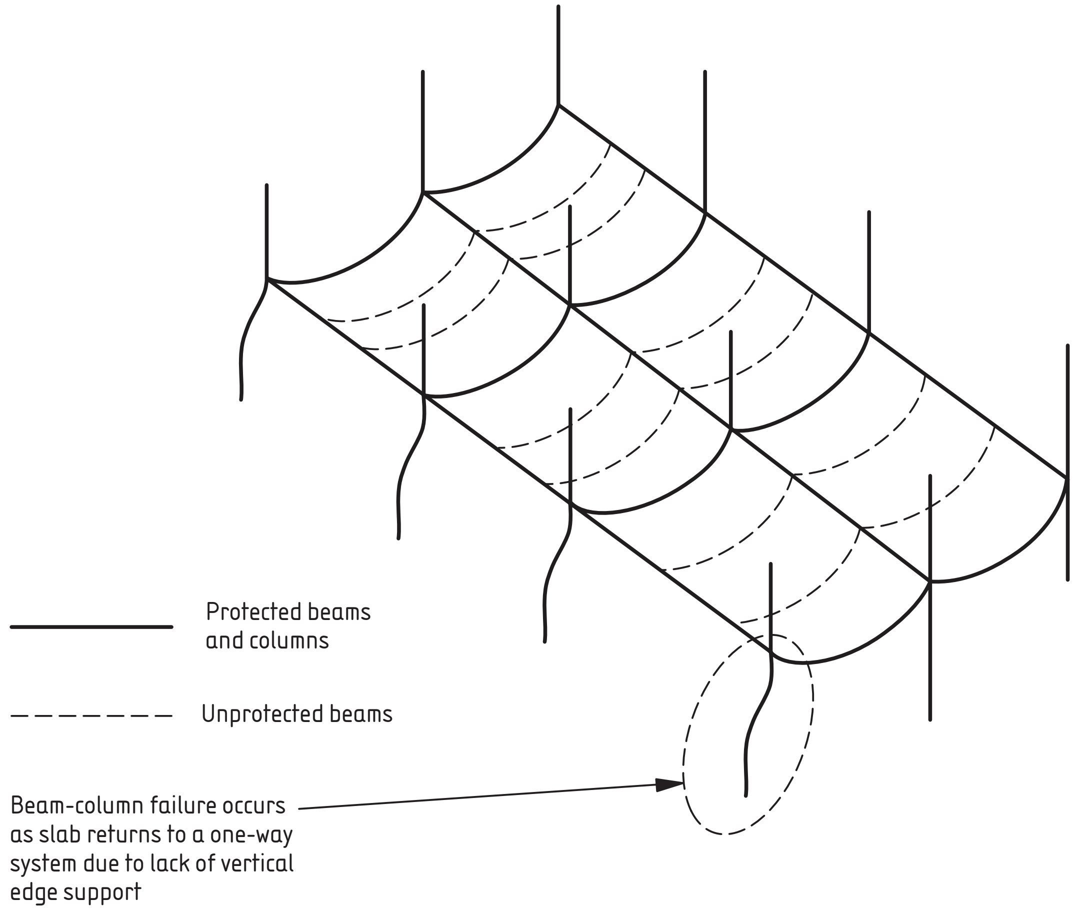
Figure 32 **Schematic representation of the compressive and tensile forces of a floor zone during fire**



The behaviour of the system changes as a fire progresses. Under ambient conditions, the floor slab spans in one direction between secondary beams. The secondary beams transfer the load into primary beams and columns. Initially, the exposed steel beams heat rapidly and expand, with little reduction in strength. The concrete slab heats more slowly, causing thermal bowing towards the heat source. Progressive reduction in steel strength and stiffness then causes very high compressive strains in the steel beams. Restraint to thermal expansion further increases this compressive straining.

As the temperature of the exposed beams increases, they begin to lose strength and deflect. At this point, the performance of the slab to which they are attached plays an increasingly important role in supporting the floor loads. The characteristics of the slab, together with the way it is supported, control the way in which it carries loads. The slab's residual flexural strength can, at this stage, be great enough for it to carry the load at low deflections between protected beams. If the slab is well supported against vertical deflection along lines which divide it into reasonably square areas, tensile membrane action can be generated as a load-carrying mechanism. The slab is then forced into double-curvature and hangs as a tensile membrane in its middle regions, while a peripheral compressive "ring beam" is generated. This forms a self-equilibrating mechanism which supports the slab loading. As the temperature increases, the slab continues to deflect and this can lead to a tensile fracture within the slab. The overall stability of the system relies on the vertical supports at the edge of the tensile membrane zone. If the temperature of these members reaches a point at which they are no longer able to support the applied load, they begin to deflect, tensile membrane action is lost, and a structural collapse could occur (see Figure 33).

Figure 33 **Illustration of the defection of a multi-zone composite floor system with protected and unprotected members**

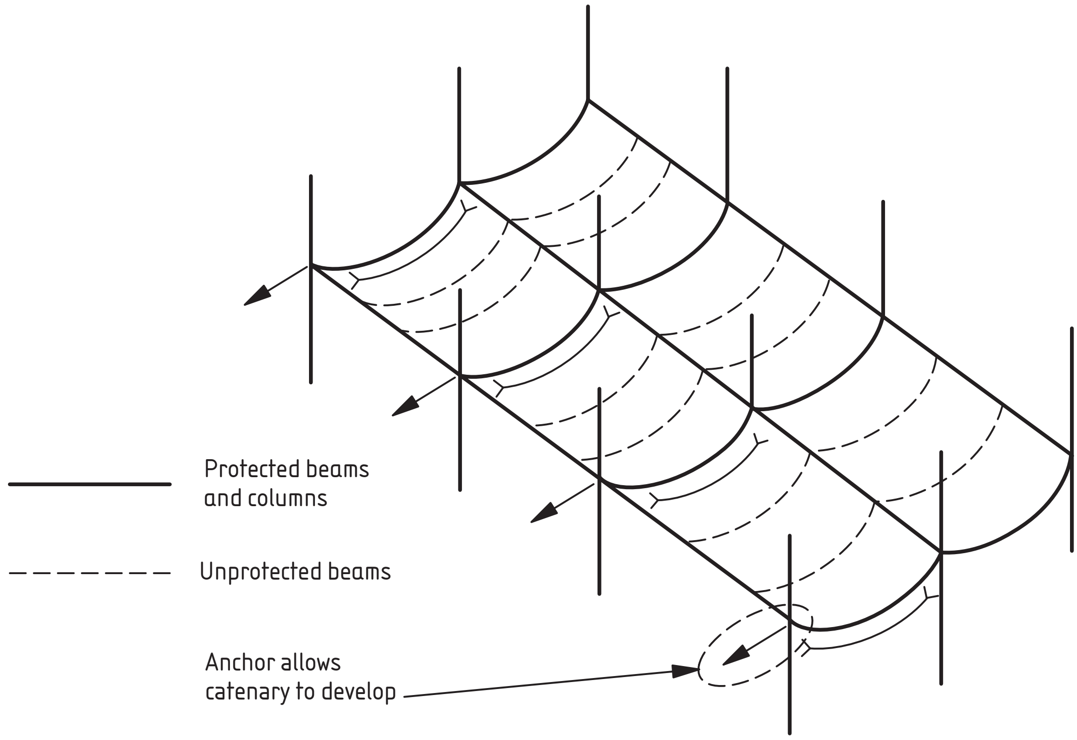


If, however, the slab's support is such that it is one-way-spanning, including situations where the supported edges form a rectangle with a high aspect ratio, then it hangs in single curvature from its longer supported edges. This is catenary action, distinguished from tensile membrane action by the fact that it is not self-equilibrating but needs to be anchored in the horizontal sense at the support edges of the slab (see Figure 34).

There are different methods available for determining the effects of tensile membrane action. Whichever method is adopted a number of checks should be made. The capacity of the slab to support the applied load under tensile membrane action should be checked. This is a function of the tensile capacity of the reinforcement in the central zone of the slab and the aspect ratio of the floor design zone (the closer the ratio is to unity, the greater the capacity).

The vertical support at floor design zone edges is critical to the overall stability of the system and should be checked. Even though a beam is protected, it is not guaranteed against deflection. Once a beam reaches its failure temperature it fails relatively quickly. Therefore, it is important to consider the performance of the edge beams and the margin of safety against failure.

Figure 34 **Illustration of catenary action developed in a multi–zone composite floor system**



If the floor is a compartment floor its integrity should be maintained. Therefore, it is important to consider the deflections, curvature and strains that can occur within the slab at fire limit state, both in the middle of the slab and in hogging over the protected beams.

Protected and unprotected beams expand and contract at different rates. These different rates of expansion can result in large connection forces, particularly where beams connect into rigid structures such as concrete cores.

Failure of the floor system can manifest itself as one or a combination of the following:

- a tension crack in the centre of the slab;
- a large crack due to high hogging moments over the beams at the edge of the floor design zone;
- local crushing of the slab at the corners of the floor design zone as a result of the high compression forces generated around the edge of the floor design zone;
- local failure in beams due to high compression generated around the edge of the floor design zone;
- connection failure due to the high compressions and tensions that result from differential heating and expansion of protected and unprotected beams; and

  NOTE 1 This can occur during the heating or cooling phases.
- failure of protected secondary beams to support the applied load in bending.

  NOTE 2 This is likely to result in a catastrophic failure as the system transfers from tensile membrane action into catenary action. Anchorage to the catenary action is mainly provided by the *sway-stiffness of the perimeter columns at the fire floor and the floor above, and since the columns are being heated, they lose stiffness and can buckle inwards as the floor deflects.*

12.3.14.2 Empirical data from testing

The Steel Construction Institute (SCI) document, *Fire Safe Design – A new approach to multi-storey steel framed buildings* [49] provides tabulated data for determining the reinforcement requirements within the slab and the performance requirements of the protected beams. The tables are for standard fire exposure only and provide pass-fail information. Deflections and connection forces are not predicted so it is not possible to determine the margin of safety that is achieved by the design.

Examination of the tables shows that the slab performance can be significantly increased by increasing the amount of reinforcement. The designer should ensure that this additional capacity is realistic for slabs with a high amount of reinforcement.

12.3.14.3 Simplistic calculation methods

The data within the SCI tables are based on a yield line analysis where the capacity of the slab is enhanced to account for the additional load carrying capacity derived from tensile membrane action. The floor design zone is assumed to act in isolation, so slab edges are assumed to simply support and no account is taken of any continuity over the edge supports. The enhancement factor has been calibrated against the results from a number of fire tests, including those conducted at Cardington. When using this method, the validation should be appropriate for the intended use.

The method determines the amount of deflection required within the slab to generate sufficient tensile membrane action to support the applied load. A deflection limit is used as the pass-fail acceptance criterion. The deflection limit suggested within the documentation is based on BS 476-20 and is applied to the mechanical deflection only (deflections caused by thermal bowing are not included). The method can be used for any fire exposure.

Evaluation of the method shows that the slab performance can be significantly improved by increasing the amount of reinforcement. The designer should ensure that this additional capacity is realistic for slabs with a high amount of reinforcement.

12.3.14.4 Advanced calculation methods

Finite element analysis is often used to predict the performance of composite steel framed structures and determine the most economical structural/fire protection solution. Computer models have shown that the composite metal deck helps reduce the impact of some of the unknowns with regards to the performance of the concrete, and the value achieved is sufficient to justify the expense.

Finite element analysis should be conducted in accordance with the recommendations of **12.3.4.3**.

Finite element analysis software usually adopts a smeared layer model (as opposed to a discrete model) for concrete and reinforcement elements. This means that the reinforcement is effectively treated as a continuous layer throughout the width of the slab rather than a series of discrete bars, assuming that straight normals remain straight. Therefore, effects such as localization and bonding or de-bonding cannot be predicted, so reinforcement fracture cannot be predicted. As a result, the designer should ensure that:

- local behaviour of reinforcement is not important in the context of the analyses;
- local failures of the slab and/or reinforcement does not occur; and
- appropriate acceptance criteria are imposed to protect against local failures.

Annex A (informative) Temperature-dependent properties of construction materials

A.1 General

Annex A provides information on the elevated temperature, thermal and mechanical properties of various materials used in construction. The data have largely been obtained from national and international standards, fire engineering design codes, peer reviewed published papers and manufacturers' literature.

Other data can be used providing they are generated from a bona fide source and represent the behaviour of materials when exposed to the conditions of a fire.

Some elevated temperature properties are influenced by the heating conditions, e.g. steady-state versus transient or the rate of heating, and these can influence the thermal and structural response of building elements. The properties should be obtained under similarly realistic fire-exposure conditions, requiring a considerable investment in research.

A.2 Concrete

A.2.1 Thermal properties of concrete

A.2.1.1 General

Concrete is the generic description for a range of different materials. The thermal properties of each material can vary depending on the concrete mix proportions, the volume of water and the aggregate used. Designers could find modest changes in the thermal properties of different concretes makes little difference to their temperature development when heated, and it is sufficient to characterize the material as:

- ordinary concrete (density > 1 900 kg/m^3);
- lightweight concrete (density ≤ 1 900 kg/m^3); or
- high strength concrete (density >1 900 kg/m^3).

In BS EN 1992-1-2 high strength concrete can adopt the thermal properties of ordinary concrete, although in the technical literature there are significant differences. These data, primarily from Kodur and Sultan, are also included [50].

A.2.1.2 Thermal elongation/contraction of ordinary weight concrete

The thermal elongation of concrete, $\Delta L/L_0$, is governed by the type of aggregate. At concrete temperatures, θ, less than 150 °C, concrete undergoes shrinkage as moisture is driven from the solid matrix.

The thermal elongation, $\Delta L/L_0$, of concrete is given by:

Siliceous aggregates:

$$\Delta L/L_0 = -1.8 \times 10^{-4} + 9 \times 10^{-6}\theta + 2.3 \times 10^{-11} \theta^3 \quad \text{for } 20\ ^\circ\text{C} \le \theta \le 700\ ^\circ\text{C} \qquad \text{(A1)}$$

$$\Delta L/L_0 = 14 \times 10^{-3} \text{ for } 700\ ^\circ\text{C} < \theta \le 1\,200\ ^\circ\text{C} \qquad \text{(A2)}$$

In simple calculation models the thermal elongation for siliceous aggregates can be determined from:

$$\Delta L/L_0 = 18 \times 10^{-6} (\theta - 20) \qquad (A3)$$

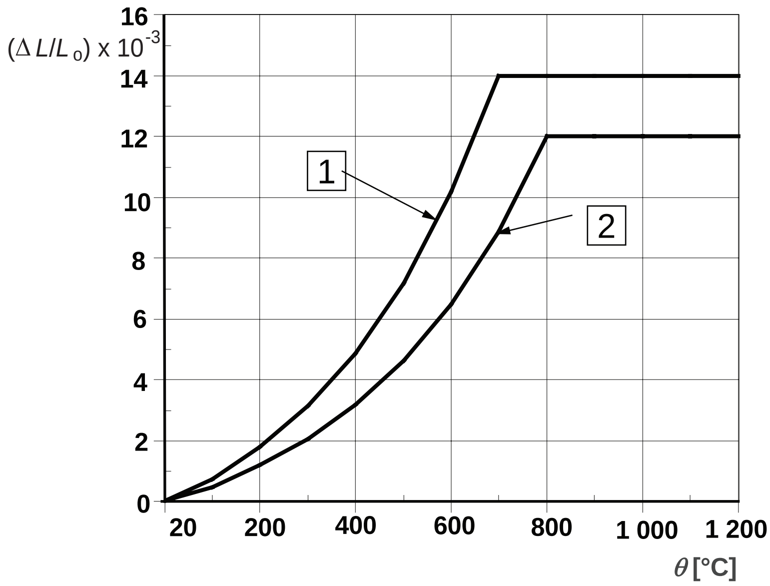
Calcareous aggregates:

$$\Delta L/L_0 = -1.2 \times 10^{-4} + 6 \times 10^{-6}\,\theta + 1.4 \times 10^{-11}\,\theta^3$$
$$\text{for } 20\ °C \le \theta \le 805\ °C \qquad (A4)$$

$$\Delta L/L_0 = 12 \times 10^{-3}\,\theta \text{ for } 805\ °C < \theta \le 1\,200 °C \qquad (A5)$$

The variation in thermal elongation with temperature is given in Figure A.1.

Figure A.1 **Variation in thermal strain with temperature for siliceous and calcareous concrete**



Key

1 Siliceous aggregate

2 Calcareous aggregate

A.2.1.3 Thermal elongation/contraction of lightweight concrete

For lightweight concrete the variation of elongation with temperature, $\Delta L/L_0$, is given as a linear relationship by:

$$\Delta L/L_0 = 8 \times 10^{-6}(\theta - 20) \qquad (A6)$$

A.2.1.4 Thermal elongation/contraction of high strength concrete

The thermal strain of high strength siliceous and calcareous concrete at elevated temperature is given as follows [50]:

Siliceous aggregate concrete:

$$\Delta L/L_0 = -2.0 \times 10^{-4} + 11 \times 10^{-6}\,\theta \text{ for } 0\ °C \le \theta \le 450\ °C \qquad (A7)$$

$$\Delta L/L_0 = -11.5 \times 10^{-3} + 36 \times 10^{-6}\,\theta \text{ for } 450\ °C < \theta \le 650\ °C \qquad (A8)$$

$$\Delta L/L_0 = 11.9 \times 10^{-3}\theta \text{ for } 650\ °C < \theta \le 1\,000\ °C \qquad (A9)$$

Calcareous aggregate concrete:

$$\Delta L/L_0 = -2.0 \times 10^{-4} + 8 \times 10^{-6}\,\theta \text{ for } 0\ °C \le \theta \le 450\ °C \qquad (A10)$$

$$\Delta L/L_0 = -6.1 \times 10^{-3} + 21 \times 10^{-6}\,\theta \text{ for } 450\ °C < \theta \le 920\ °C \qquad (A11)$$

$$\Delta L/L_0 = 24.2 \times 10^{-3} - 12 \times 10^{-6}\,\theta \text{ for } 920\ °C < \theta \le 1\,000\ °C \qquad (A12)$$

A.2.1.5 Specific heat of normal weight concrete

The specific heat, $C_c\,(\theta)$, (J/kg K) of dry concrete (moisture = 0%) for siliceous and calcareous aggregates is given by:

$$C_c\,(\theta) = 900 \text{ for } 20\ °C \le \theta \le 100\ °C \qquad (A13)$$

$$C_c\,(\theta) = 900 + (\theta - 100) \text{ for } 100\ °C < \theta \le 200\ °C \qquad (A14)$$

$$C_c\,(\theta) = 1\,000 + (\theta - 200)/2 \text{ for } 200\ °C < \theta \le 400\ °C \qquad (A15)$$

$$C_c\,(\theta) = 1\,100 \text{ for } 400\ °C < \theta \le 1\,200\ °C \qquad (A16)$$

For simple calculation models, the specific heat can be considered independently of temperature, with an assumed value of 1 000 J/kg K.

Where moisture is not explicitly considered in the calculation method, the specific heat of concrete can be modelled by a constant value, $C_{c.peak}$ (the peak value for C_c in Figure A.2), between 100 °C and 115 °C, with a linear decrease between 115 °C and 200 °C.

$C_{c,peak}$ = 900 for moisture content of 0% of concrete weight;

$C_{c,peak}$ = 1 470 for moisture content of 1.5% of concrete weight;

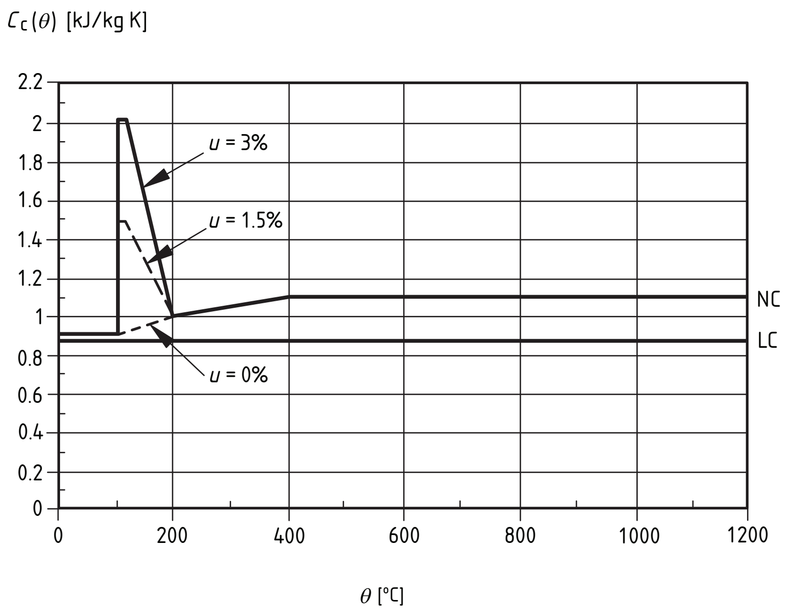
$C_{c,peak}$ = 2 020 for moisture content of 3.0% of concrete weight;

$C_{c,peak}$ = 5 600 for moisture content of 10.0% of concrete weight.

The latter could occur in concrete-filled hollow sections in which the free moisture is unable to escape.

Figure A.2 illustrates the variation of specific heat with temperature.

Figure A.2 **Variation of specific heat with temperature for normal weight concrete (NC) and lightweight concrete (LC) as a function of temperature**



Key

u is moisture content as a percentage of concrete weight

A.2.1.6 Specific heat of lightweight concrete

For lightweight concrete, the specific heat = 840 J/kg K and can be considered independently of temperature (see Figure A.2).

A.2.1.7 Thermal capacity of high strength concrete

Kodur [50] provides further data on the thermal capacity, $C_c \times \rho_c$, (Jm^3/kg^2K), of high strength concrete:

For siliceous concrete:

$$C_c \times \rho_c = (0.005\theta + 1.70) \times 10^6 \text{ for } 0\ °C \leq \theta \leq 200\ °C \quad (A17)$$

$$C_c \times \rho_c = 2.70 \times 10^6\theta \text{ for } 200\ °C < \theta \leq 400\ °C \quad (A18)$$

$$C_c \times \rho_c = (0.013\ \theta - 2.50) \times 10^6 \text{ for } 400\ °C < \theta \leq 500\ °C \quad (A19)$$

$$C_c \times \rho_c = (0.005\ \theta + 1.70) \times 10^6 \text{ for } 500\ °C < \theta \leq 600\ °C \quad (A20)$$

$$C_c \times \rho_c = 2.70 \times 10^6\theta \text{ for } 600\ °C < \theta \leq 1\,000\ °C \quad (A21)$$

For calcareous concrete:

$$C_c \times \rho_c = 2.45 \times 10^6 \text{ for } 0\ °C \leq \theta \leq 400°C \quad (A22)$$

$$C_c \times \rho_c = (0.026\ 0\ \theta - 12.850) \times 10^6 \text{ for } 400\ °C < \theta \leq 475\ °C \quad (A23)$$

$$C_c \times \rho_c = (0.014\ 3\ \theta - 6.295) \times 10^6 \text{ for } 475\ °C < \theta \leq 650\ °C \quad (A24)$$

$$C_c \times \rho_c = (0.189\ 4\ \theta - 120.11) \times 10^6 \text{ for } 650\ °C < \theta \leq 735\ °C \quad (A25)$$

$$C_c \times \rho_c = (-0.263\ 0\ \theta + 212.140) \times 10^6 \text{ for } 735\ °C < \theta \leq 800\ °C \quad (A26)$$

$$C_c \times \rho_c = 2.00 \times 10^6\theta \text{ for } 800\ °C < \theta \leq 1\,000\ °C \quad (A27)$$

A.2.1.8 Thermal conductivity of normal weight concrete

The thermal conductivity, λ_c (W/mK), of normal weight concrete is between the following limits.

Upper limit:

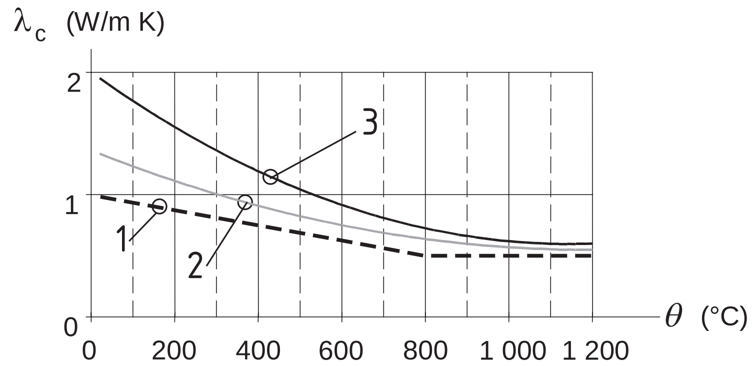
$$\lambda_c = 2.0 - 0.2451\ (\theta/100) + 0.010\ 7\ (\theta/100)^2 \text{ for } 20\ °C \leq \theta \leq 1\,200\ °C \quad (A28)$$

Lower limit:

$$\lambda_c = 1.36 - 0.136\ (\theta/100) + 0.005\ 7\ (\theta/100)^2 \text{ for } 20\ °C \leq \theta \leq 1\,200\ °C \quad (A29)$$

The variation of the upper and lower limits of thermal conductivity with temperature is shown in Figure A.3.

Figure A.3 **Variation of thermal conductivity of concrete with temperature**



Key

1 Light weight concrete
2 Normal weight concrete lower limit
3 Normal weight concrete upper limit

For simple calculation models the thermal conductivity can be considered independently of temperature in which:

$$\lambda_c = 1.60 \text{ W/mK} \quad \text{(A30)}$$

A.2.1.9 Thermal conductivity of lightweight concrete

For lightweight concrete the thermal conductivity can be determined from:

$$\lambda_c = 1.0 - (\theta/1\,600) \text{ for } 20\ °\text{C} \le \theta \le 800\ °\text{C} \quad \text{(A31)}$$

$$\lambda_c = 0.5 \text{ for } \theta > 800\ °\text{C} \quad \text{(A32)}$$

NOTE See Figure A.3.

A.2.1.10 Thermal conductivity of high strength concrete

The thermal conductivity of high strength siliceous and calcareous concrete is given by [50]:

Siliceous concrete:

$$\lambda_c = 2.0 - 0.001\,1\theta \text{ for } 0\ °\text{C} \le \theta \le 1\,000\ °\text{C} \quad \text{(A33)}$$

Calcareous concrete:

$$\lambda_c = 2.00 - 0.001\,3\theta \text{ for } 0\ °\text{C} \le \theta \le 300\ °\text{C} \quad \text{(A34)}$$

$$\lambda_c = 2.21 - 0.002\,0\theta \text{ for } 300\ °\text{C} < \theta \le 1\,000\ °\text{C} \quad \text{(A35)}$$

A.2.1.11 Density of normal weight concrete

The variation of the density of normal weight concrete, ρ_c (kg/m^3), is influenced by the loss in free moisture and can be described by:

$$\rho_c(\theta) = \rho_c(20\ °\text{C}) \text{ for } 20\ °\text{C} \le \theta \le 115\ °\text{C} \quad \text{(A36)}$$

$$\rho_c(\theta) = \rho_c(20\ °\text{C}) \times \{1 - 0.02(\theta - 115)/85\} \text{ for } 115°\text{C} < \theta \le 200\ °\text{C} \quad \text{(A37)}$$

$$\rho_c(\theta) = \rho_c(20\ °\text{C}) \times \{0.98 - 0.03(\theta - 200)/200\} \text{ for } 200\ °\text{C} < \theta \le 400\ °\text{C} \quad \text{(A38)}$$

$$\rho_c(\theta) = \rho_c(20\ °\text{C}) \times \{0.95 - 0.07(\theta - 400)/800\} \text{ for } 400\ °\text{C} < \theta \le 1\,200\ °\text{C} \quad \text{(A39)}$$

Alternatively, the variation in density can be described by:

$$\rho_c = 2\,354 - 23.47(\theta/100) \quad \text{(A40)}$$

A.2.1.12 Density of lightweight concrete

The density for un-reinforced lightweight concrete for structural fire design is in the range of:

$$\rho_c = 1\,600 \text{ kg/m}^3 \text{ to } 2\,000 \text{ kg/m}^3 \quad \text{(A41)}$$

A.2.1.13 Emissivity of concrete

For all concrete types the emissivity of concrete, $\varepsilon_{s,c}$ can be taken as 0.8.

A.2.2 Mechanical properties of concrete

A.2.2.1 General

Designers should also consider the susceptibility of concrete to destructive spalling when exposed to fire. Concretes of higher strengths, lesser permeability, higher moisture content and loaded to higher stress levels are susceptible to surface damage and spalling when heated (see Connolly [46]).

High strength concrete is more susceptible to spalling due to its low permeability, which causes the build up of pore pressure during heating. The extremely high water vapour pressure cannot escape and, at 3000 °C, it can build up to 8 MPa.

The three main types of concrete spalling are:

- surface spalling which affects aggregate on the concrete's surface resulting in concrete fragments up to 20 mm in diameter becoming detached;
- corner break-off or sloughing off which occurs in the later stages of a fire and affects vulnerable concrete that is heated on two planes, e.g. wall corners; and
- explosive spalling where early, rapid heat-rise forcibly and explosively separates pieces of concrete at high pressure.

Explosive spalling is the most dangerous form of spalling and is primarily caused by the build-up of water vapour pressure in concrete during fire. If the concrete is not very permeable, water vapour formed within the pores during heating is unable to dissipate and high pressures build. When the pressures exceed the tensile strength of the concrete, explosive spalling occurs. Concrete is more susceptible to spalling when under rapid heating conditions as opposed to heat exposure over a period of time. Concrete made with limestone, lightweight and/or air-dried aggregate is less susceptible to spalling than that made with siliceous aggregate. Practical methods can be employed to mitigate against spalling, e.g. adding polypropylene (PP) fibres into the concrete mix. The heat from a fire melts the polypropylene fibres, creating passageways along which water vapour can dissipate, avoiding a build-up of pressure.

Concrete is less susceptible to spalling when sprayed with a coating that slows down the rate of heat transfer from fire. The rate of temperature change in the concrete is as important in the causation of spalling as ongoing exposure to high temperature itself.

Attaching a preformed thermal barrier over the concrete surface, a method sometimes used in tunnel construction, can also prevent spalling.

To counteract the risk of spalling, vents in the concrete can be created to alleviate pore pressure.

A.2.2.2 Strength characteristics, elastic modulus and stress-strain behaviour

A.2.2.2.1 General

Guidance on the elevated temperature behaviour of reinforced concrete during a fire is given in BS EN 1992-1-2. The design procedure given in BS EN 1992-1-2 is a modification of the conventional ambient temperature approach to structural analysis. The strength of concrete is reduced significantly at temperatures above 350 °C.

*NOTE BS EN 1994-1-2 also provides data on lightweight concrete (see **A.2.2.2.4**).*

In the design of concrete elements, temperature-dependent strength reduction factors are applied to heated components. The strength reduction factors depend on the rate of heating under which they have been determined so the methodology is not always applicable to concrete elements exposed to non-standardised fire conditions. Pre-stressed and post tensioned concrete members also require special consideration.

The following data can be applied to heating rates between 2 K and 50 K per minute and cover the majority of heating conditions during a fire.

A.2.2.2.2 Normal weight concrete – compressive strength

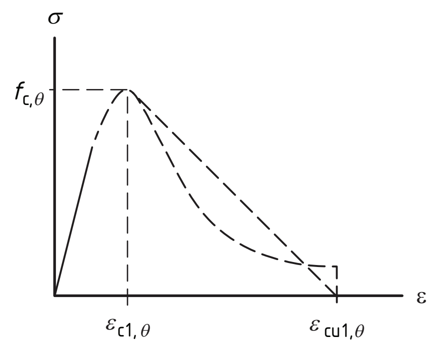
The strength and deformation properties of uni-axially stressed, normal weight concrete at elevated temperatures, is obtained from the stress-strain relationships as shown in Figure A.4.

These can be described by two parameters where:

$f_{c,\theta}$ is the compressive strength; and

$\varepsilon_{c,\theta}$ is the strain corresponding to $f_{c,\theta}$.

Figure A.4 **Mathematical model for stress-strain relationships under compression at elevated temperatures (see BS EN 1992-1-2:2004, Figure 3.1)**



For range $\varepsilon \leq \varepsilon_{c1,\theta}$ stress $\left[\sigma(\theta)\right] = \dfrac{3\varepsilon f_{c,\theta}}{\varepsilon_{c1,\theta}\left[2+\left(\dfrac{\varepsilon}{\varepsilon_{c1,\theta}}\right)^3\right]}$

For range $\varepsilon_{c1,\theta} < \varepsilon \leq \varepsilon_{cu1,\theta}$ a descending branch should be adopted for numerical purposes.

NOTE Linear or non-linear models are permitted.

Values for $\varepsilon_{cu1,\theta}$ defining the range of the descending branch can be taken from Table A.1, column 4 for normal weight concrete with siliceous aggregates and column 7 for normal weight concrete with calcareous aggregates.

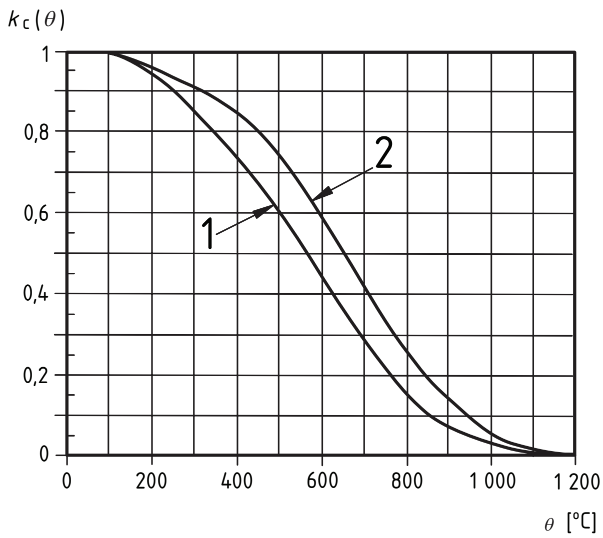
The parameters given in Table A.1 can be used for normal weight concrete with siliceous aggregates, or calcareous aggregates containing at least 80% calcareous aggregate by weight.

Table A.1 **Values for the main parameters of the stress-strain relationships of normal weight concrete with siliceous or calcareous aggregates (see BS EN 1992-1-2:2004, Table 3.1)**

Concrete temperature θ	Siliceous aggregates			Calcareous aggregates		
°C	$f_{c,\theta} / f_{c,k}$ [–]	$\varepsilon_{c1,\theta}$ [–]	$\varepsilon_{cu1,\theta}$ [–]	$f_{c,\theta} / f_{c,k}$ [–]	$\varepsilon_{c1,\theta}$ [–]	$\varepsilon_{cu1,\theta}$ [–]
1	2	3	4	5	6	7
20	1.00	0.0025	0.0200	1.00	0.0025	0.0200
100	1.00	0.0040	0.0225	1.00	0.0040	0.0225
200	0.95	0.0055	0.0250	0.97	0.0055	0.0250
300	0.85	0.0070	0.0275	0.91	0.0070	0.0275
400	0.75	0.0100	0.0300	0.85	0.0100	0.0300
500	0.60	0.0150	0.0325	0.74	0.0150	0.0325
600	0.45	0.0250	0.0350	0.60	0.0250	0.0350
700	0.30	0.0250	0.0375	0.43	0.0250	0.0375
800	0.15	0.0250	0.0400	0.27	0.0250	0.0400
900	0.08	0.0250	0.0425	0.15	0.0250	0.0425
1 000	0.04	0.0250	0.0450	0.06	0.0250	0.0450
1 100	0.01	0.0250	0.0475	0.02	0.0250	0.0475
1 200	0.00	—	—	0.00	—	—

A comparison of siliceous and calcareous concrete in compression is shown in Figure A.5.

Figure A.5 **Variation in coefficient $k_c(\theta)$ for describing the characteristic strength, $f_{c,k}$, for siliceous and calcareous aggregates at elevated temperatures (see BS EN 1992-1-2:2004, Figure 4.1)**



Key

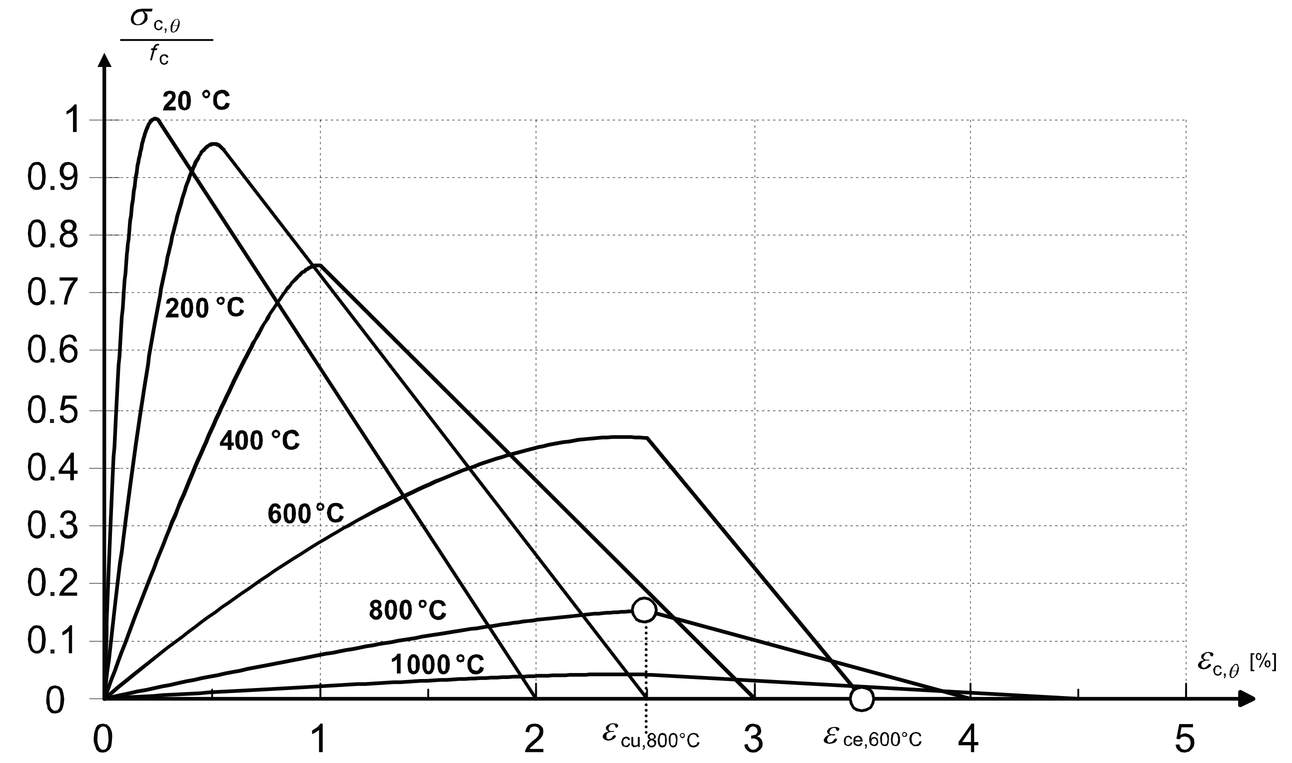
1 Normal weight concrete with siliceous aggregates

2 Normal weight concrete with calcareous aggregates

BS EN 1994-1-2 gives a set of stress-strain curves for siliceous concrete which is shown in Figure A.6.

NOTE BS EN 1994-1-2 uses different symbols for concrete strain from those in BS EN 1992-1-2 (see Key to Figure A.6).

Figure A.6 **Stress-strain relationships for normal weight siliceous concrete at elevated temperatures (see BS EN 1994-1-2:2005, Figure B.1)**



Key

$\varepsilon_{ce,\theta}$ is the maximum concrete strain (c.f. $\varepsilon_{cu1,\theta}$ in Figure A.4)

$\varepsilon_{cu,\theta}$ is the concrete strain corresponding to $f_{c,\theta}$ (c.f. $\varepsilon_{c1,\theta}$ in Figure A.4)

Concrete cooling to an ambient temperature of 20 °C after reaching a maximum temperature of θ_{max} does not recover its initial compressive strength, f_c.

Therefore, for the descending branch of the concrete heating curve, both the values of $\varepsilon_{cu,\theta}$ and the slope of the descending branch of the stress-strain relationship can be maintained equal to the corresponding value for θ_{max}.

The residual compressive strength of concrete after being heated to a maximum temperature θ_{max} and cooled to ambient temperature, 20 °C, can be given as follows:

$$f_{c,\theta,20} = \varphi f_c \quad \text{(A42)}$$

where:

$$\varphi = k_{c,\theta\,max} \text{ for } 20\ °C \le \theta_{max} < 100\ °C \quad \text{(A43)}$$

$$\varphi = 1.0 - [0.235\ (\theta_{max} - 100)/200] \text{ for } 100\ °C \le \theta_{max} < 300\ °C \quad \text{(A44)}$$

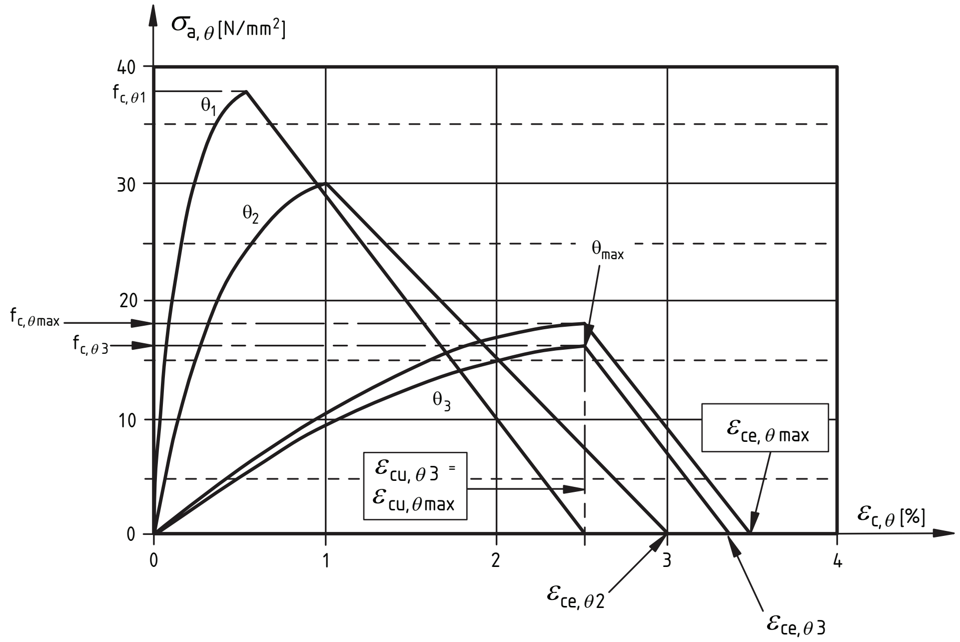
$$\varphi = 0.9\ k_{c,\theta\,max} \text{ for } \theta_{max} \ge 300\ °C \quad \text{(A45)}$$

$k_{c,\theta\,max}$ is the reduction factor.

(During the cooling down of concrete with $\theta_{max} \geq \theta \geq 20$ °C, the corresponding compressive cylinder strength $f_{c,\theta}$ can be interpolated in a linear way between $f_{c,\theta\,max}$ and $f_{c,\theta,20}$.

This is illustrated in Figure A.7 for a grade 40/50 concrete strength.

Figure A.7 **Stress-strain curves allowing for cooling of a grade 40/50 concrete (see BS EN 1994-1-2:2005, Figure C.2)**



A.2.2.2.3 Normal weight concrete – tensile strength

The tensile strength of concrete can usually be ignored. If it is necessary to consider the tensile strength, when using the simplified or advanced calculation method, the following equations can be used in which the reduction of the characteristic tensile strength of concrete is allowed for by the coefficient $k_{c,t}$ (θ):

$$f_{ck,t}(\theta) = k_{c,t}(\theta)\, f_{ck} \quad \text{(A46)}$$

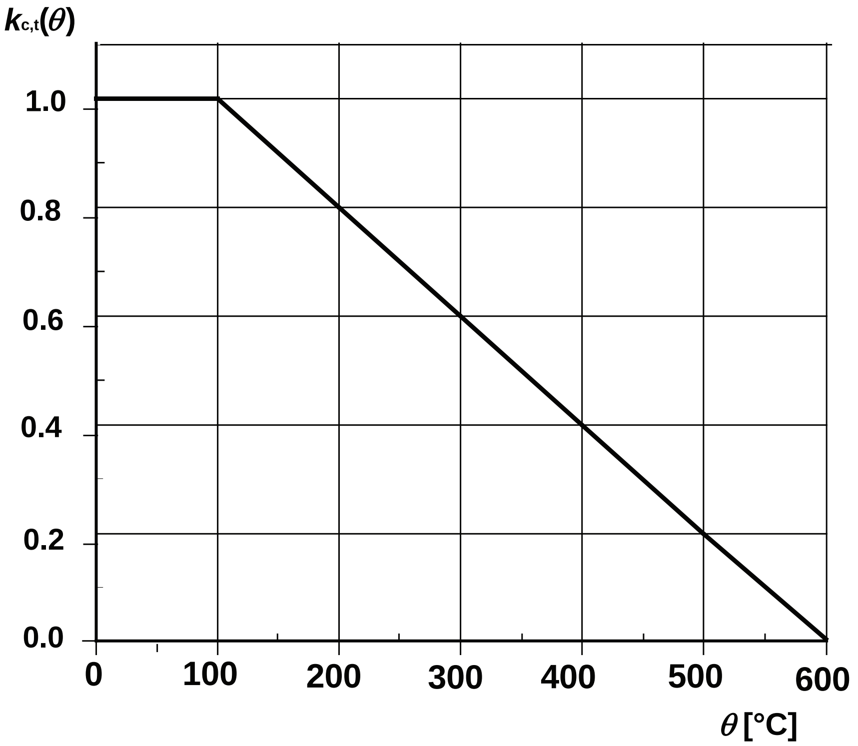
In the absence of more accurate information the following $k_{c,t}$ (θ) values should be used:

$$k_{c,t}(\theta) = 1.0 \text{ for } 20\ °C \leq \theta \leq 100\ °C \quad \text{(A47)}$$

$$k_{c,t}(\theta) = 1.0 - 1.0\,(\theta - 100)/500 \text{ for } 100\ °C < \theta \leq 600\ °C \quad \text{(A48)}$$

Equations A47 and A48 are described in Figure A.8.

Figure A.8 **Coefficient $k_{c,t}$ (θ) allowing for decrease of tensile strength ($f_{ck,t}$) of concrete at elevated temperatures (see BS EN 1992-1-2:2004, Figure 3.2)**



A.2.2.2.4 Lightweight concrete

For lightweight concrete BS EN 1994-1-2 adopts the same stress-strain model used for normal weight concrete as given in **A.2.2.2.2**. However, the values for the reduction in strength $k_{c,\theta} = f_{c,\theta} / f_{c,k}$ of lightweight concrete are different (see Table A.2).

Table A.2 **Values for the two main parameters of the stress-strain relationship – lightweight concrete at elevated temperatures**

Concrete temperature θ_c	**Lightweight concrete**	
°C	$k_{c,\theta} = f_{c,\theta} / f_{c,k}$	$\varepsilon_{cu,\theta} \times 10^3$
20	1.00	2.5
100	1.00	4.0
200	1.00	5.5
300	1.00	7.0
400	0.88	10.0
500	0.76	15.0
600	0.64	25.0
700	0.52	25.0
800	0.40	25.0
900	0.28	25.0
1 000	0.16	25.0
1 100	0.04	25.0
1 200	0	–

The values of $\varepsilon_{cu,\theta}$, if needed, should be obtained from testing carried out in compression.

At elevated temperatures the reduction in strength of lightweight concrete is less than normal weight concrete.

A.2.2.2.5 High strength concrete (HSC)

A.2.2.2.5.1 General

BS EN 1992-1-2 gives information on the strength characteristics of high strength concrete which takes into account the risk of spalling.

In BS EN 1992-1-2 the reduction in the strength properties of high strength concrete at elevated temperature are given in three classes and recommendations for protection against spalling are given for two ranges of HSC. The properties and recommendations are given for fire exposure corresponding to the standard temperature-time curve only.

A reduction in strength, $f_{c,\theta}/\, f_{ck}$, at elevated temperature should be made in accordance with Table A.3 (see also Figure A.9).

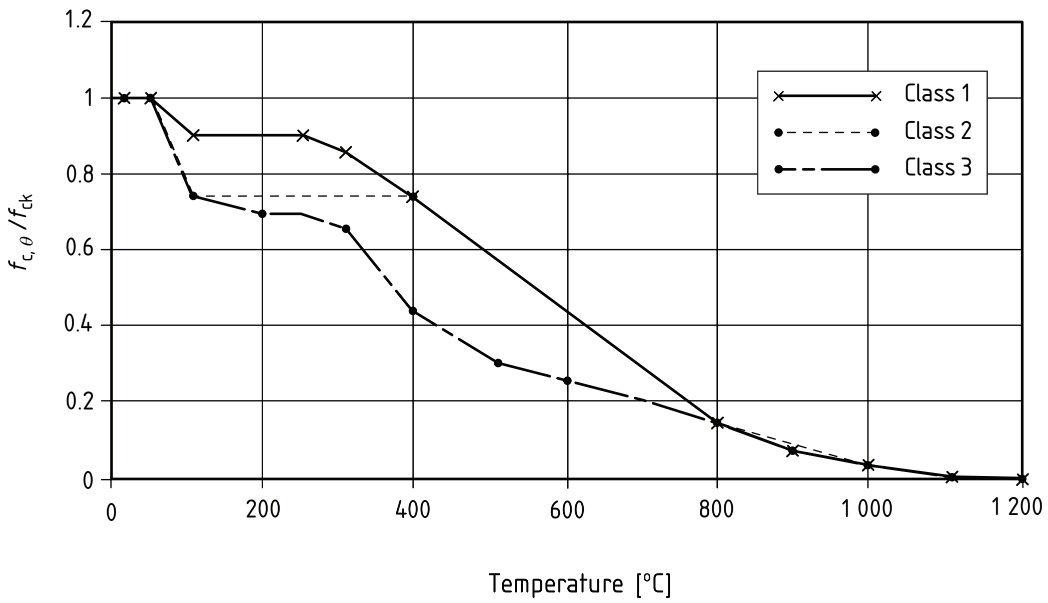
The recommended class for:

- concrete grades C 55/67 and C 60/75 is class 1;
- concrete grades C 70/85 and C 80/95 is class 2; and
- concrete grades C 90/105 is class 3.

Table A.3 **Reduction of strength at elevated temperature**

Concrete temperature	**Strength reduction factor** $f_{c,\theta}/\, f_{ck}$		
°C	**Class 1**	**Class 2**	**Class 3**
20	1	1	1
50	1	1	1
100	0.9	0.75	0.75
200	—	—	0.7
250	0.9	—	—
300	0.85	—	0.65
400	0.75	0.75	0.45
500	—	—	0.3
600	—	—	0.25
700	—	—	—
800	0.15	0.15	0.15
900	0.08	—	0.08
1 000	0.04	—	0.04
1 100	0.01	—	0.01
1 200	0	0	0

Figure A.9 **Reduction of strength at elevated temperature**



A.2.2.2.5.2 Spalling

For concrete grades C 55/67 to C 80/95 the rules in BS EN 1992-1-2:2004, **4.5** apply, provided that the maximum content of silica fume is less than 6% by weight of cement. For higher contents of silica fume the following rules apply.

For concrete grades 80/95 < C ≤ 90/105 at least one of the following prevention methods should be provided:

a) a reinforcement mesh with a nominal cover of 15 mm, wires with a diameter ≥ 2 mm and a pitch ≤ 50 mm × 50 mm;

   NOTE The nominal cover to the main reinforcement should be ≥ 40 mm.

b) a type of concrete that is not susceptible to spalling under fire exposure (demonstrated by local experience or by testing);

c) protective layers that are not susceptible to spalling of concrete under fire exposure;

d) inclusion of more than 2 kg/m^3 of monofilament propylene fibres in the concrete mix.

A.3 Steel, and wrought and cast iron

A.3.1 General

The behaviour of steel, and wrought and cast iron components and members in fire is strongly dependent upon their composition, heat treatment and rolling processes.

Structural steel and steel components can be broadly placed in several groups which fall within one of three categories; carbon steel (e.g. mild and low alloy, high strength steel, reinforcing steels and pre-stressing wires, light gauge steel, steels used for bolts and welds), stainless steel and wrought and cast iron.

The mechanical properties of mild and low alloy steel sections are strongly influenced by their rolling process (hot or cold finished and thermo-mechanically controlled rolled). Open section beams and columns and plate for fabricated sections for buildings are usually hot rolled in accordance with BS EN 10025-1 and BS EN 10025-2, of which mild steel (S275), and high strength steel (S355) are the most widely used. Hollow sections (square, round and rectangular) can be hot or cold formed. Hot rolled hollow sections are usually supplied in accordance with BS EN 10210.

Reinforcing bars and wires can be hot finished and cold twisted and pre-strained high tensile wire, strand and bars should be supplied in accordance with BS EN 10080 and prEN 10138. The elevated temperature behaviour of hot-formed reinforcing bars is similar to that of hot rolled steel sections. In contrast, cold twisted bar and high strength wires obtain their increased strength from strain hardening during manufacture. Consequently, at elevated temperatures, they lose their strength more quickly in proportion to rising temperatures than hot finished products.

Light gauge (hot and cold rolled) products are supplied in sections and profiles, e.g. "C" and "Z" shapes for purlins and side rails, to support profiled steel sheeting in warehouses, steel decking for composite floors, structural support to lightweight partitions, box sections for lintels, joists and hollow sections in lightweight structural framing systems (see BS EN 10346 and BS EN 10218).

Structural bolts and fasteners are manufactured from mild and low alloy steels. They can be hot finished, cold formed or quenched and tempered to achieve their design strength. Bolts and fasteners which are cold formed or heat treated suffer a greater proportional loss in strength than hot finished mild steel components. For load bearing structural frames, grade 4.6, 8.8 and 10.9 bolts are generally used, with grade 8.8 being the most common. Bolts supplied to grade 8.8 and 10.9 are quenched and tempered to achieve their high strength. These loose their strength rapidly (over temper) once their temperature is raised above their tempering temperature (usually 500 °C to 600 °C).

Welds in structural members are designed to have strength properties at least as great as the parent material. The critical area is often in the heat affected zone and the choice of welding processes (method and pre-heat) and electrodes are specified to achieve the desired strength and notch toughness properties.

Stainless steels are available in a wide range of compositions, broadly divided into five groups according to their metallurgical structure. These groups are austenitic, duplex, ferritic, martensitic and precipitation hardened and are designed to suit their end use in the environment where high temperature strength/creep resistance, oxidation and chemical resistance are required. Austenitic and duplex stainless steels are the most widely used in architectural and structural engineering applications.

Wrought and cast iron structural members are no longer manufactured for use in building design but they are commonplace in the refurbishment of old buildings originally built in the eighteenth and nineteenth centuries as well as the turn of the twentieth century. Cast iron is a very brittle and variable material and, whilst it is able to sustain high compressive forces, it performs very poorly in tension. Wrought iron is more ductile and more suitable in tension but it is variable depending upon the residual chemical elements, e.g. phosphorus and sulphur, as well as the manufacturing process routes. In old Victorian buildings it is common to find a combination of cast iron columns and wrought iron beams.

A.3.2 Thermal properties

A.3.2.1 Thermal elongation of carbon steels

Steel progressively expands right up to its melting point unless it undergoes an intermediate phase change.

From approximately 720 °C to 860 °C carbon steels go through a phase transformation from ferrite to austenite steel with little net change in elongation. During this period of heating, ferrite, a body-centred cubic structure, transforms to austenite, a face-centred cubic structure. Since the latter has a more closely packed atomic structure, this results in a contraction of the material. However, the contraction is offset by the expansion of the material as it continues to be heated, and the net effect is little change in thermal strain. In BS EN 1993-1-2 the expansion of steel is given by the relationships:

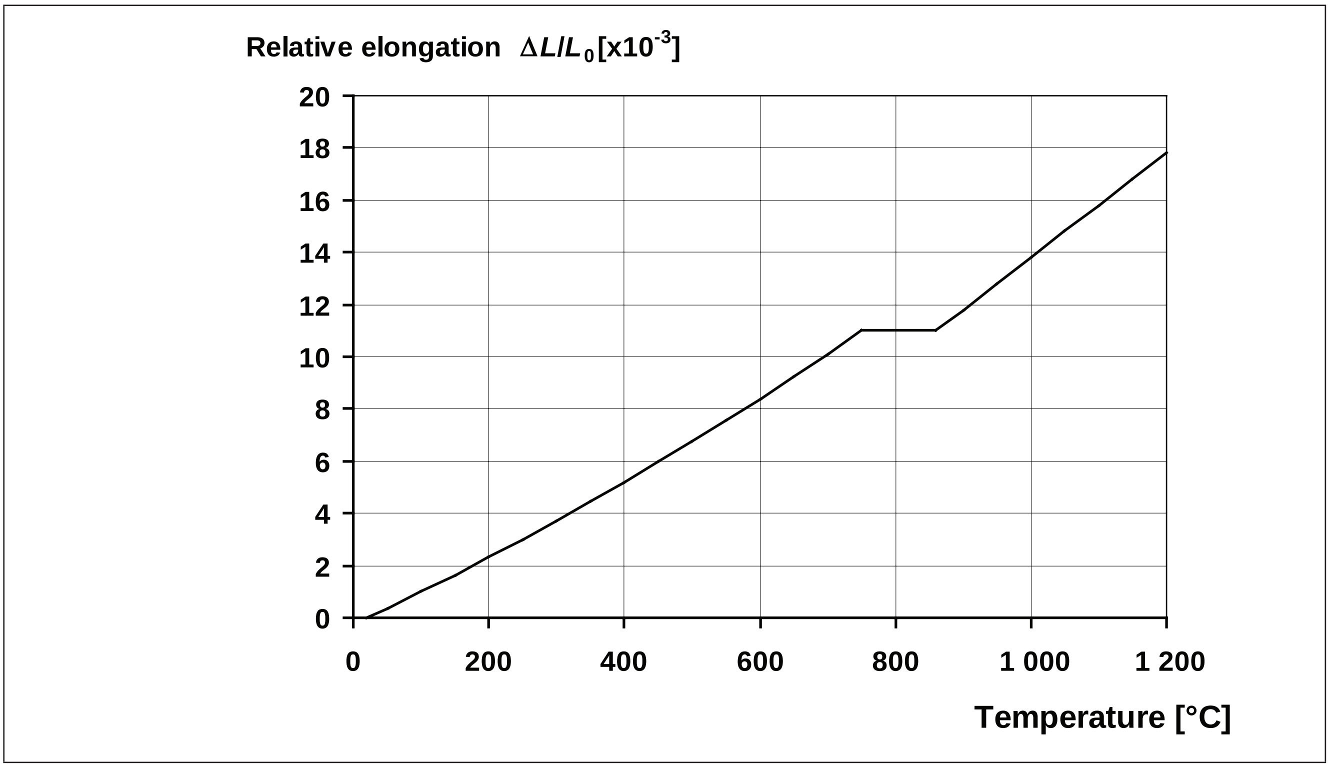
$$\Delta L/L_0 = 1.2 \times 10^{-5}\theta + 0.4 \times 10^{-8}\theta^2 - 2.416 \times 10^{-4} \text{ for } 20\text{ °C} \le \theta < 750\text{ °C} \quad \text{(A49)}$$

$$\Delta L/L_0 = 1.1 \times 10^{-2} \text{ for } 750\text{ °C} \le \theta \le 860\text{ °C} \quad \text{(A50)}$$

$$\Delta L/L_0 = 2 \times 10^{-5}\theta - 6.2 \times 10^{-3} \text{ for } 860\text{ °C} < \theta \le 1\,200\text{ °C} \quad \text{(A51)}$$

These relationships are illustrated in Figure A.10.

Figure A.10 **Thermal elongation of carbon steel as a function of the temperature**



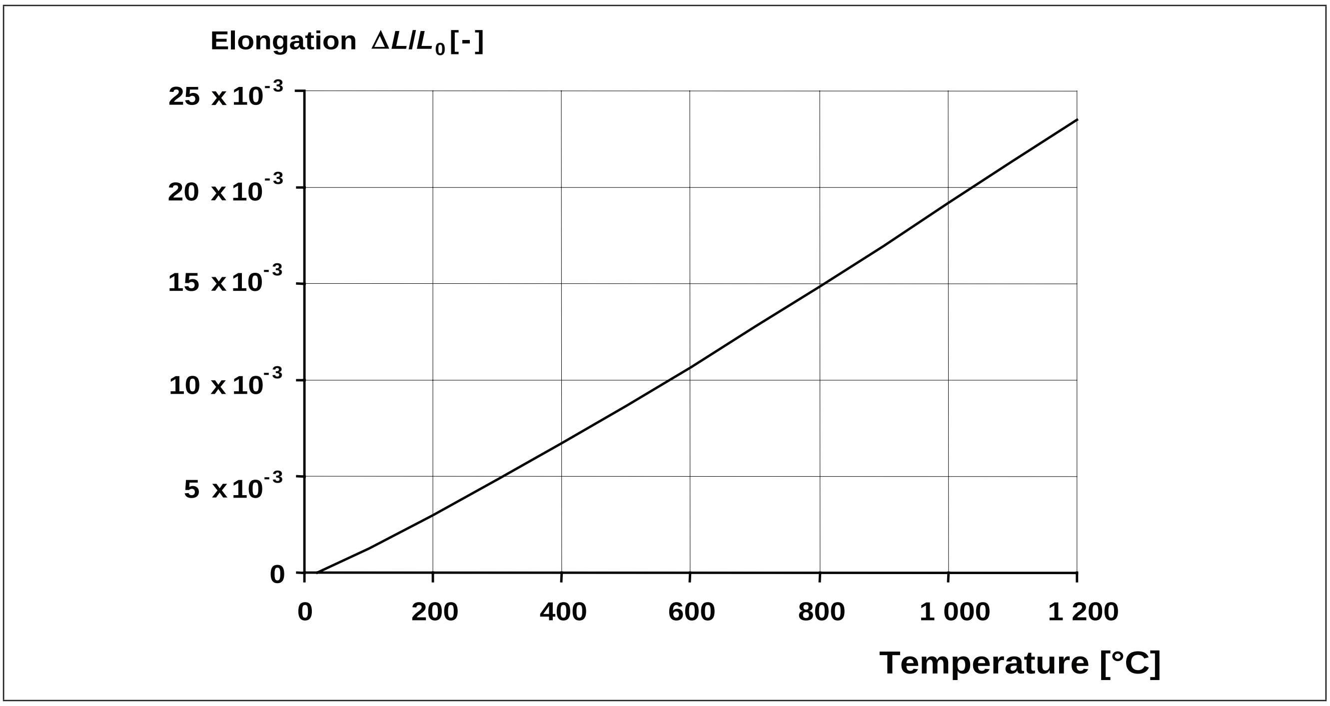
A.3.2.2 Thermal elongation of stainless steel

Austenitic stainless steels do not undergo the same transformation as carbon steels and, therefore, the thermal elongation of austenitic stainless steel $\Delta L / L_0$ can be determined from the following:

$$\Delta L/L_0 = (16 + 4.79 \times 10^{-3}\theta - 1.243 \times 10^{-6}\theta^2) \times (\theta - 20) \times 10^{-6} \quad \text{(A52)}$$

This is illustrated in Figure A.11.

Figure A.11 **Thermal elongation of austenitic stainless steel as a function of temperature**



A.3.2.3 Thermal elongation of cast iron and wrought iron

The thermal elongation of wrought iron or cast iron at elevated temperatures is not well understood. Equation A52 can be used with appropriate caution at temperatures up to 700 °C.

A.3.2.4 Specific heat capacity of carbon steels

For structural steel there is little effect of composition on specific heat capacity. During heating at temperatures of around 715 °C to 730 °C, carbon steels pass through the Curie point. This is a change in magnetic domain from the ordered ferromagnetic to the disordered paramagnetic state. Although this change is partially dependent on composition, it results in a sharp peak in heat capacity over a small temperature range. During heating of a steel member, the change in specific heat capacity with time through the Curie point usually shows as a dwell period in which energy (heat) is absorbed in changing its magnetic domain. During cooling the reverse takes place.

In BS EN 1993-1-2 the relationship between specific heat for carbon steels, C_a, (J/kgK) and temperature is given by:

$$C_a = 425 + 7.73 \times 10^{-1}\theta - 1.69 \times 10^{-3}\theta^2 + 2.22 \times 10^{-6}\theta^3$$
$$\text{for } 20\ °C \le \theta < 600\ °C \quad \text{(A53)}$$

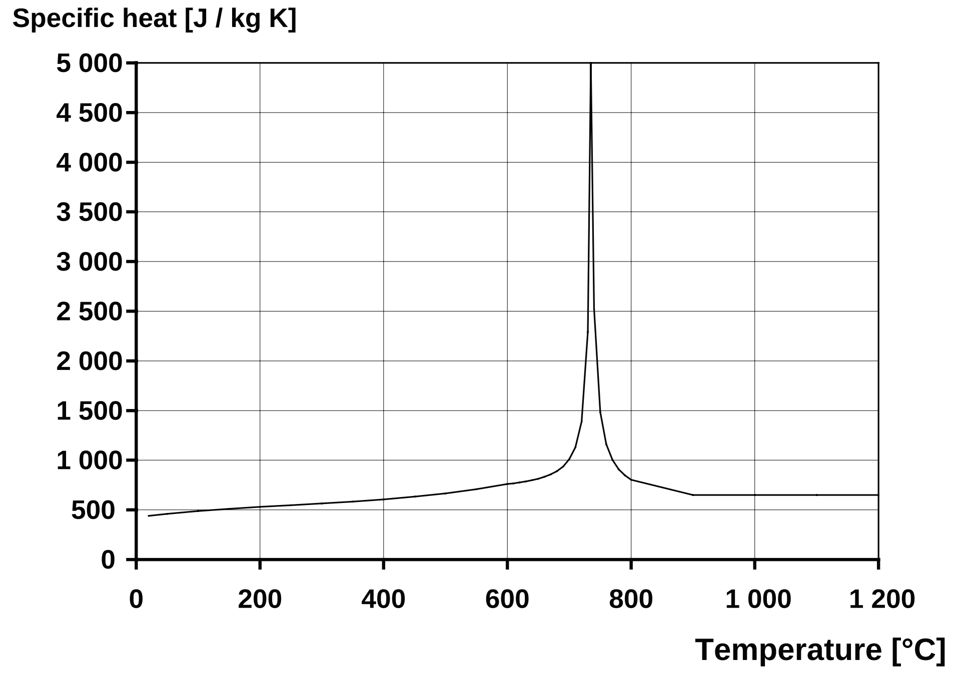
$$C_a = 666 + \{13002/(738 - \theta)\} \text{ for } 600\ °C \le \theta < 735\ °C \quad \text{(A54)}$$

$$C_a = 545 + \{17820/(\theta - 731)\} \text{ for } 735\ °C \le \theta < 900\ °C \quad (A55)$$

$$C_a = 650 \text{ for } 900\ °C \le \theta \le 1200\ °C \quad (A56)$$

The variation of the specific heat with temperature is illustrated in Figure A.12.

Figure A.12 **Specific heat of carbon steels as a function of temperature**



When accurately modelling the thermal response of steel, it is important that the time intervals chosen for heat transfer calculations include time steps within the thermal capacity "spike". However, the height (maximum value) of the spike is not as critical.

For simple calculations the specific heat can be considered to be independent of temperature. In this case an average value of C_a = 600 J/kgK should be used.

A.3.2.5 Specific heat capacity of stainless steel

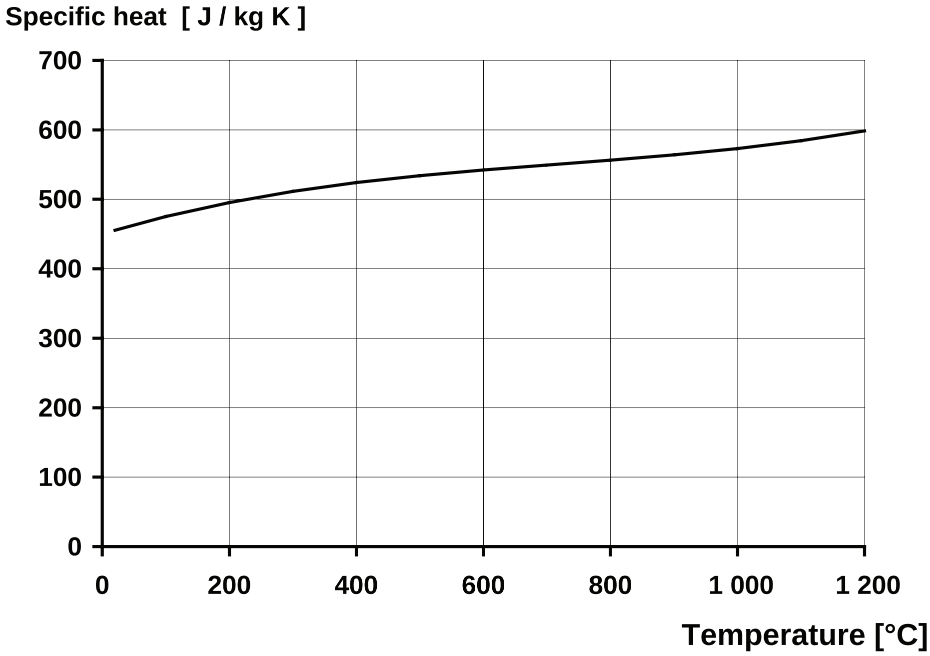
Stainless steel does not undergo the magnetic transformation encountered with carbon steels. Ferritic stainless steels are ferromagnetic and austenitic steels are paramagnetic (non-magnetic).

The specific heat of stainless steels, C_a, can be determined from the following:

$$C_a = 450 + 0.280\theta - 2.91 \times 10^{-4}\theta^2 + 1.34 \times 10^{-7}\theta^3 \quad (A57)$$

The variation of the specific heat with temperature is illustrated in Figure A.13.

Figure A.13 **Specific heat of stainless steels as a function of temperature**



A.3.2.6 Specific heat capacity of cast iron and wrought iron

Little data is available on the specific heat capacity of cast and wrought iron and therefore a value of 600 J/kgK can be used for temperatures up to 700 °C.

A.3.2.7 Thermal conductivity of carbon steels

The variation of thermal conductivity, λ_a, (W/mK) with temperature is influenced by phase changes of the atomic structure and can be approximated by the following relationships:

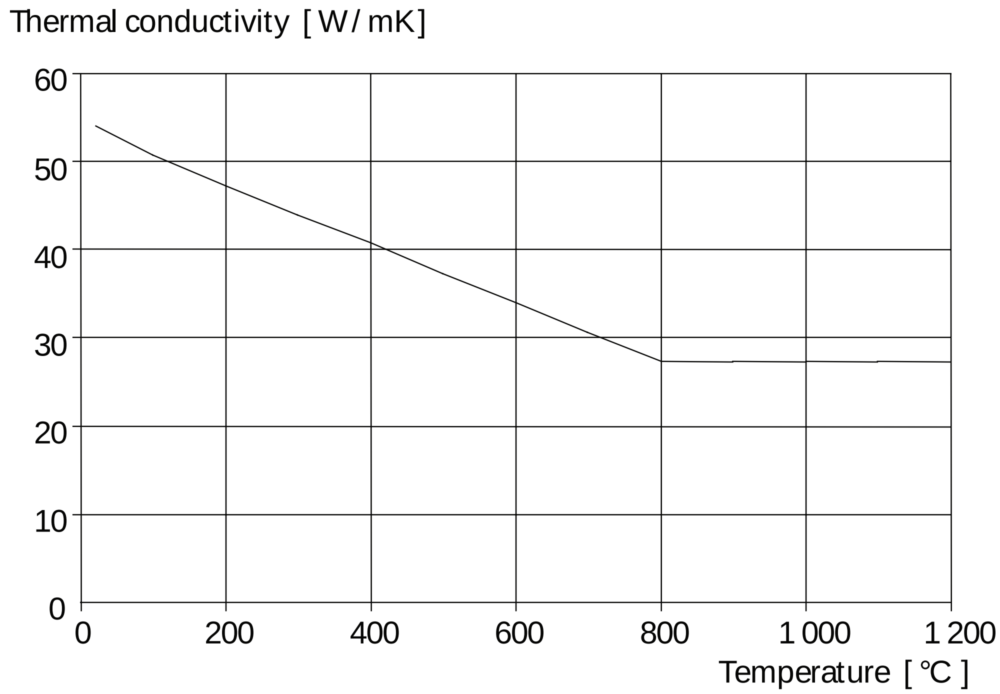
$$\lambda_a = 54 - 3.33 \times 10^{-2}\theta \text{ for } 20\ °C \leq \theta \leq 800\ °C \tag{A58}$$

$$\lambda_a = 27.3 \text{ for } 800\ °C < \theta \leq 1\,200\ °C \tag{A59}$$

The variation of the thermal conductivity with temperature is illustrated in Figure A.14.

For simple calculations, the thermal conductivity can be assumed to be independent of temperature. In this case an average value of λ_a = 45 W/mK should be used.

Figure A.14 **Thermal conductivity of carbon steel as a function of temperature**



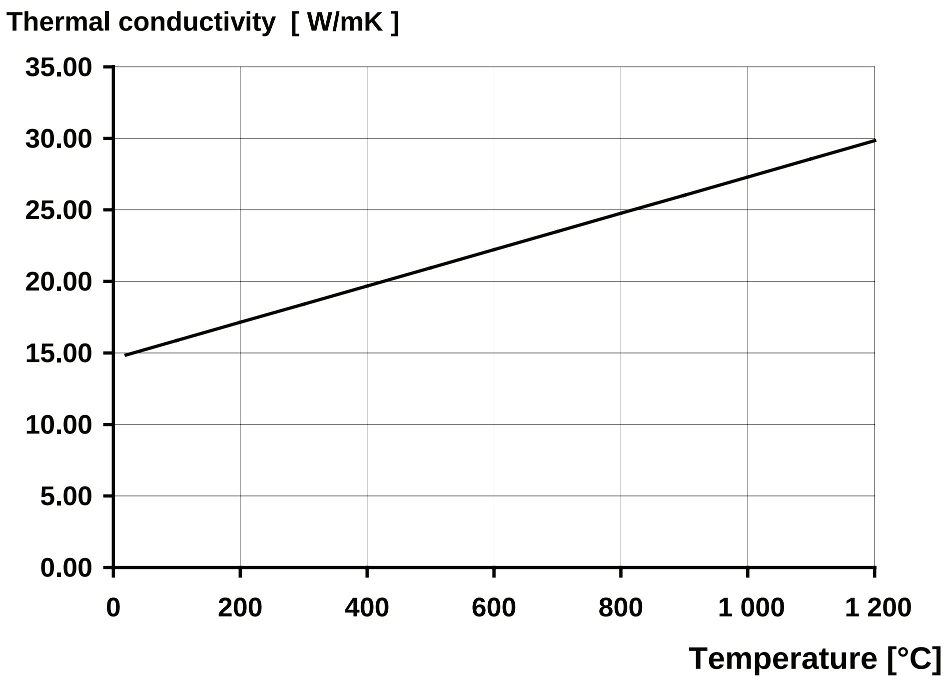
A.3.2.8 Thermal conductivity of stainless steel

The thermal conductivity of stainless steel is given by the relationship:

$$\lambda_a = 14.6 + 1.27 \times 10^{-2}\theta \text{ W/mK} \quad \text{(A60)}$$

The variation of the thermal conductivity with temperature is illustrated in Figure A.15.

Figure A.15 **Thermal conductivity of stainless steel as a function of temperature**



A.3.2.9 Thermal conductivity of cast iron and wrought iron

There is little data available on the thermal conductivity of cast and wrought iron and therefore a value of 45 W/mK can be assumed for temperatures up to 1200 °C.

A.3.2.10 Density of carbon steels

The density of carbon steels, ρ_a, (kg/m^3) is influenced by phase changes in the microstructure and remains almost constant during the temperature range of 720 °C to 880 °C, where ferrite (α) is transforms to austenite (γ). The density of carbon steels, ρ_a, (kg/m^3) varies with temperature according to the following relationships:

$$\rho_a = 7856 - 2.867 \times 10^{-1}\theta - 5.0 \times 10^{-10}\theta \text{ for } 0\ °C < \theta \leq 720\ °C \quad (A61)$$

$$\rho_a = 7610 \text{ for } 720\ °C < \theta \leq 880\ °C \quad (A62)$$

$$\rho_a = 7610 - 5.08 \times 10^{-1}\theta \text{ for } 880\ °C < \theta \leq 1000\ °C \quad (A63)$$

For general calculations density can be assumed constant at 7850 kg/m^3.

A.3.2.11 Density of stainless steel

The density of stainless steels varies as a function of temperature, as given in Table A.4.

Table A.4 **Density of stainless steel at elevated temperatures**

Temperature °C	20	100	200	400	600	800	1000
Density kg/m^3	7900	7850	7800	7700	7600	7550	7450

A.3.2.12 Density of cast iron and wrought iron

For general use, the density of cast and wrought iron can be assumed to have a value of 7 850 kg/m^3.

A.3.2.13 Emissivity of carbon steels

When heated, the surface of carbon steels oxidizes which affects the heat transfer characteristics. Technical literature shows that the emissivity can vary from a minimum of 0.8 up to 1.0 depending upon the surface cleanliness and the oxide. In BS EN 1991-1-2 a default value of 0.8 is given which was used in the UK calibration.

A.3.2.14 Emissivity of stainless steel

The emissivity of stainless steels varies according to the temperature and the alloy composition. A general value of 0.63 can be taken.

A.3.2.15 Emissivity of cast iron and wrought iron

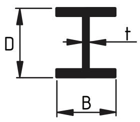
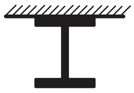
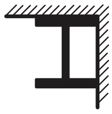
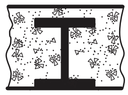
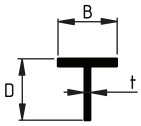
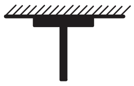
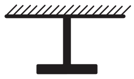
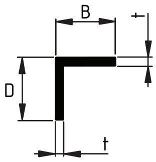
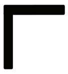
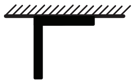
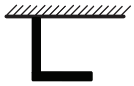
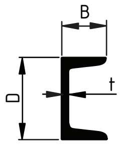
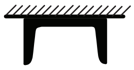
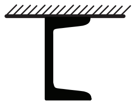
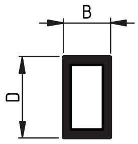
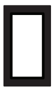
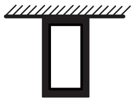
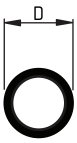
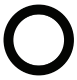
The emissivity of cast and wrought iron can be assumed to have a value of 0.8.

A.3.2.16 Calculation of heated perimeter

The method for calulating H_p for unprotected iron and steel members is shown in Figure A.16.

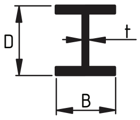
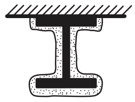
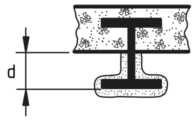
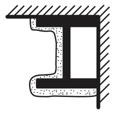
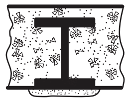
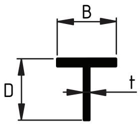
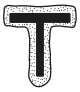
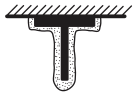
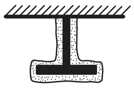
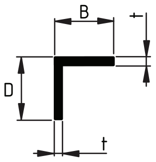
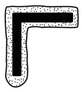
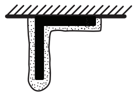
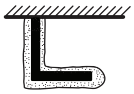
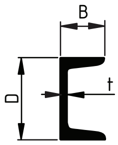
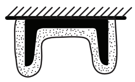
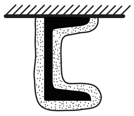
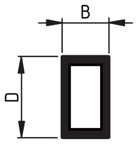
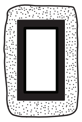
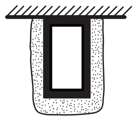
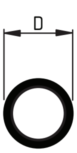
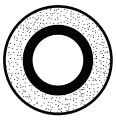
The method for calculating H_p for protected iron and steel members is shown in Figure A.17.

Figure A.16 **Determination of heated perimeter (H_p) for various configurations of unprotected steel**

Universal beams, universal columns and joists (plate and castellated) 	H_p	4 sides  $2B + 2D + 2(B - t)$ $= 4B + 2D - 2t$	3 sides  $B + 2D + 2(B - t)$ $= 3B + 2D - 2t$	3 sides  $B + 2d + (B - t)$ $= 2B + 2d - t$	2 sides  $B + D + 2(B - t) / 2$ $= 2B + D - t$	1 side  B
Structural and rolled tees 	H_p	4 sides  $2B + 2D$	3 sides  Flange to soffit $B + 2D$	3 sides  Toe of web to soffit $B + 2D + (B - t)$ $= 2B + 2D - t$		
Angles 	H_p	4 sides  $2B + 2D$	3 sides  $B + 2D$	3 sides  $B + 2D + (B - t)$ $= 2B + 2D - t$		
Channels 	H_p	4 sides  $2B + 2D + 2(B - t)$ $= 4B + 2D - 2t$	3 sides  $2B + D + 2(B - t)$ $= 4B + D - 2t$	3 sides  $B + 2D + 2(B - t)$ $= 3B + 2D - 2t$		
Hollow sections, square or rectangular 	H_p	4 sides  $2B + 2D$	3 sides  $B + 2D$			
Hollow sections, circular 	H_p	 πD				
Example using 203 × 203 × 52 kg/m universal column B = 203.9 mm D = 206.2 mm t = 8.0 mm A = 66.4 cm^2		a) Profile protection – 4 sided exposure $H_p = 4B + 2D - 2t$ $\therefore H_p = 4 \times 203.9 + 2 \times 206.2 - 2 \times 8.0 =$ 1 212 mm = 1.212 m H_p/A = 1.212/0.006 64 = 182.5 m^{-1}		b) Profile protection – 3 sided exposure $H_p = 3B + 2D - 2t$ = 611.7 + 412.4 – 16 = 1 008 mm = 1.008 m H_p/A = 1.008/0.006 64 = 151.8 m^{-1}		

NOTE *The values are approximate in that radii at corners and roots of all sections are ignored.*

Figure A.17 **Determination of H_p for various configurations of protected steel members**

Universal beams, universal columns and joists (plate and castellated) 	H_p	4 sides  $2B + 2D + 2(B - t)$ $= 4B + 2D - 2t$	3 sides  $B + 2D + 2(B - t)$ $= 3B + 2D - 2t$	3 sides  $B + 2d + (B - t)$ $= 2B + 2d - t$	2 sides  $B + D + 2(B - t) / 2$ $= 2B + D - t$	1 side  B
Structural and rolled tees 	H_p	4 sides  $2B + 2D$	3 sides  Flange to soffit $B + 2D$	3 sides  Toe of web to soffit $B + 2D + (B - t)$ $= 2B + 2D - t$		
Angles 	H_p	4 sides  $2B + 2D$	3 sides  $B + 2D$	3 sides  $B + 2D + (B - t)$ $= 2B + 2D - t$		
Channels 	H_p	4 sides  $2B + 2D + 2(B - t)$ $= 4B + 2D - 2t$	3 sides  $2B + D + 2(B - t)$ $= 4B + D - 2t$	3 sides  $B + 2D + 2(B - t)$ $= 3B + 2D - 2t$		
Hollow sections, square or rectangular 	H_p	4 sides  $2B + 2D$	3 sides  $B + 2D$			
Hollow sections, circular 	H_p	4 sides  πD				
Example using 203 × 203 × 52 kg/m universal column $B = 203.9$ mm $D = 206.2$ mm $t = 8.0$ mm $A = 66.4$ cm^2		a) Profile protection – 4 sided exposure $H_p = 4B + 2D - 2t$ $\therefore H_p = 4 \times 203.9 + 2 \times 206.2 - 2 \times 8.0 =$ 1 212 mm = 1.212 m $H_p/A = 1.212/0.006\,64 = 182.5$ m^{-1}		b) Profile protection – 3 sided exposure $H_p = 3B + 2D - 2t$ $= 611.7 + 412.4 - 16$ $= 1\,008$ mm $= 1.008$ m $H_p/A = 1.008/0.006\,64 = 151.8$ m^{-1}		
NOTE The values are approximate in that radii at corners and roots of all sections are ignored.						

A.3.3 Mechanical properties – strength characteristics, Young's (elastic) modulus and stress-strain behaviour

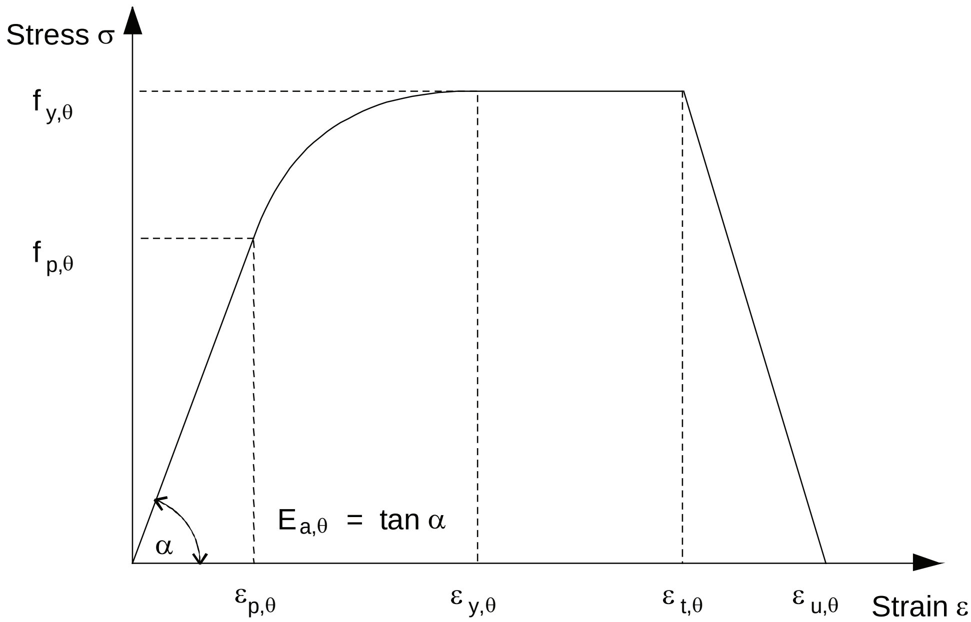
A.3.3.1 Carbon steel – hot finished structural steels

Much of the information presented in BS EN 1993-1-2 and BS EN 1994-1-2 was derived from tests carried out by Corus (formerly British Steel) and based upon elevated temperature transient tests. This type of test implicitly includes the effect of creep and, therefore, the faster the heating rate the stronger the steel appears to be. Structural steel grades 235, 275 and 355 all behave proportionally in a similar manner and, therefore, stress-strain curves and strength reduction values for these three grades are identical.

Steels which are normalized after rolling (primarily structural plate), or controlled rolled to achieve grades 420 and 460 strength levels have, on a proportional basis, a slightly lower strength at the higher temperatures (above 600 °C) and should therefore be used with caution.

For heating rates between 2 K/min and 50 K/min which cover the majority of heating (transient) conditions during a fire, the strength and deformation properties of steel at elevated temperatures are described by a model shown in Figure A.18.

Figure A.18 **Stress-strain relationships for hot finished, structural steel at elevated temperatures (see BS EN 1993-1-2:2005, Figure 3.1)**



Key

$f_{y,\theta}$ is the effective yield strength

$f_{p,\theta}$ is the proportional limit

$E_{a,\theta}$ is the slope of the linear elastic range

$\varepsilon_{p,\theta}$ is the strain at the proportional limit

$\varepsilon_{y,\theta}$ is the yield strain

$\varepsilon_{t,\theta}$ is the limiting strain for yield strength $f_{y,\theta}$

$\varepsilon_{u,\theta}$ is the ultimate strain

The first part of the curve is a linear line up to the proportional limit, $f_{p\theta}$, in which the elastic modulus, $E_{a,\theta}$, is equal to the slope (this corresponds to strain range I in BS EN 1994-1-2). The second part of the curve is represented by an elliptical expression, which depicts the transition from elastic to plastic behaviour (this corresponds to strain range II in BS EN 1994-1-2). The third part of the curve is a flat region in which there is no further work hardening (this corresponds to strain range III in BS EN 1994-1-2). The last part of the curve is characterized by a linear expression decreasing to zero (this corresponds to strain range IV in BS EN 1994-1-2). The strength of steel is taken as zero at 1 200 °C, which is slightly conservative.

Each part of the curve can be described by a series of mathematical relationships as given in Table A.5.

Table A.5 **Mathematical formulations of stress-strain relationships for hot finished structural steel at elevated temperatures (see BS EN 1993-1-2:2005, Figure 3.1)**

Strain range	**Stress σ**		**Tangent modulus**	
$\varepsilon \le \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$		$E_{a,\theta}$	
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + (b/a)\left[a^2 - \left(\varepsilon_{y,\theta} - \varepsilon\right)^2\right]^{0.5}$		$\dfrac{b\left(\varepsilon_{y,\theta} - \varepsilon\right)}{a\left[a^2 - \left(\varepsilon_{y,\theta} - \varepsilon\right)^2\right]^{0.5}}$	
$\varepsilon_{y,\theta} \le \varepsilon \le \varepsilon_{t,\theta}$	$f_{y,\theta}$		0	
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta}\left[1 - \left(\varepsilon - \varepsilon_{t,\theta}\right)/\left(\varepsilon_{u,\theta} - \varepsilon_{t,\theta}\right)\right]$		—	
$\varepsilon = \varepsilon_{u,\theta}$	0.00		—	
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta}/E_{a,\theta}$	$\varepsilon_{y,\theta} = 0.02$	$\varepsilon_{t,\theta} = 0.15$	$\varepsilon_{u,\theta} = 0.20$
Functions	$a^2 = \left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}\right)\left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c/E_{a,\theta}\right)$ $b^2 = c\left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}\right)E_{a,\theta} + c^2$ $c = \dfrac{\left(f_{y,\theta} - f_{p,\theta}\right)^2}{\left(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}\right)E_{a,\theta} - 2\left(f_{y,\theta} - f_{p,\theta}\right)}$			

The strength of steel at elevated temperatures can be described by the following parameters/reduction factors as given in BS EN 1993-1-2:2005, **3.2.1**(3):

- effective yield strength, relative to yield strength at 20 °C: $k_{y,\theta} = f_{y,\theta}/f_y$
- proportional limit, relative to yield strength at 20 °C: $k_{p,\theta} = f_{p,\theta}/f_y$
- slope of linear elastic range, relative to slope at 20 °C: $k_{E,\theta} = E_{a,\theta}/E_a$

BS EN 1994-1-2:2005, Table 3.2 gives reduction factors of stress-strain relationships of structural steel at elevated temperatures. This table is reproduced here as Table A.6.

NOTE Some of the symbols used in BS EN 1994-1-2 for stress-strain relationships are different from those given in BS EN 1993-1-2 which are reproduced in the list above.

Stress-strain relationships, with strain hardening included, are shown in Figure A.19, where:

- for strains up to 2%, Figure A.19 is in conformity with BS EN 1994-1-2:2005, Figure A.1 (range I and II);
- for strains between 2% and 4%, at temperatures below 400 °C, a linear increasing branch is assumed (range IIIa);

- for strains between 4% and 15% (range IIIb), a horizontal plateau is considered with $\varepsilon_{au,\theta} = 15\%$;
- for strains between 15% and 20% a decreasing branch (range IV) is considered with $\varepsilon_{ae,\theta} = 20\%$.

Table A.6 **Reduction factors k_θ for stress- strain relationships of structural steel at elevated temperatures (see BS EN 1994-1-2:2005, Table 3.2)**

Steel temperature θ_a (°C)	$k_{E,\theta} = \dfrac{E_{a,\theta}}{E_a}$	$k_{p,\theta} = \dfrac{f_{ap,\theta}}{f_{ay}}$	$k_{y,\theta} = \dfrac{f_{ay,\theta}}{f_{ay}}$	$k_{u,\theta} = \dfrac{f_{au,\theta}}{f_{ay}}$
20	1.00	1.00	1.00	1.25
100	1.00	1.00	1.00	1.25
200	0.90	0.807	1.00	1.25
300	0.80	0.613	1.00	1.25
400	0.70	0.42	1.00	
500	0.60	0.36	0.78	
600	0.31	0.18	0.47	
700	0.13	0.075	0.23	
800	0.09	0.05	0.11	
900	0.0675	0.0375	0.06	
1 000	0.045	0.025	0.04	
1 100	0.0225	0.0125	0.02	
1 200	0.00	0.00	0.00	

Figure A.19 **Graphical presentation of the stress-strain relationships of hot rolled structural steel at elevated temperatures, with strain-hardening included (see BS EN 1994-1-2:2005, Figure A.2)**

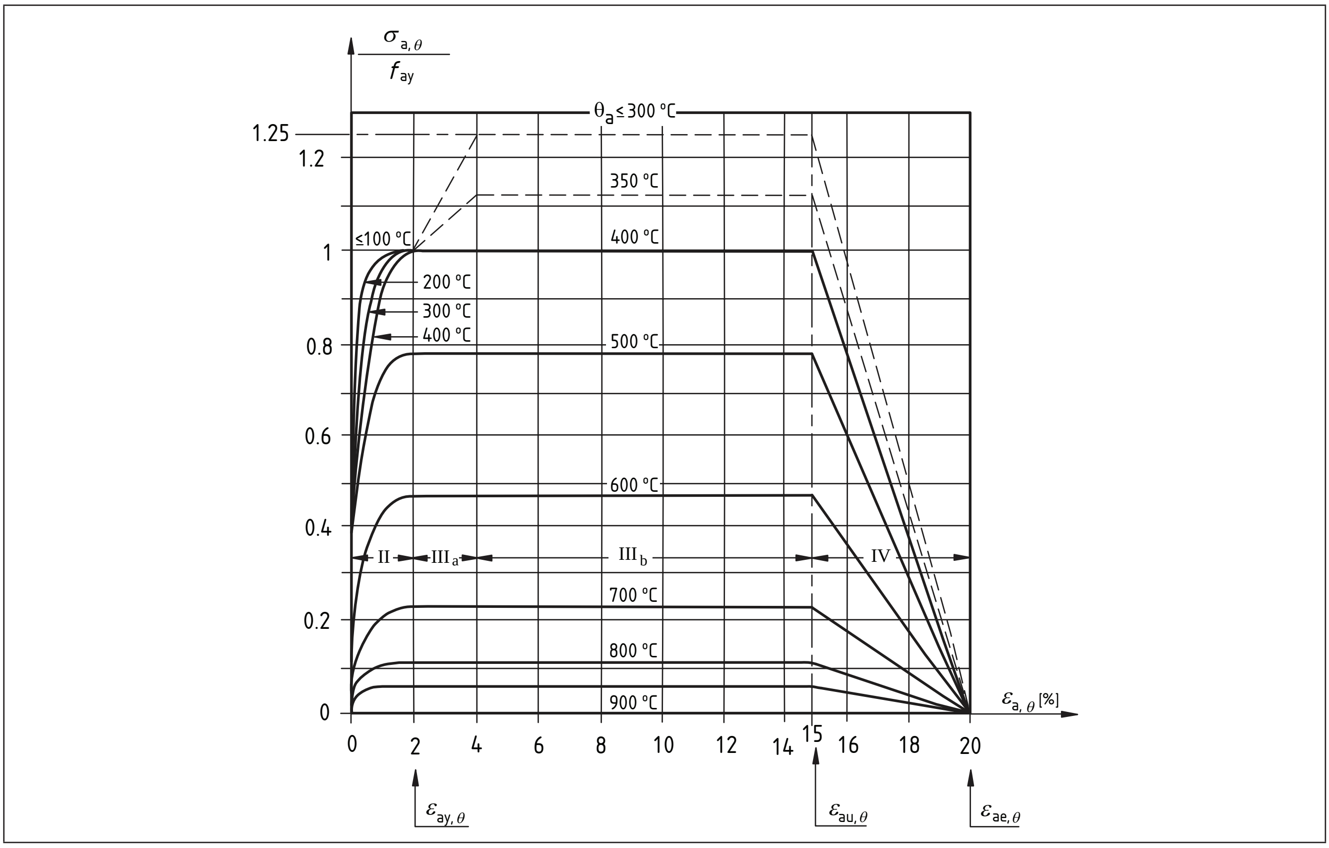
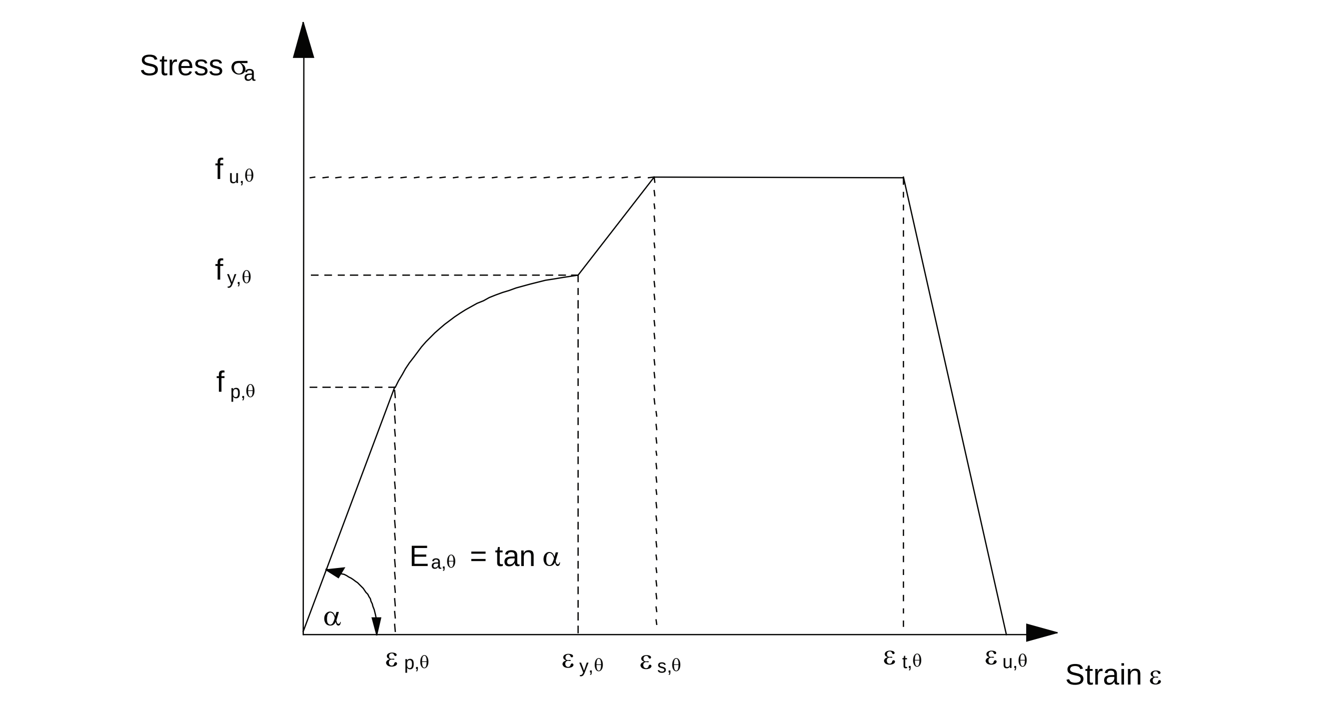


Table A.6 and Figure A.19 show that, for temperatures up to 400 °C and strains greater than 2%, an additional strain-hardening component can be introduced. This is shown schematically in Figure A.20.

Figure A.20 **Alternative stress-strain relationship for steel allowing for strain-hardening (see BS EN 1993-1-2:2005, Figure A.1)**



The strain-hardening beyond 2% strain is described as:

$$\sigma_a = 50\,(f_{u,\theta} - f_{y,\theta})\,\varepsilon + 2\,f_{y,\theta} - f_{u,\theta} \text{ for } 0.02 < \varepsilon < 0.04 \quad \text{(A64)}$$

$$\sigma_a = f_{u,\theta} \text{ for } 0.04 \le \varepsilon \le 0.15 \quad \text{(A65)}$$

$$\sigma_a = f_{u,\theta}\,[1 - 20\,(\varepsilon - 0.15)] \text{ for } 0.15 < \varepsilon < 0.20 \quad \text{(A66)}$$

$$\sigma_a = 0.00 \text{ for } \varepsilon \ge 0.20 \quad \text{(A67)}$$

The ultimate strength at elevated temperature, allowing for strain-hardening, can be applied as follows:

$$f_{u,\theta} = 1.25\,f_{y,\theta} \text{ for } \theta < 300\ ^\circ\text{C} \quad \text{(A68/69)}$$

$$f_{u,\theta} = f_{y,\theta}\,(2 - 0.0025\theta) \text{ for } 300\ ^\circ\text{C} \le \theta < 400\ ^\circ\text{C} \quad \text{(A70)}$$

$$f_{u,\theta} = f_{y,\theta} \text{ for } \theta \ge 400\ ^\circ\text{C} \quad \text{(A71)}$$

The effect of strain-hardening should only be considered if the analysis is based upon advanced calculation models and if it is shown that local failures, e.g. local buckling, shear failure, do not occur due to increased strains.

Strength reduction values, as given in Table A.7, are provided for grades 275 and 355 steels for limiting strains of 0.5%, 1.5% and 2%. These are based upon work reported by Kirby and Preston [51]. The values cover members in tension or compression and members in bending for non-composite and composite action.

Table A.7 **Strength reduction factor for structural steel grades 275 and 355 to BS EN 10025-1 and BS EN 10025-2**

Strain (%)	Temperature °C													
	100	**150**	**200**	**250**	**300**	**350**	**400**	**450**	**500**	**550**	**600**	**650**	**700**	**750**
0.5	0.97	0.96	0.95	0.90	0.85	0.83	0.80	0.72	0.62	0.49	0.38	0.27	0.19	0.13
1.5	1.0	1.0	1.0	1.0	1.0	0.97	0.96	0.90	0.76	0.61	0.46	0.33	0.22	0.15
2.0	1.0	1.0	1.0	1.0	1.0	1.0	0.97	0.93	0.78	0.63	0.47	0.34	0.23	0.16

A more detailed discussion on the choice of the limiting values of strain and how these compare with BS EN 1993-1-2 and BS EN 1994-1-2 is given by Lawson and Newman [52].

Table A.8 and Table A.9 provide the stress-strain data for grades 275 and 355 structural steels.

For structures undergoing refurbishment in which mild steels supplied in accordance with BS 15 were used in the original construction, work by Kirby [53] has shown that the same strength reduction factors can be applied.

Following a fire, hot finished structural steelwork can be expected to regain most if not all of its strength. For hot rolled grade 235 and grade 275 (mild steel 40A, 43A), at least 90% of the minimum specified properties are expected to return. For higher strength steel, grade 355 (50B), at least 80% of its minimum specified properties can be restored. However, steel is usually supplied with strength properties well above the specified minimum and, therefore, even if some loss in strength occurs the properties might still be above the minimum requirements. Further information and guidance is given in Kirby et al [54].

Table A.8 **Elevated temperature stress-strain data for grade 275 structural steel**

Strain	**Stress in N/mm² at various temperatures in degrees centigrade**																			
%	20	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	*	*
0.01	18.4	18.4	18.4	17.3	16.5	15.7	15.7	14.6	13.2	11.8	9.3	6.9	5.5	4.1	1.9	1.9	1.6	*	*	*
0.02	36.9	36.9	35.8	35.2	33.0	31.9	31.4	28.9	26.7	23.4	19.0	13.5	11.6	8.3	4.1	3.9	3.3	*	*	*
0.03	55.3	54.4	54.4	53.6	49.8	47.6	46.7	43.5	39.9	35.2	28.6	20.4	17.1	12.4	6.1	5.8	5.5	*	*	*
0.04	73.7	72.9	72.0	70.9	66.3	63.5	62.4	57.8	53.3	46.7	38.2	27.0	22.5	16.5	8.3	8.0	7.4	*	*	*
0.05	92.1	89.9	90.5	88.6	82.8	79.2	78.1	72.3	66.8	58.6	47.6	33.8	28.3	20.6	10.4	9.9	9.3	*	*	*
0.06	110.6	109.7	107.8	106.2	99.0	95.2	93.8	86.9	80.0	70.1	56.9	40.7	34.1	24.8	12.4	11.8	11.3	*	*	*
0.07	128.7	127.3	126.2	123.8	115.5	111.1	109.4	101.2	93.2	82.0	66.8	47.3	39.6	28.9	14.6	13.8	12.9	*	*	*
0.08	147.1	145.5	143.8	142.2	132.3	126.8	124.8	115.8	106.4	93.5	76.2	54.2	45.1	32.7	16.5	15.4	14.3	*	*	*
0.09	165.8	163.9	162.3	159.5	148.8	142.7	133.7	125.4	117.4	105.3	85.5	60.8	48.9	36.0	18.7	16.5	14.6	*	*	*
0.10	184.3	182.3	179.8	177.1	165.3	158.7	140.3	132.0	123.8	112.8	95.2	67.7	52.3	38.5	20.6	17.9	14.9	*	*	*
0.12	221.1	218.3	215.6	212.3	198.3	183.4	152.4	144.6	136.9	122.7	104.5	81.4	58.0	42.9	24.8	19.8	15.4	*	*	*
0.14	256.6	249.7	238.4	230.7	212.6	194.7	161.4	154.3	147.1	131.4	113.6	88.6	63.5	45.9	27.2	21.2	15.4	*	*	*
0.16	264.0	256.9	249.1	241.2	223.3	205.1	169.4	162.3	154.8	138.6	119.9	95.7	67.7	48.7	29.1	22.5	15.7	*	*	*
0.18	271.4	264.5	259.6	248.9	231.6	211.8	176.0	168.9	161.7	144.9	126.2	101.5	71.8	51.1	31.1	23.4	16.0	*	*	*
0.20	275.0	266.8	262.6	251.4	237.1	216.4	182.3	174.9	167.8	149.9	131.4	106.2	75.6	53.1	32.7	24.5	16.2	*	*	*
0.25	275.0	267.3	264.5	255.5	246.7	224.4	195.3	188.1	180.7	161.4	142.4	113.8	82.8	58.3	37.1	26.7	16.8	*	*	*
0.30	275.0	267.6	265.1	258.2	252.2	230.4	206.8	199.4	191.7	171.1	151.3	119.3	89.4	63.0	41.0	28.9	17.3	*	*	*
0.35	275.0	267.6	265.4	260.1	256.0	234.3	215.3	207.9	200.5	179.6	157.8	124.6	94.3	67.4	44.3	30.8	17.9	*	*	*
0.40	275.0	267.6	265.6	261.3	258.0	237.1	223.3	215.3	207.4	186.5	163.4	128.7	98.4	70.4	47.0	32.5	18.4	*	*	*
0.50	275.0	267.9	265.9	263.2	260.1	243.1	234.8	227.1	219.4	198.3	171.1	135.3	104.0	74.0	51.1	34.9	19.5	*	*	*
0.60	275.0	267.9	265.9	263.5	260.7	248.1	243.4	235.7	228.0	207.6	178.5	142.2	108.4	76.4	53.3	36.0	20.6	13.8	*	*
0.70	275.0	267.9	266.2	263.7	261.3	251.9	248.9	242.8	236.8	215.6	185.9	148.2	111.9	78.6	54.7	36.9	21.7	14.9	*	*
0.80	275.0	267.9	266.2	264.0	261.8	256.0	253.8	249.7	245.3	222.2	191.1	153.2	115.8	80.8	55.8	37.7	22.8	15.7	*	*
0.90	275.0	267.9	266.2	264.3	262.1	259.6	257.4	254.1	251.1	227.4	195.5	157.3	118.8	82.5	56.9	38.2	23.9	16.5	*	*
1.00	275.0	267.9	266.2	264.5	262.6	261.8	260.1	258.5	255.5	231.6	198.6	160.6	120.4	83.9	57.8	39.1	25.0	17.3	*	*
1.20	*	*	*	*	*	*	264.8	262.9	259.1	238.7	202.9	164.2	123.2	86.3	59.4	40.2	27.2	18.7	*	*
1.40	*	*	*	*	*	*	*	265.1	261.8	244.2	206.3	167.2	125.4	88.6	60.8	41.3	29.1	19.8	16.0	*
1.60	*	*	*	*	*	*	*	*	264.0	249.4	209.0	169.4	127.3	90.5	61.9	42.1	30.3	20.6	16.5	*
1.80	*	*	*	*	*	*	*	*	265.6	253.8	211.5	171.1	129.0	91.8	63.0	42.9	31.1	21.5	16.8	*
2.00	*	*	*	*	*	*	*	*	*	256.9	213.4	172.4	130.3	92.7	63.8	43.5	31.3	21.7	17.1	14.3

Table A.9 **Elevated temperature stress-strain data for grade 355 structural steel**

Strain	Stress in N/mm^2 for various temperatures in degrees centigrade																			
%	20	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	*	*
0.01	18.8	18.8	18.8	17.8	16.3	15.6	15.6	14.6	13.1	11.7	9.6	6.7	5.7	4.3	2.1	2.1	1.8	*	*	*
0.02	37.6	37.6	36.6	35.9	33.0	31.6	31.2	28.8	26.6	23.4	18.8	13.5	11.4	8.2	4.3	3.9	3.9	*	*	*
0.03	56.1	55.4	55.4	54.3	49.3	47.2	46.5	43.3	39.8	35.1	28.4	20.2	17.0	12.4	6.0	6.0	5.7	*	*	*
0.04	74.9	74.2	73.1	72.4	65.7	63.2	62.1	57.5	53.3	46.5	38.0	27.0	22.4	16.3	8.2	7.8	7.5	*	*	*
0.05	93.7	92.7	91.9	90.2	82.4	78.8	77.7	72.1	66.4	58.2	47.6	33.7	28.0	20.6	10.3	9.9	9.2	*	*	*
0.06	112.5	111.5	109.7	107.9	98.7	94.8	93.4	86.6	79.5	69.9	56.8	40.5	33.7	24.5	12.4	11.7	11.4	*	*	*
0.07	131.0	129.2	128.5	125.7	115.0	110.4	109.0	100.8	93.0	81.7	66.4	47.2	39.4	28.8	14.2	13.8	13.1	*	*	*
0.08	149.8	148.0	146.3	144.5	131.7	126.4	124.3	115.4	106.1	93.4	76.0	54.0	45.1	32.7	16.3	15.6	14.9	*	*	*
0.09	168.6	166.9	165.1	162.2	148.0	142.0	139.9	129.6	119.3	105.1	85.2	60.7	50.8	36.9	18.5	17.8	16.7	*	*	*
0.10	187.4	185.7	182.8	180.3	164.4	158.0	155.1	143.8	132.4	116.4	94.8	67.4	56.4	40.8	20.6	19.5	18.5	*	*	*
0.12	224.7	222.2	219.4	215.8	197.4	189.6	183.2	171.1	159.0	139.5	113.6	80.9	66.0	49.0	24.5	22.0	19.9	*	*	*
0.14	262.3	259.5	256.0	252.4	230.4	221.2	198.8	188.5	177.9	158.0	132.8	94.4	74.2	53.6	28.8	24.5	19.9	*	*	*
0.16	299.6	296.1	292.5	288.3	263.4	251.0	211.6	201.6	191.3	170.0	145.2	107.9	81.7	58.2	33.0	26.6	20.2	*	*	*
0.18	337.3	333.7	328.4	313.8	287.9	264.8	221.9	212.6	203.1	180.3	156.2	120.3	88.0	62.5	36.6	28.4	20.6	*	*	*
0.20	355.0	345.1	333.7	313.8	300.7	273.0	231.8	222.2	213.0	188.5	165.8	130.6	94.1	66.0	39.8	30.2	20.9	*	*	*
0.25	355.0	348.6	338.7	326.6	316.3	287.2	250.6	241.8	232.5	206.3	182.8	145.6	105.1	74.5	46.2	33.7	21.7	*	*	*
0.30	355.0	349.7	341.2	332.6	325.2	296.8	267.0	257.0	247.4	220.5	195.3	154.1	115.0	81.3	51.5	36.6	22.4	*	*	*
0.35	355.0	350.0	342.6	336.5	330.5	302.5	278.0	268.4	258.8	231.8	203.8	160.8	121.8	87.0	57.2	39.8	23.1	*	*	*
0.40	355.0	350.0	344.0	338.7	333.0	306.0	288.3	278.0	267.7	240.7	210.9	166.1	127.1	90.0	60.7	41.9	23.8	*	*	*
0.50	355.0	350.4	345.4	340.8	335.8	313.8	303.2	293.2	283.3	256.0	220.8	174.7	134.2	95.5	66.0	45.1	25.2	*	*	*
0.60	355.0	350.4	345.8	341.2	336.5	320.2	314.2	304.2	294.3	268.0	230.4	183.5	139.9	98.7	68.9	46.5	26.6	17.8	*	*
0.70	355.0	350.7	346.1	341.9	337.3	325.2	321.3	313.5	305.7	278.3	240.0	191.3	144.5	101.5	70.6	47.6	28.0	19.2	*	*
0.80	355.0	350.7	346.5	342.2	338.0	330.5	327.7	322.3	316.7	286.8	246.7	197.7	149.5	104.4	72.1	48.6	29.5	20.2	*	*
0.90	355.0	350.7	346.8	342.6	338.3	335.1	332.3	328.0	324.1	293.6	252.4	203.1	153.4	106.5	73.5	49.3	30.9	21.3	*	*
1.00	355.0	351.1	347.2	343.3	339.0	338.0	335.8	333.7	329.8	298.9	256.3	207.3	155.5	108.3	74.5	50.4	32.3	22.4	*	*
1.20	*	*	*	*	*	*	341.9	339.4	334.4	308.1	262.0	212.3	159.0	111.5	76.7	51.8	35.1	24.1	*	*
1.40	*	*	*	*	*	*	*	342.2	338.0	315.2	266.3	215.8	161.9	114.3	78.5	53.3	37.6	25.6	20.6	*
1.60	*	*	*	*	*	*	*	*	340.8	322.0	269.8	218.7	164.4	116.8	79.9	54.3	39.0	26.6	21.3	*
1.80	*	*	*	*	*	*	*	*	342.9	327.7	273.0	220.8	166.5	118.6	81.3	55.4	40.1	27.7	21.7	*
2.00	*	*	*	*	*	*	*	*	*	331.6	275.5	222.6	168.3	119.6	82.4	56.1	40.8	28.0	22.0	18.5

A.3.3.2 Carbon steel – hot and cold worked reinforcing steels

The strength and deformation properties of reinforcing steels at elevated temperatures can be obtained from the same mathematical expressions as given for structural steel.

A.3.3.3 Carbon steel – cold worked reinforcing steels

The elastic modulus is considered to be the same as that for hot rolled steels at ambient temperature.

At elevated temperatures the three main parameters describing the stress-strain relationship for cold worked reinforcing steels are given in Table A.10 which has been adapted from BS EN 1992-1-2:2004, Table 3.2a and uses the notation given in Table A.6.

Table A.10 **Values for the three main parameters $\left(E_{a\theta};f_{ap,\theta};f_{a,\theta}\right)$ of the stress-strain relationships for cold worked reinforcing steel**

Steel temperature θ (°C)	$\frac{E_{a,\theta}}{E_a}$	$\frac{f_{ap,\theta}}{f_{ay}}$	$\frac{f_{a,\theta}}{f_{ay}}$
20	1.00	1.00	1.00
100	1.00	0.96	1.00
200	0.87	0.92	1.00
300	0.72	0.81	1.00
400	0.56	0.63	0.94
500	0.40	0.44	0.67
600	0.24	0.26	0.40
700	0.08	0.08	0.12
800	0.06	0.06	0.11
900	0.05	0.05	0.08
1 000	0.03	0.03	0.05
1 100	0.02	0.02	0.03
1 200	0.00	0.00	0.00

For the decreasing branch of the stress-strain relationship the values for hot finished structural steel can be adopted.

A.3.3.4 Carbon steel – cold worked (wires and strands) and quenched and tempered (bars) pre-stressing steel

The strength and deformation properties of pre-stressing steel at elevated temperatures can be obtained by the same mathematical model as hot finished reinforcing steel.

Values for the parameters for cold worked (wires and strands) and quenched and tempered (bars) pre-stressing steel at elevated temperatures are described by the expressions listed in equation A72 (see Table A.11):

$$f_{py,\theta} / (\beta f_{pk}),\ f_{pp,\theta} / (\beta f_{pk}),\ E_{p,\theta} / E_p,\ \varepsilon_{pt,\theta}\ [-],\ \varepsilon_{pu,\theta}\ [-] \qquad \text{(A72)}$$

BS EN 1992-1-2 gives a choice for the value of β of class A or class B.

For class A:

$$\beta = \left[\left(\frac{\varepsilon_{ud} - f_{p0.1k}/E_p}{\varepsilon_{uk} - f_{p0.1k}/E_p} \right) \times \left(\frac{f_{pk} - f_{p0.1k}}{f_{pk}} \right) + \frac{f_{p0.1k}}{f_{pk}} \right] \tag{A.73}$$

where the definitions and values of ε_{ud}, ε_{uk}, $f_{p0.1k}$, f_{pk} and E_p at normal temperatures are given in BS EN 1992-1-1:2004, **3.3**.

For class B:

$\beta = 0.9$

NA to BS EN 1992-1-2 gives the UK decision for the value of β as class A.

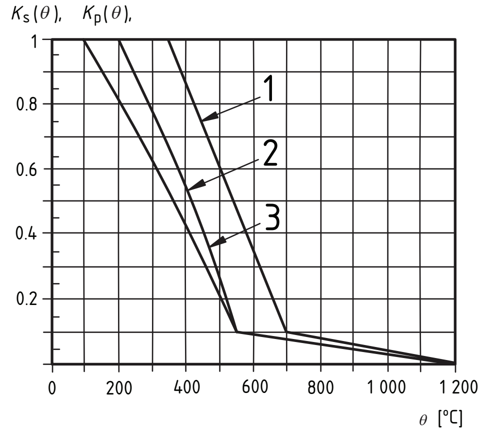
Table A.11 **Values for the parameters of the stress-strain relationship of cold worked (cw) (wires and strands) and quenched and tempered (q & t) (bars) pre-stressing steel at elevated temperatures (see BS EN 1992-1-2:2004, Table 3.3)**

Steel temperature θ °C	$f_{py,\theta}/(\beta f_{pk})$			$f_{pp,\theta}/(\beta f_{pk})$		$E_{p,\theta}/E_p$		$\varepsilon_{pt,\theta}$ [-]	$\varepsilon_{pu,\theta}$ [-]
	cw		q & t	cw	q & t	cw	q & t	cw, q & t	cw, q & t
	Class A	Class B							
1	2a	2b	3	4	5	6	7	8	9
20	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.050	0.100
100	1.00	0.99	0.98	0.68	0.77	0.98	0.76	0.050	0.100
200	0.87	0.87	0.92	0.51	0.62	0.95	0.61	0.050	0.100
300	0.70	0.72	0.86	0.32	0.58	0.88	0.52	0.055	0.105
400	0.50	0.46	0.69	0.13	0.52	0.81	0.41	0.060	0.110
500	0.30	0.22	0.26	0.07	0.14	0.54	0.20	0.065	0.115
600	0.14	0.10	0.21	0.05	0.11	0.41	0.15	0.070	0.120
700	0.06	0.08	0.15	0.03	0.09	0.10	0.10	0.075	0.125
800	0.04	0.05	0.09	0.02	0.06	0.07	0.06	0.080	0.130
900	0.02	0.03	0.04	0.01	0.03	0.03	0.03	0.085	0.135
1 000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.090	0.140
1 100	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.095	0.145
1 200	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.100	0.150

NOTE For intermediate values of temperature, linear interpolation may be used.

In BS EN 1992-1-2 the reduction of characteristic strength as a function of temperature, θ, is given as shown in Figure A.21.

Figure A.21 **Reference curves for critical temperature of reinforcing and pre-stressing steels**



Key

1 Reinforcing steel [K_s (θ)]

2 Pre-stressing steel (bars: prEN 10138-4) [K_p (θ)]

3 Pre-stressing steel (wires and strands: prEN 10138-2 and prEN 10138-3) [K_p (θ)]

A.3.3.5 Carbon steel – light gauge steel

The elastic modulus is considered to be the same as that for hot rolled steels at elevated temperatures.

BS EN 1993-1-2 and BS EN 1994-1-2 do not specifically describe the strength characteristics of cold formed steels (see the British Steel report by Sidey and Teague [55]).

In general the strength of cold formed steel is proportionally lower than hot rolled structural steel, particularly in the temperature range 400 °C to 600 °C. However, the data presented is based upon the lower 95% confidence limit, whereas the data for hot rolled structural steel is typical behaviour for steel with ambient temperature properties at the specified minimum.

The strength reduction values corresponding to 0.5%, 1.5% and 2% strain are given in Table A.12. The ultimate strength, $k_{max,\theta}$, at temperature θ can be taken as equivalent to the strength reduction value corresponding to 2% strain.

Table A.12 **Strength reduction factor for cold formed galvanized steel to BS EN 10147**

Strain	**Temperature °C**													
(%)	**100**	**150**	**200**	**250**	**300**	**350**	**400**	**450**	**500**	**550**	**600**	**650**	**700**	**750**
0.5	1.0	1.0	0.95	0.89	0.83	0.76	0.68	0.58	0.47	0.37	0.27	—	—	—
1.5	1.0	1.0	1.0	0.99	0.95	0.88	0.82	0.69	0.56	0.45	0.35	—	—	—
2.0	1.0	1.0	1.0	1.0	1.0	0.93	0.87	0.73	0.59	0.49	0.39	—	—	—

BS EN 1993-1-2:2005, Annex E provides reduction factors for the design strength and elastic modulus of carbon steel used for class 4 light gauge sections. For hot rolled and welded thin walled sections the reduction factor for the design strength $k_{p0.2,\theta}$, is taken relative to the yield strength, f_y, at 20 °C as:

$$k_{p0.2,\theta} = f_{p0.2,\theta} / f_y \quad \text{(A74)}$$

For cold formed light gauge sections the reduction factor for the design strength, $k_{p0.2,\theta}$, is taken relative to the basic yield strength at 20°C, $f_{yb,}$ as follows:

$$k_{p0.2,\,\theta} = f_{p0.2,\,\theta} / f_{yb} \tag{A75}$$

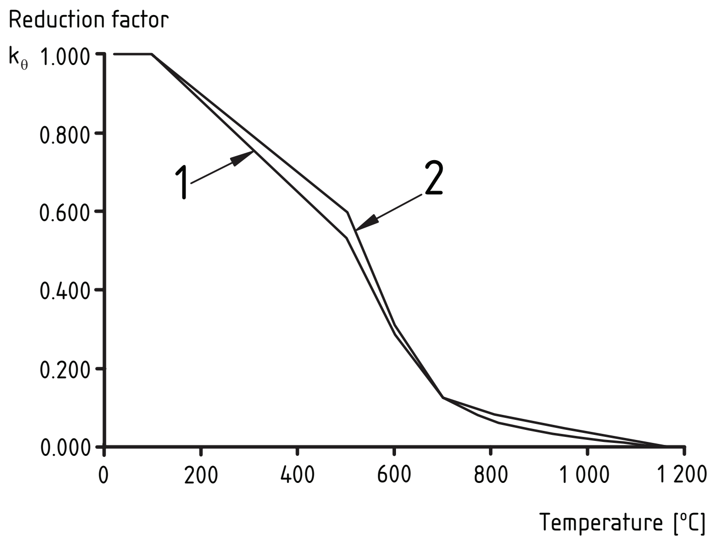
These functions are presented in Table A.13 and Figure A.22.

Table A.13 **Reduction factors for carbon steel for the design of class 4 sections at elevated temperatures**

Steel temperature θ_a	**Reduction factor (relative to f_y) for the design strength of hot rolled and welded thin walled sections** $k_{p0.2,\,\theta} = f_{p0.2,\,\theta} / f_y$	**Reduction factor (relative to f_{yb}) for the design strength of cold formed thin walled sections** $k_{p0.2,\,\theta} = f_{p0.2,\,\theta} / f_{yb}$
20 °C	1.00	
100 °C	1.00	
200 °C	0.89	
300 °C	0.78	
400 °C	0.65	
500 °C	0.53	
600 °C	0.30	
700 °C	0.13	
800 °C	0.07	
900 °C	0.05	
1 000 °C	0.03	
1 100 °C	0.02	
1 200 °C	0.00	

NOTE For intermediate values of the steel temperature, linear interpolation may be used.

Figure A.22 **Reduction factors for the stress-strain relationship of cold formed and hot rolled thin walled steel at elevated temperatures**



Key

1 Design yield strength $k_{p0.2,\theta} = f_{p0.2,\theta} / f_y$

2 Slope of linear elastic range $k_{E,\theta} = E_{a,\theta} / E_a$

A.3.3.6 Carbon steel – bolts

The capacities of grade 8.8 bolts in double shear and tension at elevated temperatures up to 800 °C have been derived by Kirby [56] and have now been incorporated into BS EN 1993-1-2.

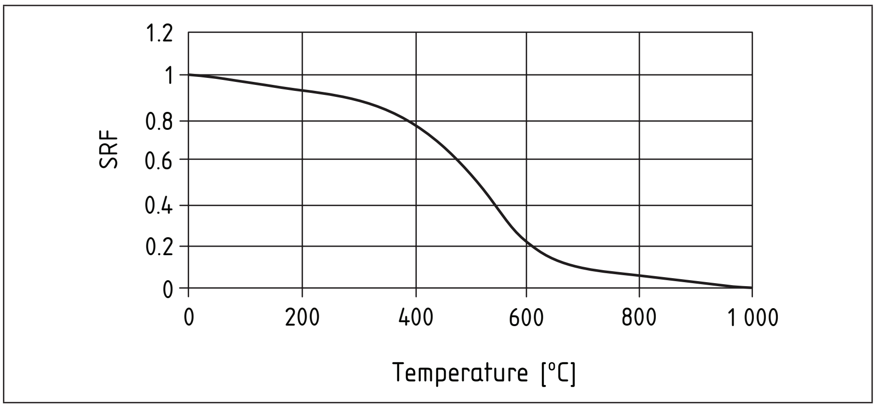
Grade 8.8 bolts gain their high strength primarily by a quench and temper process route. The tempering process is usually carried out between 500 °C and 600 °C. During a fire, if the tempering temperature is exceeded, over softening can occur and there is a marked reduction in strength. Since grade 10.9 bolts are also produced by a quench and temper process, they exhibit similar characteristics at elevated temperatures.

At elevated temperatures work by Kirby [56] has shown the strength reductions in shear and tension for grade 8.8 bolts are very similar and the factors given in Table A.14 apply in both cases. This is also illustrated in Figure A.23.

Table A.14 **Strength reduction factors for grade 8.8 bolts in shear and tension**

Temperature °C	**Strength reduction factor for bolts, $k_{b,\theta}$ (shear and tension)**
20	1.000
100	0.968
150	0.952
200	0.935
300	0.903
400	0.775
500	0.550
600	0.220
700	0.100
800	0.067
900	0.033
1 000	0.000

Figure A.23 **Strength reduction factors (SRF) for grade 8.8 bolts in shear and tension at elevated temperatures**



In BS EN 1993-1-2, the same strength reduction factor is applied for bolts in shear and tension regardless of the bolt type.

While grade 10.9 bolts are manufactured by a similar process route to grade 8.8 bolts, they lose their strength more quickly at elevated temperatures, particularly above 500 °C. Therefore the strength of grade 10.9 bolts above 500 °C should be assumed to be lower than that of grade 8.8 bolts.

In contrast, since grade 4.6 bolts are not produced by a quench and temper process route they retain their strength better at elevated temperatures. Table A.14 and Figure A.23 underestimate their performance in fire.

For friction grip bolts it is assumed the bolts slip in fire and the fire resistance of a single bolt may be designed for shear and bearing.

A.3.3.7 Carbon steel – welds

The elevated temperature behaviour of butt and fillet welds have been derived by Latham and Kirby [57].

Table A.15 provides information on the design strength of butt welds for each type of welding process.

Table A.15 **Strength reduction factors for butt welds**

Weld type	**MMA[A)] butt weld 2.5 kJ/mm Fortrex E 7018[B)]**			**GAW[A)] butt weld 3.0 kJ/mm OP121T/SD3[B)]**			**GMAW[A)] butt weld 2.5 kJ/mm Carbofil[B)]**		
Temperature °C	**Strength reduction factors for strain (%) of:**			**Strength reduction factors for strain (%) of:**			**Strength reduction factors for strain (%) of:**		
	0.5	**1.5**	**2.0**	**0.5**	**1.5**	**2.0**	**0.5**	**1.5**	**2.0**
20	1.000	1.000	1.000	1.0	1.0	1.0	1.000	1.000	1.000
350	0.801	1.000	1.000	0.730	1.0	1.0	<0.831	1.000	1.000
400	0.759	>0.837	1.000	0.700	>0.841	>0.841	0.827	1.000	1.000
450	0.720	0.816	>0.837	0.658	0.772	0.816	0.793	>0.831	1.000
500	0.619	0.722	0.759	0.572	0.671	0.700	0.739	0.815	0.831
550	0.502	0.610	0.619	0.433	0.530	0.557	0.634	0.698	0.729
600	0.358	0.427	0.450	0.282	0.355	0.368	0.435	0.499	0.532
650	0.230	0.230	0.285	0.181	0.231	0.242	0.259	0.290	0.309
700	0.153	0.153	0.116	0.105	0.126	0.130	0.163	0.176	0.188

A) MMA: manual metal arc welding.
GAW: gas arc welding.
GMAW: gas metal arc welding.

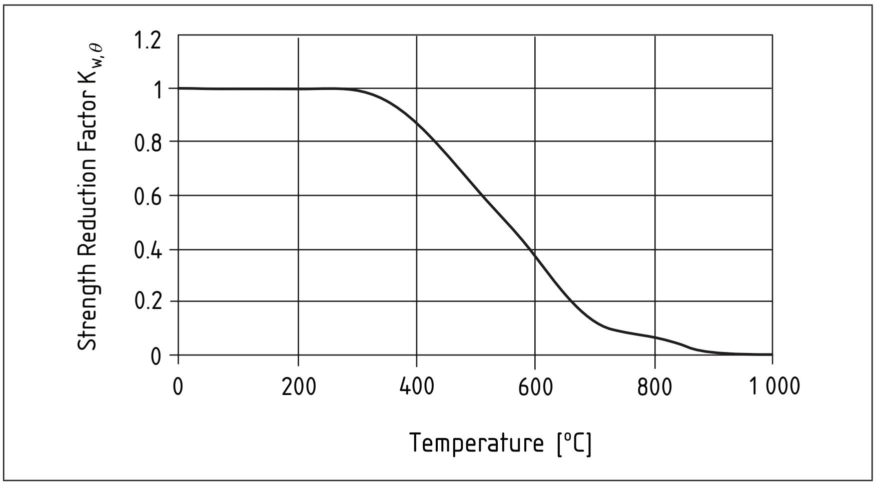
B) These are trade names and are given for the convenience of users of this Published Document. This does not constitute an endorsement by BSI of the products named.

For fillet welds the strength reduction factor for the design resistance presented in BS EN 1993-1-2 is also based upon work by Latham and Kirby [57] and is given in Table A.16 and Figure A.24.

Table A.16 **Strength reduction factors for fillet welds at elevated temperatures (see BS EN 1993-1-2:2005, Table D.1)**

Temperature °C	Reduction factor for welds, $k_{w,\theta}$
20	1.000
100	1.000
150	1.000
200	1.000
300	1.000
400	0.876
500	0.627
600	0.378
700	0.130
800	0.074
900	0.018
1 000	0.000

Figure A.24 **Strength reduction factors for fillet welds at elevated temperatures**



According to BS EN 1993-1-2 the design strength of butt welds for temperatures up to 700 °C should be taken as that of the weaker part of the connecting member, using strength reduction factors for hot finished structural steel. At temperatures above 700 °C the strength reduction factors for fillet welds should be used.

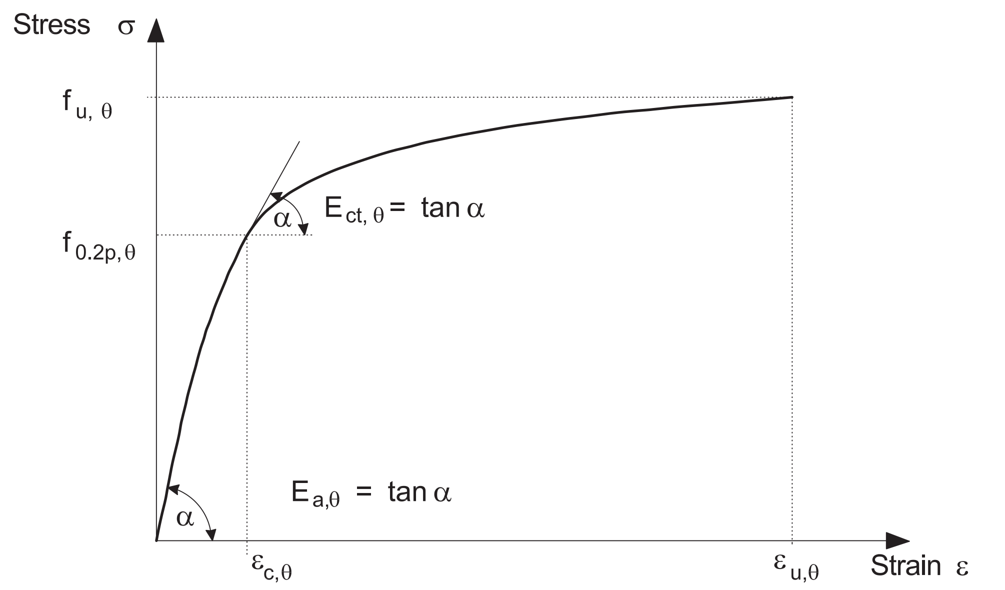
A.3.3.8 Carbon steel – Poisson's ratio

The Poisson's ratio for all carbon steels is 0.3 and is independent of temperature.

A.3.3.9 Stainless steel

The stress-strain behaviour of stainless steels can be described by the general model in Figure A.25 and the mathematical relationships given in Table A.17 and applies to heating rates between 2 K/min and 50 K/min. Much of this is based upon work by Burgan [36].

Figure A.25 **Stress-strain model for stainless steel at elevated temperatures**



Key

$f_{u,\theta}$ is the tensile strength

$f_{0.2p,\theta}$ is the proof strength at 0.2% plastic strain

$E_{a,\theta}$ is the slope of the linear elastic range

$E_{ct,\theta}$ is the slope at proof strength

$\varepsilon_{c,\theta}$ is the total strain at proof strength

$\varepsilon_{u,\theta}$ is the ultimate strain

Table A.17 **Stress-strain parameters for stainless steel**

Strain range	**Stress** σ	**Tangent modulus** E_t
$\varepsilon \le \varepsilon_{c,\theta}$	$\dfrac{E\varepsilon}{1+a\varepsilon^b}$	$\dfrac{E\left(1+a\varepsilon^b-ab\varepsilon^b\right)}{\left(1+a\varepsilon^b\right)^2}$
$\varepsilon_{c,\theta} < \varepsilon \le \varepsilon_{u,\theta}$	$f_{0.2p,\theta}-e+(d/c)\sqrt{c^2-\left(\varepsilon_{u,\theta}-\varepsilon\right)^2}$	$\dfrac{d\left(\varepsilon_{u,\theta}-\varepsilon\right)}{c\sqrt{c^2-\left(\varepsilon_{u,\theta}-\varepsilon\right)^2}}$
Parameters	$\varepsilon_{c,\theta} = f_{0.2p,\theta}/E_{a,\theta} + 0.002$	
Functions	$a=\dfrac{E_{a,\theta}\varepsilon_{c,\theta}-f_{0.2p,\theta}}{f_{0.2p,\theta}\varepsilon_{c,\theta}}$ $\quad b=\dfrac{\left(1-\varepsilon_{c,\theta}E_{ct,\theta}/f_{0.2p,\theta}\right)E_{a,\theta}\varepsilon_{c,\theta}}{\left(E_{a,\theta}\varepsilon_{c,\theta}/f_{0.2p,\theta}-1\right)f_{0.2p,\theta}}$ $c^2=\left(\varepsilon_{u,\theta}-\varepsilon_{c,\theta}\right)\left(\varepsilon_{u,\theta}-\varepsilon_{c,\theta}+\dfrac{e}{E_{ct,\theta}}\right)$ $\quad d^2=e\left(\varepsilon_{u,\theta}-\varepsilon_{c,\theta}\right)E_{ct,\theta}+e^2$ $e=\dfrac{\left(f_{u,\theta}-f_{0.2p,\theta}\right)^2}{\left(\varepsilon_{u,\theta}-\varepsilon_{c,\theta}\right)E_{ct,\theta}-2\left(f_{u,\theta}-f_{0.2p,\theta}\right)}$	

For structural design in fire, three sets of parameters are necessary to describe the behaviour at elevated temperatures relative to behaviour at 20 °C. These are:

elastic modulus:

$$k_{E,\theta} = E_{a,\theta}/E_a \tag{A76}$$

0.2% proof strength:

$$k_{0.2p,\theta} = f_{0.2p,\theta}/f_y \tag{A77}$$

ultimate tensile strength:

$$k_{u,\theta} = f_{u,\theta}/f_u \tag{A78}$$

See Table A.18.

The strength at 2% absolute strain is particularly introduced for stainless steel member design. It can be considered in the same way as the parameter $k_{p,\theta}$ for structural steel and is related to two specific strength levels, namely $f_{0.2p}(\theta)$ and $f_u(\theta)$. Since these are considered to be independent from each other in the mathematical model, a special parameter $k_{2\%,\theta}$ can be used for calculating the strength at an absolute strain of 2% with following expression:

$$f_{y,\theta} = f_{0.2p,\theta} + k_{2\%,\theta}\,(f_{u,\theta} - f_{0.2p,\theta}) \tag{A79}$$

For the use of advanced calculation methods, Table A.19 gives additional values for the stress-strain relationship of several stainless steels at elevated temperatures.

Table A.18 **Factors for determination of strain and stiffness of stainless steel at elevated temperatures**

Steel temperature θ_a °C	**Reduction factor (relative to E_a) for the slope of the linear elastic range** $k_{E,\theta} = E_{a,\theta} / E_a$	**Reduction factor (relative to f_y) for proof strength** $k_{0.2p,\theta} = f_{0.2p,\theta} / f_y$	**Reduction factor (relative to f_u) for tensile strength** $k_{u,\theta} = f_{u,\theta} / f_u$	**Factor for determination of the yield strength $f_{y,\theta}$** $k_{2\%,\theta}$
Grade 1.4301				
20	1.00	1.00	1.00	0.26
100	0.96	0.82	0.87	0.24
200	0.92	0.68	0.77	0.19
300	0.88	0.64	0.73	0.19
400	0.84	0.60	0.72	0.19
500	0.80	0.54	0.67	0.19
600	0.76	0.49	0.58	0.22
700	0.71	0.40	0.43	0.26
800	0.63	0.27	0.27	0.35
900	0.45	0.14	0.15	0.38
1 000	0.20	0.06	0.07	0.40
1 100	0.10	0.03	0.03	0.40
1 200	0.00	0.00	0.00	0.40
Grade 1.4401 / 1.4404				
20	1.00	1.00	1.00	0.24
100	0.96	0.88	0.93	0.24
200	0.92	0.76	0.87	0.24
300	0.88	0.71	0.84	0.24
400	0.84	0.66	0.83	0.21
500	0.80	0.63	0.79	0.20
600	0.76	0.61	0.72	0.19
700	0.71	0.51	0.55	0.24
800	0.63	0.40	0.34	0.35
900	0.45	0.19	0.18	0.38
1 000	0.20	0.10	0.09	0.40
1 100	0.10	0.05	0.04	0.40
1 200	0.00	0.00	0.00	0.40
Grade 1.4571				
20	1.00	1.00	1.00	0.25
100	0.96	0.89	0.88	0.25
200	0.92	0.83	0.81	0.25
300	0.88	0.77	0.80	0.24
400	0.84	0.72	0.80	0.22
500	0.80	0.69	0.77	0.21

Table A.18 **Factors for determination of strain and stiffness of stainless steel at elevated temperatures** *(continued)*

Steel temperature θ_a °C	**Reduction factor (relative to E_a) for the slope of the linear elastic range** $k_{E,\theta} = E_{a,\theta} / E_a$	**Reduction factor (relative to f_y) for proof strength** $k_{0.2p,\theta} = f_{0.2p,\theta} / f_y$	**Reduction factor (relative to f_u) for tensile strength** $k_{u,\theta} = f_{u,\theta} / f_u$	**Factor for determination of the yield strength $f_{y,\theta}$** $k_{2\%,\theta}$
600	0.76	0.66	0.71	0.21
700	0.71	0.59	0.57	0.25
800	0.63	0.50	0.38	0.35
900	0.45	0.28	0.22	0.38
1 000	0.20	0.15	0.11	0.40
1 100	0.10	0.075	0.055	0.40
1 200	0.00	0.00	0.00	0.40
Grade 1.4003				
20	1.00	1.00	1.00	0.37
100	0.96	1.00	0.94	0.37
200	0.92	1.00	0.88	0.37
300	0.88	0.98	0.86	0.37
400	0.84	0.91	0.83	0.42
500	0.80	0.80	0.81	0.40
600	0.76	0.45	0.42	0.45
700	0.71	0.19	0.21	0.46
800	0.63	0.13	0.12	0.47
900	0.45	0.10	0.11	0.47
1 000	0.20	0.07	0.09	0.47
1 100	0.10	0.035	0.045	0.47
1 200	0.00	0.00	0.00	0.47
Grade 1.4462				
20	1.00	1.00	1.00	0.35
100	0.96	0.91	0.93	0.35
200	0.92	0.80	0.85	0.32
300	0.88	0.75	0.83	0.30
400	0.84	0.72	0.82	0.28
500	0.80	0.65	0.71	0.30
600	0.76	0.56	0.57	0.33
700	0.71	0.37	0.38	0.40
800	0.63	0.26	0.29	0.41
900	0.45	0.10	0.12	0.45
1 000	0.20	0.03	0.04	0.47
1 100	0.10	0.015	0.02	0.47
1 200	0.00	0.00	0.00	0.47

Table A.19 **Reduction factor and ultimate strain for the use of advanced calculation methods**

Steel temperature θ_a **°C**	**Reduction factor (relative to E_a) for the slope of the linear elastic range** $k_{Ect,\theta} = E_{ct,\theta} / E_a$	**Ultimate strain** $\varepsilon_{u,\theta}$ [—]
Grade 1.4301		
20	0.11	0.40
100	0.05	0.40
200	0.02	0.40
300	0.02	0.40
400	0.02	0.40
500	0.02	0.40
600	0.02	0.35
700	0.02	0.30
800	0.02	0.20
900	0.02	0.20
1 000	0.02	0.20
1 100	0.02	0.20
1 200	0.02	0.20
Grade 1.4401 / 1.4404		
20	0.050	0.40
100	0.049	0.40
200	0.047	0.40
300	0.045	0.40
400	0.030	0.40
500	0.025	0.40
600	0.020	0.40
700	0.020	0.30
800	0.020	0.20
900	0.020	0.20
1 000	0.020	0.20
1 100	0.020	0.20
1 200	0.020	0.20
Grade 1.4571		
20	0.060	0.40
100	0.060	0.40
200	0.050	0.40
300	0.040	0.40
400	0.030	0.40
500	0.025	0.40
600	0.020	0.35
700	0.020	0.30
800	0.020	0.20

Table A.19 **Reduction factor and ultimate strain for the use of advanced calculation methods** *(continued)*

Steel temperature θ_a °C	**Reduction factor (relative to E_a) for the slope of the linear elastic range** $k_{Ect,\theta} = E_{ct,\theta} / E_a$	**Ultimate strain** $\varepsilon_{u,\theta}$ [—]
900	0.020	0.20
1 000	0.020	0.20
1 100	0.020	0.20
1 200	0.020	0.20
Grade 1.4003		
20	0.055	0.20
100	0.030	0.20
200	0.030	0.20
300	0.030	0.20
400	0.030	0.15
500	0.030	0.15
600	0.030	0.15
700	0.030	0.15
800	0.030	0.15
900	0.030	0.15
1 000	0.030	0.15
1 100	0.030	0.15
1 200	0.030	0.15
Grade 1.4462		
20	0.100	0.20
100	0.070	0.20
200	0.037	0.20
300	0.035	0.20
400	0.033	0.20
500	0.030	0.20
600	0.030	0.20
700	0.025	0.15
800	0.025	0.15
900	0.025	0.15
1 000	0.025	0.15
1 100	0.025	0.15
1 200	0.025	0.15

A.3.3.10 Cast iron and wrought iron

Cast iron is a brittle and variable quality material that is weak in tension. Accordingly it is unable to resist deformation in bending and has been known to cause catastrophic failure in fire. However cast iron can sustain high compressive loads and, for this reason, towards the end of the nineteenth century it was common to use a combination of cast iron columns and wrought iron beams. Since there is no detailed information available on the elevated temperature properties of cast iron, designers should be cautious when refurbishing older structures where cast iron members are present.

Wrought iron is more ductile than cast iron and performs in a similar way to structural steels at elevated temperatures. For design purposes the tensile strength of wrought iron can be considered to reduce linearly from 100% at ambient temperature to 10% at 750 °C.

$$\frac{F_t}{F_0} = 1 - 0.0012\theta_{wi} \qquad \text{(A80)}$$

Equation A80 should be regarded as characteristic. The wide variations in the quality of wrought iron give rise to wide variations in mechanical properties (see Kirby et al [54]).

A.3.3.11 Steel and cast and wrought iron – Poisson's ratio

The Poisson's ratio for all steels, and cast and wrought iron is independent of temperature at a value of 0.3.

A.4 Aluminium alloys

A.4.1 General

Aluminium is usually alloyed for structural applications. Aluminium alloys melt at relatively low temperatures of around 600 °C and soften at lower temperatures. Therefore, thermal and mechanical properties are usually only required for temperatures up to 500 °C.

A.4.2 Thermal properties of aluminium alloys

A.4.2.1 Thermal elongation of aluminium alloys

The thermal elongation of aluminium alloys, $\Delta L/L_{0,}$ is given by:

$$\Delta L/L_0 = 1 \times 10^{-8}\,\theta^2 + 2.25 \times 10^{-5}\theta - 4.5 \times 10^{-4}$$
$$\text{for } 0\ °\text{C} < \theta < 500\ °\text{C} \qquad \text{(A81)}$$

For simple calculations the relationship between thermal expansion and temperature can be considered linear and is given by:

$$\Delta L/L_0 = 2.5 \times 10^{-5}\,(\theta - 20) \qquad \text{(A82)}$$

A.4.2.2 Specific heat capacity of aluminium alloys

Aluminium is not magnetic and therefore does not undergo the change in specific heat capacity during heating that carbon steels do (see **A.3.2.4**).

The specific heat capacity of aluminium alloys, C_{al}, (J/kgK) increases linearly with temperature and is given by:

$$C_{al} = 0.41\theta + 903 \text{ for } 0\ °\text{C} < \theta < 500\ °\text{C} \qquad \text{(A83)}$$

A.4.2.3 Thermal conductivity of aluminium alloys

The thermal conductivity of aluminium alloys, λ_{al}, (W/mK) varies significantly with composition at elevated temperatures. For the different grades the following values should be used:

For alloys in the 3000 and 6000 series:

$$\lambda_{al} = 0.07\theta + 190 \quad \text{(A84)}$$

For alloys in the 5000 and 7000 series:

$$\lambda_{al} = 0.1\theta + 140 \quad \text{(A85)}$$

A.4.2.4 Density of aluminium alloys

The density of aluminium alloys, ρ_{al}, is approximately 2700 kg/m^3 and can be considered independent of temperature.

A.4.2.5 Emissivity of aluminium alloys

The emissivity should be taken as 0.3 for clean uncovered surfaces, and 0.7 for painted and covered (sooted) surfaces.

A.4.3 Mechanical properties – strength characteristics, elastic (Young's) modulus and stress-strain behaviour

A.4.3.1 General

The modulus of elasticity of aluminium alloys at elevated temperatures for a two hour exposure period is given in Table A.20.

Table A.20 **Elastic modulus of aluminium alloys at elevated temperatures**

Temperature, θ °C	**Modulus of elasticity**, $E_{al,\theta}$ N/mm^2 $\times$ 10^3
20	70.0
50	69.3
100	67.9
150	65.1
200	60.2
250	54.6
300	47.6
350	37.8
400	28.0
550	0

The 0.2% proof stress of aluminium alloys at elevated temperatures should be obtained from the stress ratio, $k_{0.2,\theta}$ given in Table A.21 and the 0.2% proof stress at ambient temperature, $f_{0.2}$. Effective 0.2% proof stress at elevated temperatures, θ, is given by:

$$f_{y,\theta} = k_{0.2,\theta}\, f_{0.2} \quad \text{(A86)}$$

Table A.21 **0.2% proof stress ratios, $k_{0.2,\theta}$ for aluminium alloys at elevated temperatures for up to 2 hours thermal exposure period**

Alloy	Temper	Aluminium alloy temperature °C							
		20	100	150	200	250	300	350	550
EN AW-3004	H34	1.00	1.00	0.98	0.57	0.31	0.19	0.13	0.00
EN AW-5005	O	1.00	1.00	1.00	1.00	0.82	0.58	0.39	0.00
EN AW-5005	H14 A)	1.00	0.93	0.87	0.66	0.37	0.19	0.10	0.00
EN AW-5052	H34 B)	1.00	1.00	0.92	0.52	0.29	0.20	0.12	0.00
EN AW-5083	O	1.00	1.00	0.98	0.90	0.75	0.40	0.22	0.00
EN AW-5083	H12 C)	1.00	1.00	0.80	0.60	0.31	0.16	0.10	0.00
EN AW-5454	O	1.00	1.00	0.96	0.88	0.50	0.32	0.21	0.00
EN AW-5454	H34	1.00	1.00	0.85	0.58	0.34	0.24	0.15	0.00
EN AW-6061	T6	1.00	0.95	0.91	0.79	0.55	0.31	0.10	0.00
EN AW-6063	T5	1.00	0.92	0.87	0.76	0.49	0.29	0.14	0.00
EN AW-6063	T6 D)	1.00	0.91	0.84	0.71	0.38	0.19	0.09	0.00
EN AW-6082	T4 E)	1.00	1.00	0.84	0.77	0.77	0.34	0.19	0.00
EN AW-6082	T6	1.00	0.90	0.79	0.65	0.38	0.20	0.11	0.00

A) The values may be applied also for temper H24/H34/H12/H32.

B) The values may be applied also for temper H12/H22/H32.

C) The values may be applied also for temper H22/H32.

D) The values may be applied also for EN AW-6060 T6 and T66.

E) The values do not include an increase in strength due to aging effects. It is recommended to ignore such effects.

A.4.3.2 Poisson's ratio for aluminium alloys

The Poisson's ratio for aluminium alloys can be considered independent of temperature at a value of 0.33.

A.5 Timber

A.5.1 Thermal properties of timber

A.5.1.1 Thermal shrinkage

Timber is combustible and the surface zone of timber exposed to fire undergoes dramatic changes in its response to heat between its pre- and post-carbonization (charring) phases. Timber shrinks at all stages of heat exposure. The extent of shrinkage is a function of moisture content, timber species, timber density, and the orientation of the grain. The latter point is important since the shrinkage in the direction of grain (longitudinal) is only approximately 10% of that recorded transverse to the grain. In addition, since the core of a timber structural member is insulated from the heat, and the effect of drying, longitudinal shrinkage is normally negligible in practice, whereas shrinkage of the cross-section can be significant.

A.5.1.2 Charring rate of timber

The charring depth is the distance between the outer surface of the original member and the position of the char-line and can be calculated from the time of fire exposure and the relevant charring rate.

The calculation of cross-sectional properties should be based on the actual charring depth including corner roundings. Alternatively, a notional cross-section without corner roundings can be calculated based on the notional charring rate. The position of the char-line should be taken as the 300-degree isotherm.

In BS EN 1995-1-2 the following guidance is provided.

The charring rate for one-dimensional charring can be taken as constant with time. The design charring depth can be calculated from the following equation.

$$d_{\text{char},0} = \beta_0 \, t \qquad \text{(A87)}$$

where

$d_{\text{char},0}$ is the design charring depth for one-dimensional charring;

β_0 is the one-dimensional design charring rate under standard fire exposure;

t is the time of fire exposure.

The notional charring rate, the magnitude of which includes an allowance for the effect of corner roundings and fissures should be taken as constant with time. The notional design charring depth can be calculated from the following equation.

$$d_{\text{char},n} = \beta_n \, t \qquad \text{(A88)}$$

where

$d_{\text{char},n}$ is the notional design charring depth, which includes the effect of corner roundings;

β_n is the notional design charring rate, the magnitude of which includes an allowance for the effect of corner roundings and fissures.

For surfaces of timber, unprotected throughout the time of fire exposure, design charring rates β_0 and β_n are given in Table A.22.

Table A.22 **Design charring rates for timber, LVL, wood based panels and panelling**

Timber type	β_0 mm/min	β_n mm/min
Softwood and beech:		
Glued laminated timber with a characteristic density of ≥ 290 kg/m^3	0.65	0.7
Solid timber with a characteristic density of ≥ 290kg/m^3	0.65	0.8
Hardwood:		
Solid or glued laminated hardwood with a characteristic density of 290 kg/m^3	0.65	0.7
Solid or glued laminated hardwood with a characteristic density of ≥ 450 kg/m^3	0.50	0.55
LVL with a characteristic density of ≥ 480 kg/m^3	0.65	0.7
Panels:		
Wood panelling	0.9 [A)]	—
Plywood	1.0 [A)]	—
Wood-based panels other than plywood	0.9 [A)]	—

A) The values apply to a characteristic density of 450 kg/m^3 and a panel thickness of 20 mm.

BS EN 1995-1-2 uses 450 kg/m^3 (also at 0% moisture content) as the density limit above which the slower charring rate applies.

These values relate to exposure to the standard test conditions and have been established during furnace controlled fire resistance tests. Experimental work has shown that for oxygen contents above 13%, the rate of charring starts to increase significantly. In addition, the charring rate increased when temperatures and heat fluxes were higher than those produced in standard tests. The charring behaviour in non-standard conditions should be established empirically.

For unprotected softwood the relationship between charring rate, β, and time t is shown in Figure A.26. Under parametric heating conditions the rate of charring is described as:

$$\beta_{\text{par}} = 1.5\beta_{\text{n}} \frac{0.2\Gamma^{1/2} - 0.04}{0.16\Gamma^{1/2} + 0.08} \tag{A89}$$

$$\Gamma = \frac{\left(\frac{O}{b}\right)^2}{\left(\frac{0.04}{1160}\right)^2} \tag{A90}$$

$$O = \frac{A_{\text{v}}}{A_{\text{t}}}\sqrt{h_{\text{eq}}} \tag{A91}$$

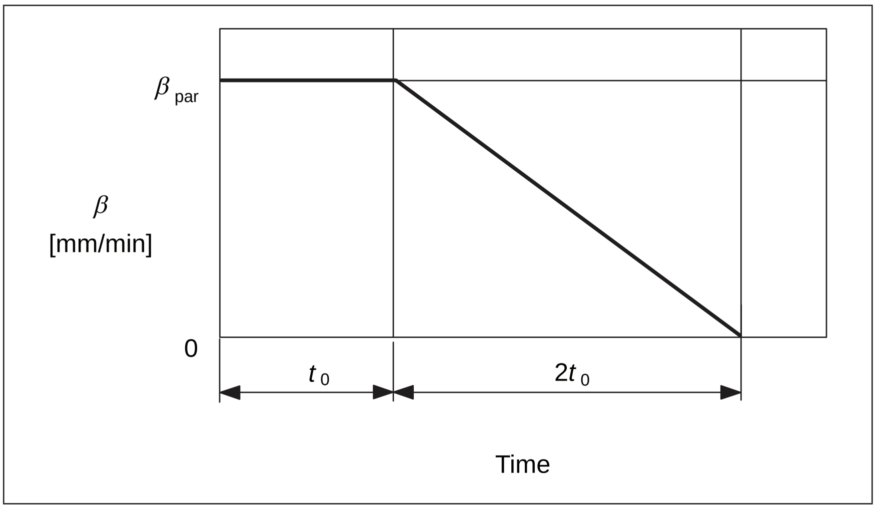
$$b = \sqrt{\rho c \lambda} \tag{A92}$$

$$h_{\text{eq}} = \sum \frac{A_i h_i}{A} \tag{A93}$$

where

- O is the opening factor, in m$^{0.5}$
- A_{v} is the total area of openings in vertical boundaries of the compartment (windows etc.) in m^2;
- A_{t} is the total area of floors, walls and ceilings that enclose the fire compartment, in m^2;
- A_i is the area of vertical opening i, in m^2;
- h_{eq} is the weighted average of the heights of all vertical openings (windows etc.), in metres;
- h_i is the height of vertical opening i, in metres;
- Γ is a factor accounting for the thermal properties of the boundaries of the compartment;
- b is the absorptivity for the total enclosure, see BS EN 1991-1-2:2002, Annex A;
- λ is the thermal conductivity of the boundary of the compartment, in Wm^{-1}K^{-1};
- ρ is the density of the boundary of the compartment, in kg/m^3;
- c is the specific heat of the boundary of the compartment, in Jkg^{-1}K^{-1}.

Figure A.26 **Relationship between charring rate and time**



The charring depth can be calculated from:

$$d_{char} = \beta_{par} t \text{ for } t \le t_0 \quad (A94)$$

$$d_{char} = \beta_{par}\left(1.5t - \frac{t^2}{4t_0} - \frac{t_0}{4}\right) \text{ for } t_0 < t \le 3t_0 \quad (A95)$$

$$d_{char} = 2\beta_{par} t_0 \text{ for } 3t_0 < t \le 5t_0 \quad (A96)$$

with

$$t_0 = 0.009\frac{q_{t,d}}{O} \quad (A97)$$

where

t_0 is the time period with a constant charring rate, in minutes;

$q_{t,d}$ is the design fire load density related to the total area of floors, walls and ceilings which enclose the fire compartment, in MJ/m^2, see BS EN 1991-1-2:2002.

The rules given above should only be used for:

$$t_0 \le 40 \text{ min} \quad (A98)$$

$$d_{char} \le b/4 \quad (A99)$$

$$d_{char} \le h/4 \quad (A100)$$

where

b is the width of the cross-section;

h is the depth of the cross-section.

An increased rate of charring can be expected at the arrises of timber members. Timber members also tend to suffer increased arris charring at the interface between timber layers.

The charring rate of glued laminated timber members can be treated in the same way as solid timber when any of the following adhesives are used:

- phenolic and aminoplastic resin;
- resorcinol fomaldehyde;
- phenol formaldehyde;
- phenol-resorcinol formaldehyde;
- urea-formaldehyde; and
- urea-melamine-formaldehyde.

These adhesives have demonstrated that, in terms of their charring rate, they can be treated as solid timber. Complex "glulam" beams, where higher grade timbers are used at the extremities of the section with low grade timbers in the core, can char at different rates.

Impregnation with flame retardant salts has been shown to increase the charring rate of timber.

A.5.1.3 Specific heat capacity and density of timber

The basic specific heat capacity, C_t, of uncharred timber is:

$$C_t = 1.114 + 4.86 \times 10^{-3}\,\theta \qquad \text{(A101)}$$

The specific heat capacity and ratio of density to dry density of softwood at elevated temperatures for service class 1, as defined in BS EN 1995-1-2, is given in Table A.23.

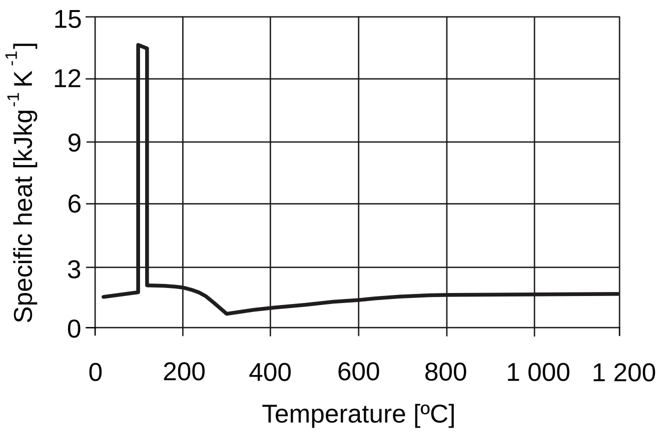
Table A.23 **Variation of specific heat capacity and density ratio of softwood at elevated temperatures**

Temperature °C	**Specific heat capacity** kJ kg^{-1} K^{-1}	**Density ratio**
20	1.53	1 + ω
99	1.77	1 + ω
99	13.60	1 + ω
120	13.50	1.00
120	2.12	1.00
200	2.00	1.00
250	1.62	0.93
300	0.71	0.76
350	0.85	0.52
400	1.00	0.38
600	1.40	0.28
800	1.65	0.26
1 200	1.65	0

NOTE ω is the moisture content as a fraction by mass.

The variation in specific heat of wood and charcoal at elevated temperatures is shown in Figure A.27.

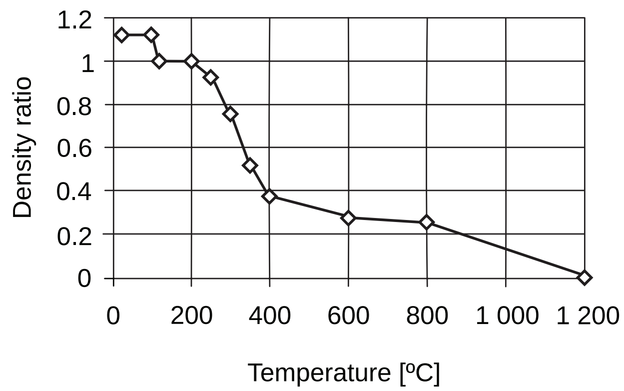
Figure A.27 **Variation in specific heat of softwood and charcoal**



Within the uncharred core of a fire exposed timber section, the density can be assumed to retain its ambient temperature value.

At elevated temperatures the density of softwood with an initial moisture content of 12% is shown in Figure A.28.

Figure A.28 **Temperature-density ratio relationship for softwood with an initial moisture content of 12%**



A.5.1.4 Thermal conductivity of timber

The thermal conductivity of uncharred timber, λ_t, is given by:

$$\lambda_t = \left(2.41 + 0.048P_w\right)\frac{\rho_t}{\rho_w} + 0.982 \qquad \text{(A102)}$$

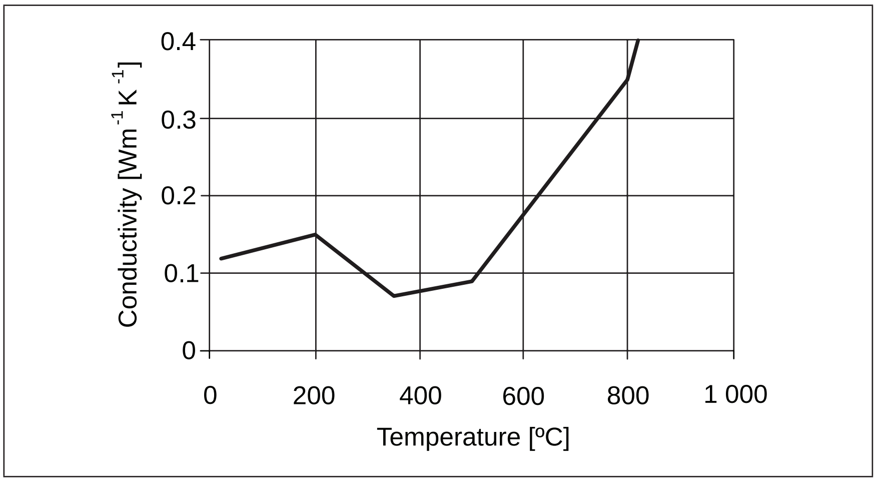
The thermal conductivity values of the char layer are apparent rather than measured values of charcoal in order to take into account increased heat transfer due to shrinkage cracks above ~500 °C and the consumption of the char layer at ~1 000 °C

Depending upon the model used for calculation, modification of the thermal properties might be necessary (see Table A.24 and Figure A.29).

Table A.24 **Variation of thermal conductivity with temperature**

Temperature °C	Thermal conductivity $Wm^{-1}K^{-1}$
20	0.12
200	0.15
350	0.07
500	0.09
800	0.35
1 200	1.50

Figure A.29 **Variation in thermal conductivity with temperature for wood and charcoal**



A.5.1.5 Emissivity of timber

The emissivity of timber can be taken as 1.0 for all temperatures.

A.5.2 Mechanical properties – strength characteristics, elastic modulus and stress-strain behaviour

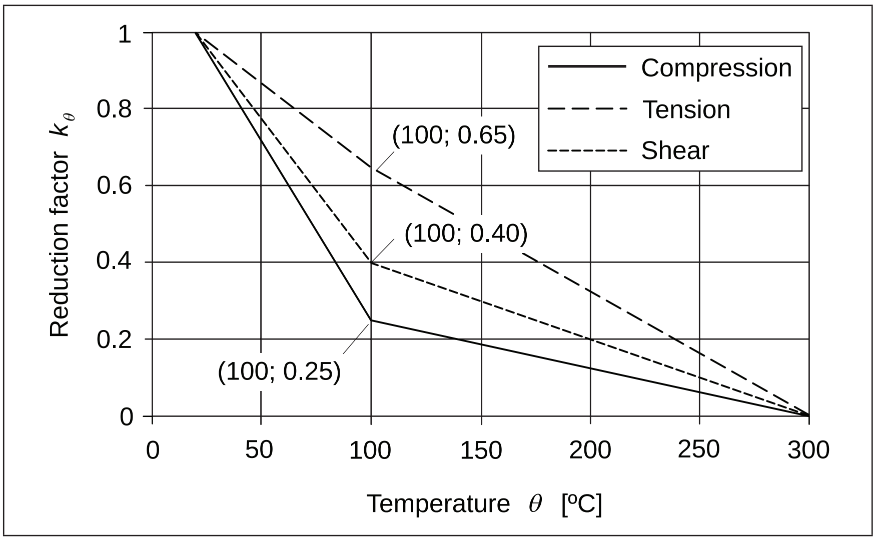
A.5.2.1 General

On exposure to fire, the outermost surface of timber elements burns, forming a char. Continued exposure results in a gradual erosion of the surface and the extension of the charred zone further into the depth of the timber. Only charred timber is affected by fire and the internal residual uncharred section is capable of maintaining a loadbearing function. The timber element continues to carry its design load only as long as the unaffected residual section retains a sufficient load carrying capacity.

A.5.2.2 Strength in compression, tension and shear

The strength for softwoods at elevated temperatures can be determined by multiplying by a temperature dependent reduction factor in accordance with Figure A.30.

Figure A.30 **Reduction factor for strength parallel to the grain for softwood**



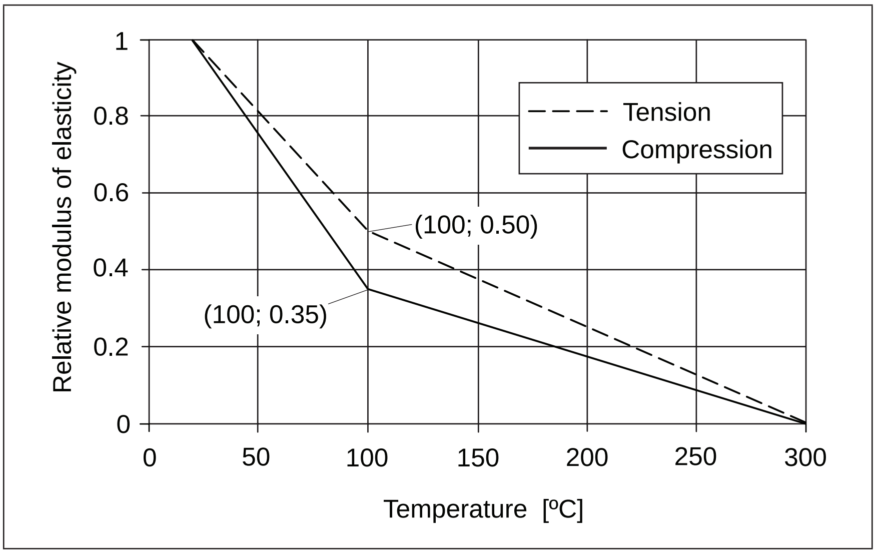
For compression perpendicular to the grain, the same reduction of strength can be applied as for compression parallel to the grain.

For shear with both stress components perpendicular to the grain (rolling shear), the same reduction of strength can be applied as for compression parallel to the grain.

A.5.2.3 Elastic modulus in tension and compression

The elastic modulus for softwoods at elevated temperatures can be determined by multiplying by a temperature dependent reduction factor in accordance with Figure A.31.

Figure A.31 **Effect of temperature on the elastic modulus of softwood parallel to the grain**



A.6 Masonry units (e.g. concrete blocks, bricks)

A.6.1 Thermal properties of masonry units

A.6.1.1 Thermal strain and shrinkage of masonry units

Masonry is a generic description of a wide range of materials. Concrete based masonry units have thermal elongation properties similar to concrete itself. Bricks are thermally inert and their expansion on heating is normally negligible.

BS EN 1996-1-2 provides graphical data on the thermal strain of a number of masonry units (see Figure A.32, Figure A.33 and Figure A.34).

Figure A.32 **Calculation values of thermal strain ε_T of clay units with unit strength 12 N/mm^2 to 20 N/mm^2 and units with a density range of 900 kg/m^3 to 1 200 kg/m^3**

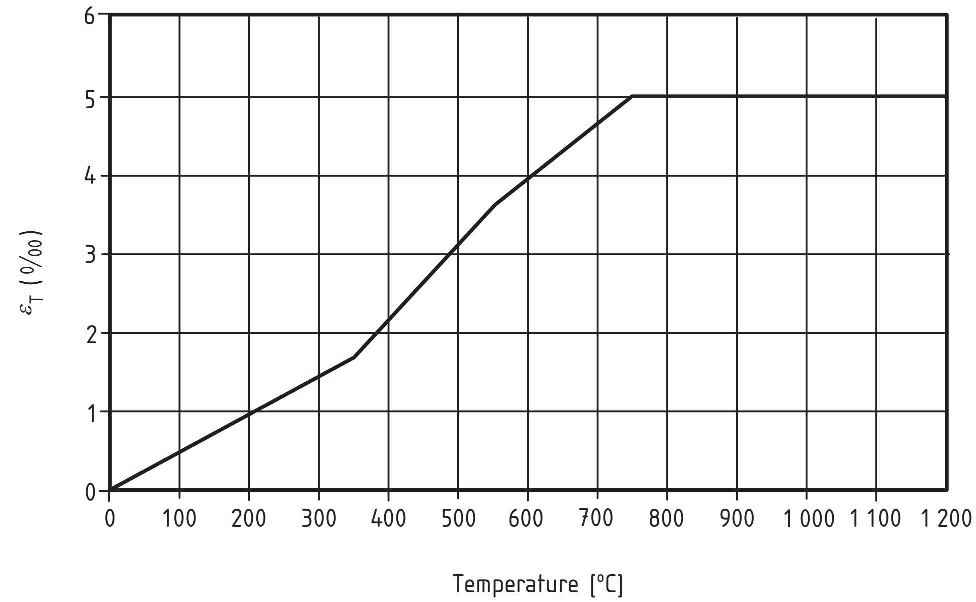


Figure A.33 **Calculation values of thermal strain ε_T of calcium silicate units with unit strength 12 N/mm^2 to 20 N/mm^2 and a density range of 1 600 kg/m^3 to 2 000 kg/m^3**

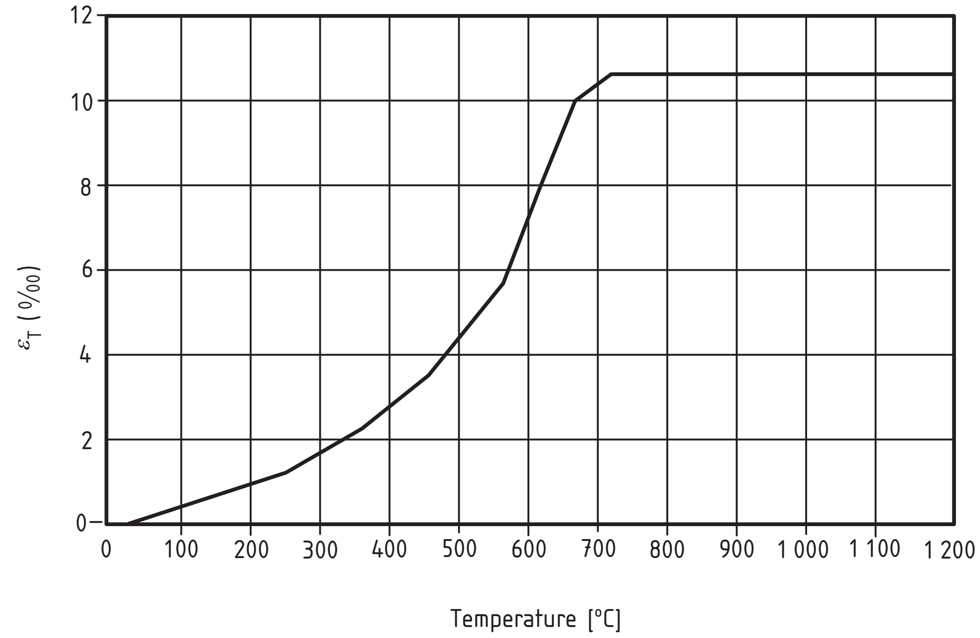
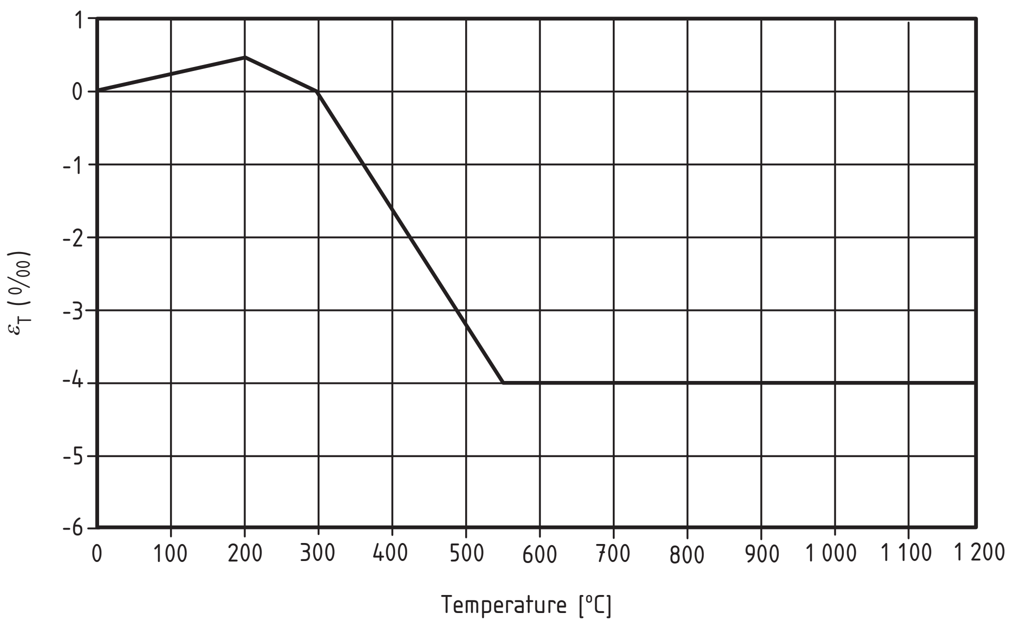


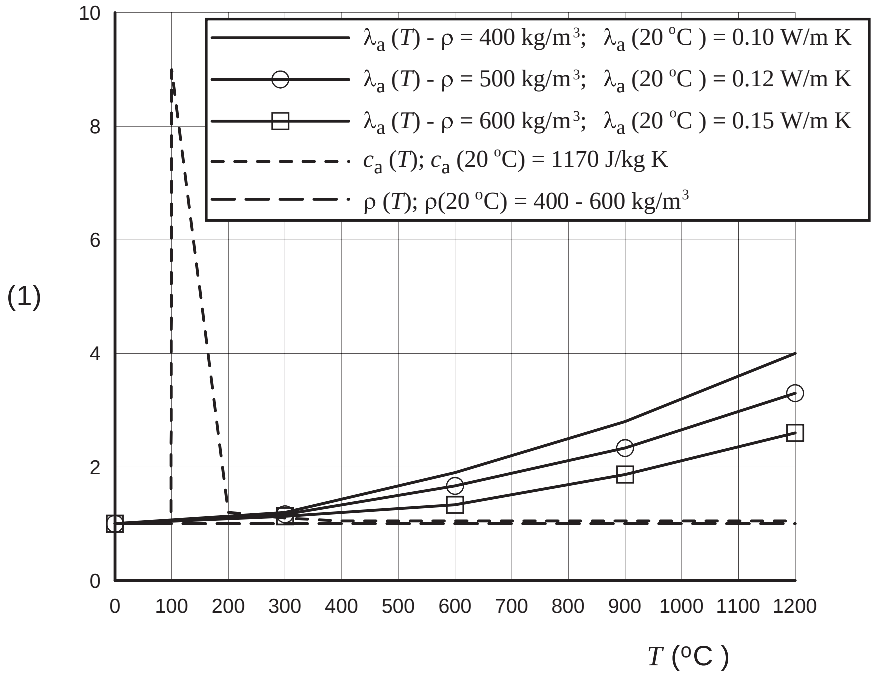
Figure A.34 **Calculation values of thermal strain ε_T of lightweight aggregate concrete units (pumice) with unit strength 4 N/mm^2 to 6 N/mm^2 and a density range of 600 kg/m^3 to 1 000 kg/m^3**



A.6.1.2 Thermal conductivity, specific heat and density of masonry units

The thermal conductivity, specific heat and density of several types of masonry units are as follows (see Figure A.35, Figure A.36, Figure A.37 and Figure A.38).

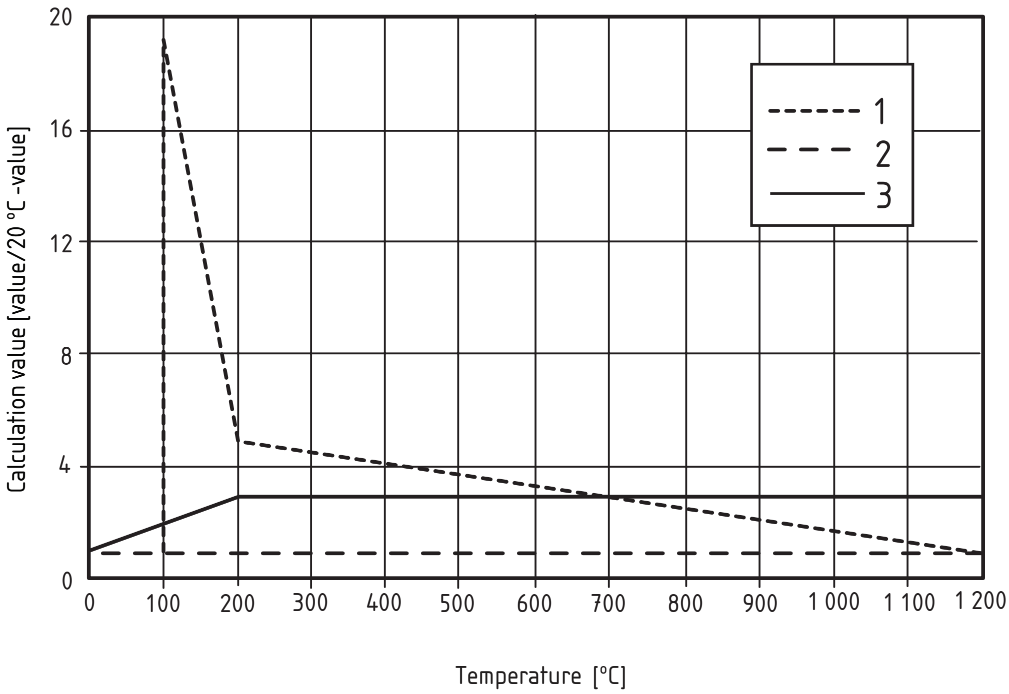
Figure A.35 **Calculation values of temperature-dependant material properties of autoclaved aerated concrete units with a density range of 400 kg/m³ to 600 kg/m³**



Key

T (°C)	temperature
λ_a	thermal conductivity
c_a	specific heat capacity
ρ	density kg/m³
1	ratio of value at temperature T to that at 20 °C

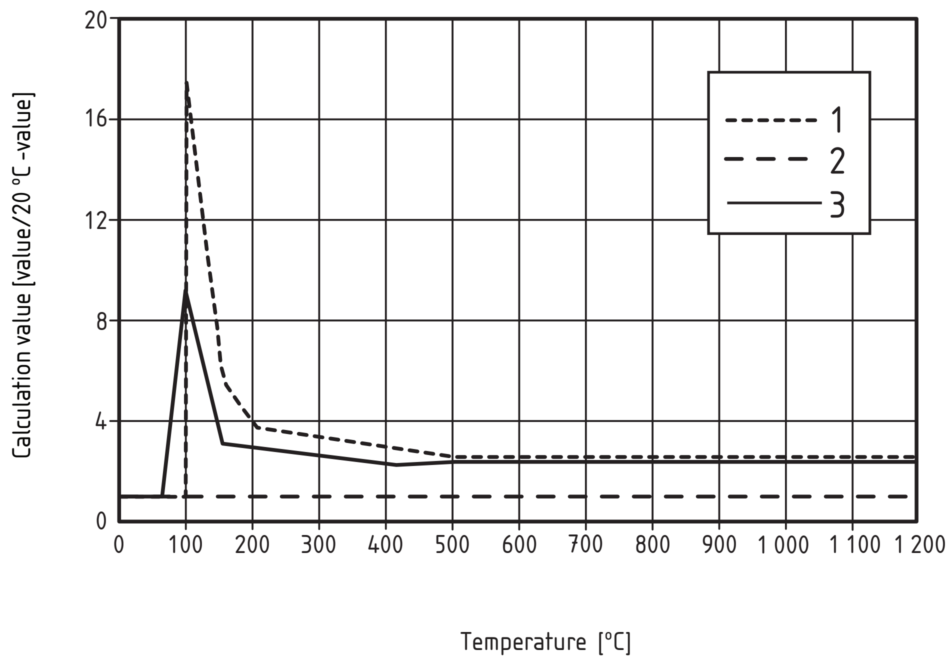
Figure A.36 **Calculation values of temperature-dependant material properties of clay units with a density range of 900 kg/m³ to 1 200 kg/m³**



Key

1 $c_a(T); c_a(20\ °C) = 564$ J/kg K 2 $\rho(T); \rho(20\ °C) = 900$ kg/m³ to 1 200 kg/m³ 3 $\lambda_a(T); \lambda_a(20\ °C) = 0.42$ W/mK

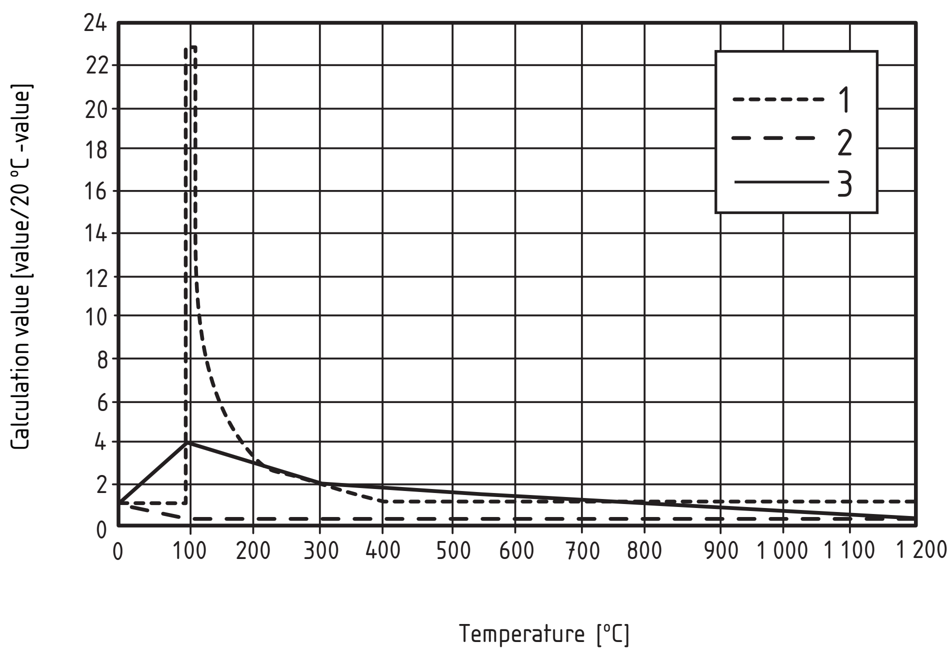
Figure A.37 **Calculation values of temperature-dependant material properties of lightweight aggregate concrete units (pumice) with a density range of 600 kg/m³ to 1 000 kg/m³**



Key

1 $c_a(T); c_a(20\ °C) = 1170$ J/kg K 2 $\rho(T); \rho(20\ °C) = 600$ kg/m³ to 1000 kg/m³ 3 $\lambda_a(T); \lambda_a(20\ °C) = 0.21$ W/m K

Figure A.38 **Calculation values of temperature-dependant material properties of calcium silicate units with a density range of 1 600 kg/m³ to 2 000 kg/m³**



Key

1 $c_a(T); c_a(20\ °C) = 1020$ J/kg K 2 $\rho(T); \rho(20\ °C) = 1600$ kg/m³ to 2 000 kg/m³ 3 $\lambda_a(T); \lambda_a(20\ °C) = 1.0$ W/m K

A.6.1.3 Emissivity of masonry units

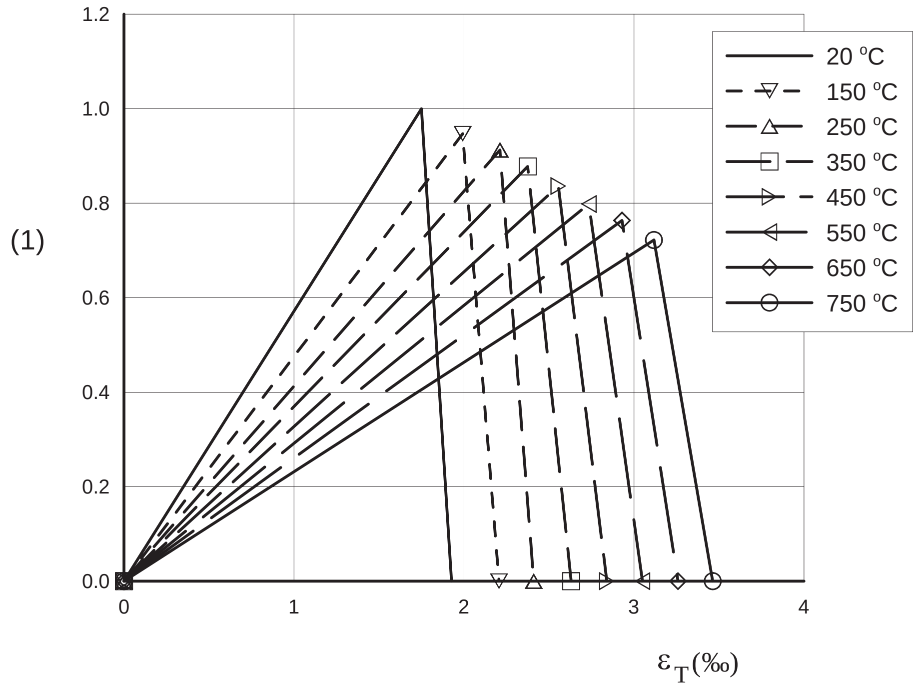
The emissivity of clay-based masonry units can be taken as 0.95. Concrete based units have an emissivity of 1.0.

A.6.2 Mechanical properties – strength characteristics, elastic modulus and stress-strain behaviour

The mechanical properties of masonry units at elevated temperatures are similar to those properties exhibited by the parent material.

The stress-strain behaviour for masonry relative to its nominal ambient temperature characteristic strength is as follows (see Figure A.39, Figure A.40, and Figure A.41).

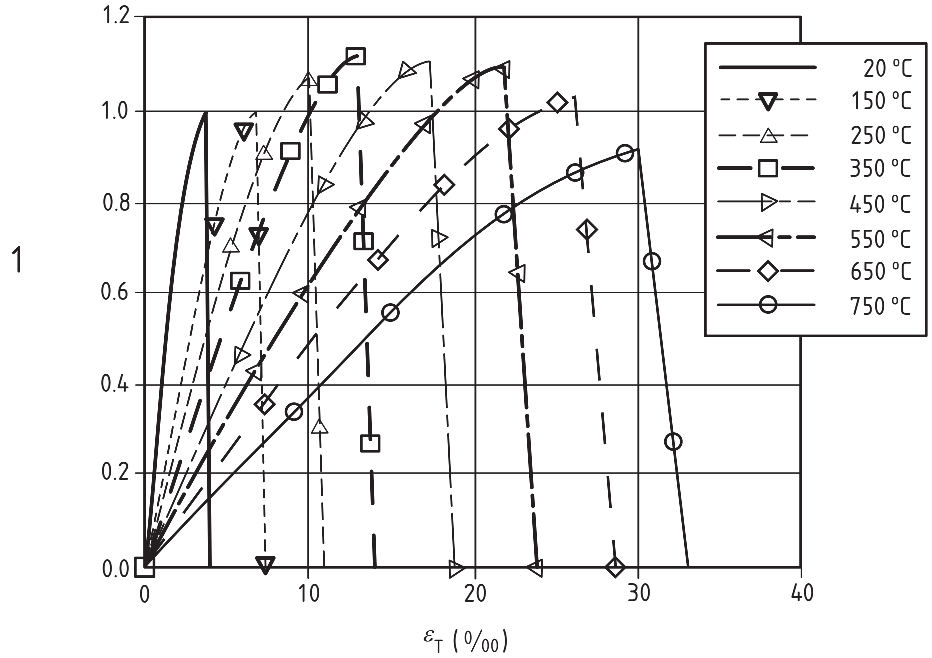
Figure A.39 **Calculation values of temperature-dependant stress-strain diagrams of clay units with unit strength of 12 N/mm^2 to 20 N/mm^2 and a density range of 900 kg/m^3 to 1200 kg/m^3**



Key

1 Ratio of strength at temperature *T* to that at 20 °C

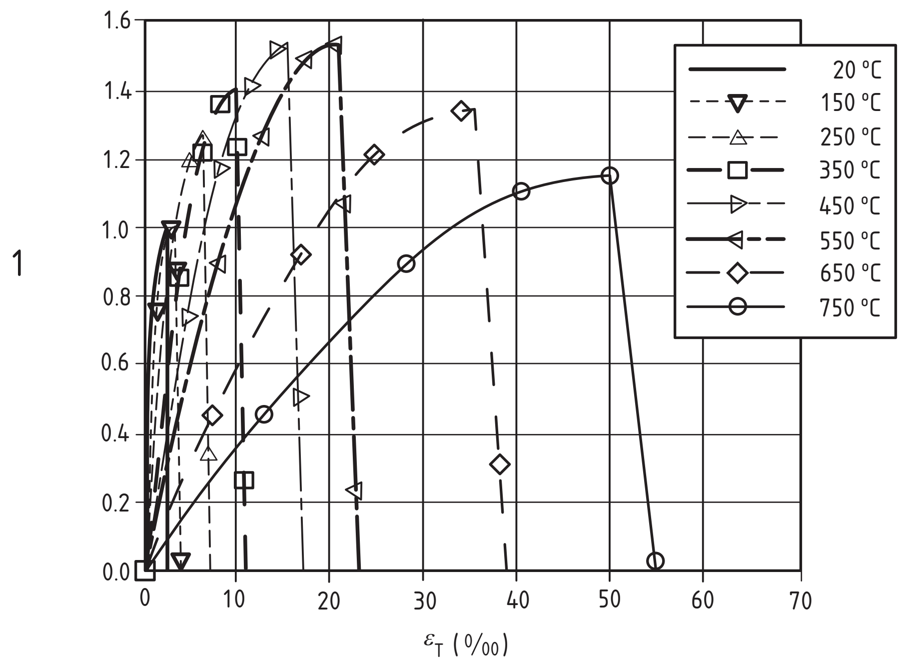
Figure A.40 **Calculation values of temperature dependent stress-strain curves of calcium silicate units with strength of 12 N/mm² to 20 N/mm² and a density range of 1600 kg/m³ to 2000 kg/m³**



Key

1 Ratio of strength at temperature *T* to that at 20 °C

Figure A.41 **Calculation values of temperature-dependent stress-strain curves of lightweight aggregate concrete units (pumice) with strength of 4 N/mm² to 6 N/mm² and a density range of 600 kg/m³ to 1000 kg/m³**



Key

1 Ratio of strength at temperature *T* to that at 20 °C

A.7 Glass

A.7.1 Thermal properties

The thermo-physical properties of common glasses are given in Table A.25.

Table A.25 **Thermal properties of common types of glass**

Property	Units	Fused quartz (silica)	Borosilicate	Soda lime	Glass ceramic
Expansion	10^{-6}/K	0.55	3.3 (20 °C to 300 °C)	9 (20 °C to 300 °C)	6.3 (20 °C to 1 000 °C)
Conductivity	W/mK	1.4	1.14	1	1.7
Specific heat	J/kgK	740	750	720	880

A.7.2 Mechanical properties

The behaviour of glasses of all types is very variable and therefore the mechanical properties should be taken as typical rather than absolute.

The uncertainty concerning a characteristic value applies primarily to tensile strength which is substantially influenced by the glassy structure, so a single characteristic tensile strength value cannot be given. Measurements on pristine samples, e.g. taken from different float lines across Europe, show a measured range of 30 N/mm^2 to 120 N/mm^2. Strengths on freshly drawn fibres can be as high as ~5 000 N/mm^2.

Glass ceramics fall into a different performance category from either soda-lime or borosilicate. The performance of the ceramic type used for fire resistance is good as there is no thermal expansion, but it is brittle. Its mechanical performance does not allow impact safety rating without laminating with an impact resistant interlayer, e.g. PVB. Glass ceramics refers to a group of widely differing compositions used for different applications and properties need to be identified with the specific type.

The mechanical properties of some commonly used glasses are given in Table A.26.

Table A.26 **Mechanical properties of some common glasses**

Property	Units	Glass type (generic)			
		Quartz (fused silica)	Borosilicate	Soda lime	Glass ceramic
Density	kg/m^3	2 200	2 230	2 500	2 530
Elastic modulus	10^4 MPa	7.2	6.3	7.3	9.1
Shear modulus	10^4 MPa	31	—	3	—
Poisons ratio	—	0.17	0.20 (25 °C to 400 °C)	0.22	0.24
Max service temperature	°C	1 000	500	450	600
Softening point	°C	1 683	800	726	—
Tensile strength	Pa	70	70	—	—
Compressive strength	MPa	1 108	—	—	~2 000
Shear strength	Pa	70	—	—	—

A.8 Plastics and composites

A.8.1 General

A wide range of plastics and composites (fibres and resins) are used in load bearing construction. These can be in the form of extruded shapes such as H sections and I beams, part of a composite section working in combination with another material, e.g. steel, or as a laminate which can be used to reinforce another structural element.

The majority of plastics and composites do not survive at very high temperatures, however, those that do are able to form a stable char which insulates the remainder of the section, similar to the way that charred wood protects unburnt timber.

Some general information about composites is provided as a comparison with other materials used in building construction.

A.8.2 Thermal properties

A.8.2.1 Plastics

Specific types of plastics and resins are introduced in Table A.27 by their trade names along with some of the more common types.

A.8.2.2 Composites

A.8.2.2.1 Ceepree

Ceepree is a blend of unleaded glass frits, which comes from a variety of sources. It is used as a filler in thermoplastic and thermosetting resins.

Ceepree has fire barrier and smoke suppression properties and prevents repeat ignition in composite materials by the following mechanisms.

- When heated beyond its activation temperature of around 350 °C, the low melting point components within the Ceepree formulation begin to melt, causing vitreous material to flow around the burning resin.
- The resultant encapsulation inhibits the access of oxygen to combustible materials and restrains carbonaceous decomposition products from being emitted as smoke.
- At higher temperatures, around 750 °C to 850 °C, components in the Ceepree formulation devitrify, which is a transition from a glassy state to a crystalline state.
- At these temperatures Ceepree acts as a high temperature adhesive, bonding the composites together. In effect, the composites then behave like a ceramic material.
- The curing described above enables the mechanical strength of the composite to be retained at the high temperatures at which it would otherwise have been lost.

A.8.2.2.2 Aluminium trihydroxide

Aluminium trihydroxide has the chemical formula $Al(OH)_3$ and is also known as aluminium hydroxide or alumina trihydrate, commonly abbreviated to ATH.

It starts to give off its chemically bonded combined water at a temperature above 200 °C. Aluminium trihydroxide is used as fire barrier filler and is suitable for use in thermoplastic and thermosetting resins.

Table A.27 **Thermal properties of some common plastics**

Material	**Parameter**					
	Melting point °C	**Linear expansion** /K	**Maximum operating temperature** °C	**Thermal conductivity** W/mK	**Specific heat** J/kgK	**Density** kg/m^3
Polytetrafluorethene (PTFE)	327	10×10^{-5}	260	0.25	1.4	2 000–2 300
Noryl [A)]	230–315	6×10^{-5}	90	0.22	—	1 060
Polyfenylene-oxide	310–340	5.2×10^{-5}	105	—	—	1 060
Thermoplastic polyester	165	7×10^{-5}	100	0.19	1.3	1 370
Polyacetate	164–167	130×10^{-6}	90–140	0.25–0.30	1.5	1 410
Polyamide (Nylon 6) [A)] (PA)	170	95×10^{-6}	140	0.17–0.30	1.6	1 130
Acrylonitril-butadieenstyrene (ABS)	220	85×10^{-6}	80	0.13–0.19	2.0–2.1	1 070
Polystyrene (PS)	160	90×10^{-6}	70	0.12–0.19	1.2–2.1	1 050
Polymethylmethacrylate (PMMA)	180	80×10^{-6}	70	0.19	1.47	1 180
Polycarbonate (PC)	225	60×10^{-6}	130	0.19–0.21	1.0–1.2	1 200
Polypropylene	160	160×10^{-6}	130	0.10–0.13	2.0	920
Soft polyvinylchloride (PVC)	165	$70\text{–}100 \times 10^{-6}$	50	~0.24	~1.05	1 200
Polyvinylchloride-chloride (PVC-C) (high temperature)	195	$6\text{–}8 \times 10^{-5}$	95–100	~0.24	~1.05	1 540
High impact strength polyvinylchloride (PVC)	120–130	100×10^{-6}	70	~0.24	~1.05	1 380
Hard-polyvinylchloride (PVC)	120–130	80×10^{-6}	70	~0.24	~1.05	1 390
Perspex [A)]	—	2.3×10^{-4}	88	—	—	1 200
Nylon [A)]	—	10×10^{-5}	—	0.3	17×10^2	1 140
Polystyrene	—	7×10^{-5}	—	0.08	13×10^2	1 060
Polythene	110–135	—	41–120	0.29–0.5	2.2	914–1 400

A) These are trade names and are given for the convenience of users of this Published Document. This does not constitute an endorsement by BSI of the products named.

A.8.3 Mechanical properties

A.8.3.1 Plastics

The mechanical properties of some plastics are given in Table A.28.

Table A.28 **Mechanical properties of some plastics**

Material	Parameter						
	Density kg/m^3	**Bending strength** N/mm^2	**Strain at failure** %	**Compressive strength** N/mm^2	**Elastic modulus** N/mm^2	**Tensile strength** N/mm^2	**Weakening point (Vicat)** °C
Polytetrafluorethene PTFE	2 000–2 300	18–20	300–500	10	400	17.5–26	327
Noryl [A)]	1 060	95	20	115	2 500	67	130
Polyfenylene-oxide	1 060	98–105	80	120	2 500	75	191
Thermoplastic polyester	1 370	12.5	150	≈ 100	3 500	74	≈ 150
Polyacetate	1 410	110	28	90	3 000	68	154
Polyamide (Nylon 6) [A)] (PA)	1 130	27	170	90	1 300	43	215–220
Acrylonitril-butadieenstyrene (ABS)	1 070	60	3	47	2 500	40	90
Polystyrene (PS)	1 050	80	15	90	2 600	42.5	70
Polymethylmethacrylate (PMMA)	1 180	140	3.5	120	3 250	75	115
Polycarbonate (PC)	1 200	75	>110	80	2 200	65	170
Polypropylene	920	45	650	46	1 300	275–300	90
Soft polyvinylchloride (PVC)	1 200	85	370–400	75	≈ 50	16–18	50–60
Polyvinylchloride-chloride (PVC-C) (high temperature)	1 540	—	≈ 100	≈ 70	70–80	57	105
High impact strength polyvinylchloride (PVC)	1 380	50	60–70	110	2 500	23–40	55–75
Hard-polyvinylchloride (PVC)	1 390	80–110	20–50	80	3 000	50–60	80
Perspex [A)]	1 200	116	4	—	2 500–3 500	75	110
Nylon [A)]	1 140	11	50	70	2 800	86	—

[A)] These are trade names and are given for the convenience of users of this Published Document. This does not constitute an endorsement by BSI of the products named.

A.8.3.2 Composites

The mechanical properties of several types of resin are given in Table A.29.

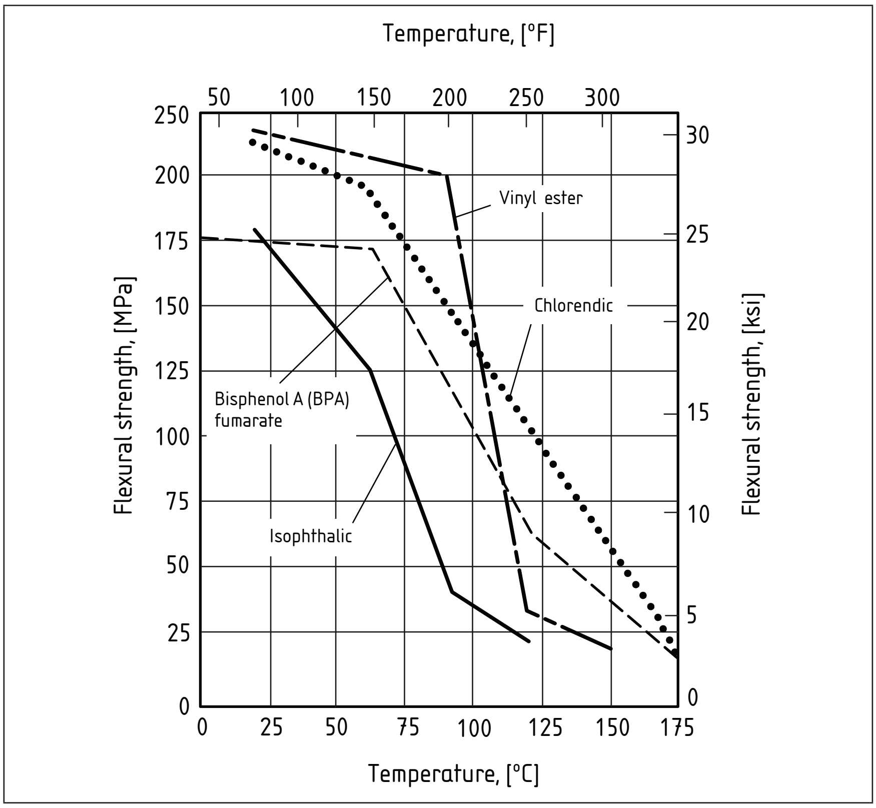
Table A.29 **Mechanical properties of some typical resins**

Material	Type [A)]	Tensile strength N/mm^2	Elastic modulus kN/mm^2	Elongation at fracture %	Flexural strength N/mm^2	Shear strength N/mm^2	Heat distortion temperature °C
Vinyl ester resins	Bisphenol-A	82	3.5	6	131	—	102
	Novolac	68	3.5	3–4	125	—	150
Phenolic resins	—	24–40	1.5–2.5	1.8	60–80	—	250
Epoxy resins	DGEBA/APTA Amine cured at 20° C	62	3.2	2	—	61	62
	DGEBA/MDA Amine cured at 12 °C	90	3.0	8	—	52	121
	Rubber modified MDA/MPDA	125	4.1	5	—	84	110

A) DGEBA: Diglycidyl ether of bisphenol A
APTA: Polyoxy propyleneamine
MDA: 4, 4' – methylene dianiline
MPDA: Metaphenylene diamine

The flexural strength of glass polyester at elevated temperatures is shown in Figure A.42.

Figure A.42 **Flexural strength of glass polyester at elevated temperatures**



The mechanical properties of several common types of fibre reinforcement are given in Table A.30.

Table A.30 **Mechanical properties of several types of fibre reinforcement**

Material	**Type**	**Elastic modulus (tension)** kN/mm^2	**Ultimate strength** N/mm^2	**Strain at failure** %
Glass	E	72	3 450	4.8
	A	70	3 030	4.4
	R	86	4 400	—
	ECR,C	69	3 030	4.4
Carbon	T300	230	3 530	1.5
	T800	294	5 490	1.9
	T1000	294	7 060	2.4
Aramid	Kevlar (49) [A)]	125	2 750	2.4

[A)] These are trade names and are given for the convenience of users of this Published Document. This does not constitute an endorsement by BSI of the products named.

Whilst the individual properties of the component parts of a composite are important in the selection of the materials, it is the overall properties that control their performance.

The strength properties of fibre reinforced polyester laminates are given in Table A.31.

Table A.31 **Strength properties of polyester laminates at elevated temperatures**

Temperature °C	**Parameter**			
	Flexural strength mN/mm^2	**Flexural modulus** mN/mm^2	**Tensile strength** mN/mm^2	**Tensile modulus** mN/mm^2
20	193	6 896	103	7 586
50	151	6 206	82	6 896
80	107	5 862	57	5 862
100	27	3 448	20	3 448

Annex B (informative) Temperature-dependent properties of non-loadbearing construction systems – Thermal properties of materials used in composite sandwich panels

B.1 General

Composite sandwich panels are primarily made up of a metal outer skin (coated carbon steel, stainless steel or aluminium) between which there is an insulating material consisting of either foam or an inert fibre.

The panel thickness varies according to the application, i.e. refrigeration, normal temperature environment. Several of the foams are combustible if the core is exposed to fire, therefore, careful consideration should be given to where and how panels are used.

The following thermal data might be helpful, but specific formulations can have slightly different performances.

B.2 Comparison of thermal expansion

A comparison of the thermal expansion of materials used in composite sandwich panels at ambient temperature is given in Table B.1.

Table B.1 **Comparison of expansion of materials used in composite sandwich panels**

Material	**Expansion, $\times 10^{-6}$** mm/mm K
Mineral (rock) wool	Negligible
Cellular glass	8.5
Expanded polystyrene	70
Extruded polystyrene	80
Phenolic foam	80
Polyurethane	100
Polyisocyanate	120
Carbon steel facing	14
Aluminium	25
Stainless steel	19

B.3 Comparison of specific heat capacity

A comparison of the specific heat capacity of materials used in composite sandwich panels at ambient temperature is given in Table B.2.

Table B.2 **Comparison of specific heat capacity of materials used in composite sandwich panels**

Material	**Specific heat capacity** kJ/kgK
Mineral (rock) wool	0.75–0.84
Polyurethane foam	1.26
Polystyrene	1.30
Expanded polystyrene	1.52
Steel	0.42
PVC	0.84–1.170
Plasterboard	0.95

B.4 Thermal conductivity

B.4.1 General

Data on the thermal conductivity of materials used in composite sandwich panels are given in **B.4.2** to **B.4.8**.

B.4.2 Mineral (rock) wool – typical values

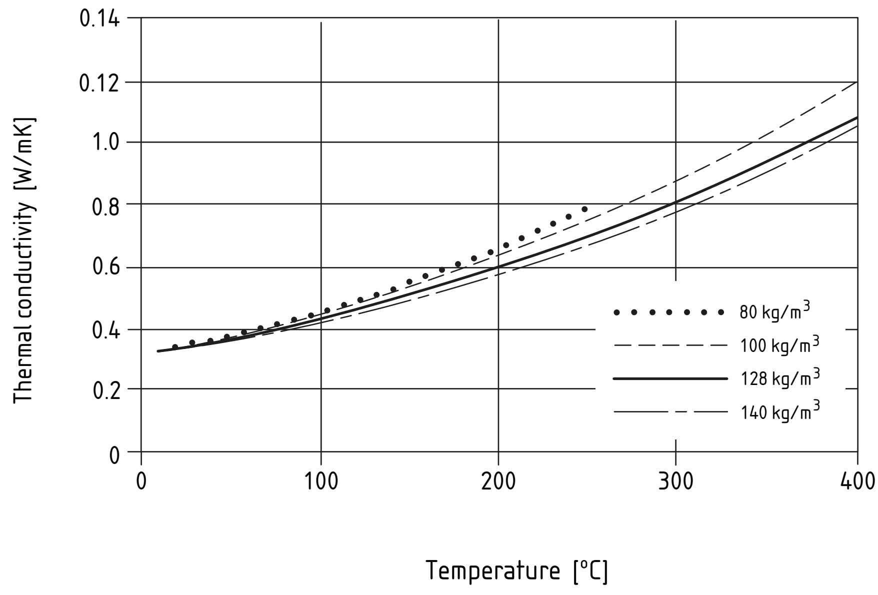
The thermal conductivity of mineral wool at elevated temperatures is given in Table B.3 for various densities.

Table B.3 **Thermal conductivity for various densities of mineral (rock) wool at elevated temperatures**

Mean temperature °C	Thermal conductivity, W/mK, at densities of:			
	80 kg/m^3	100 kg/m^3	128 kg/m^3	140 kg/m^3
10	0.033	0.033	0.033	0.033
50	0.038	0.037	0.037	0.037
100	0.045	0.044	0.044	0.044
150	0.055	0.054	0.052	0.051
200	0.066	0.064	0.061	0.060
250	0.079	0.075	0.071	0.070
300	—	0.088	0.082	0.081
350	—	0.104	0.096	0.093
400	—	0.122	0.109	0.106

Table B.3 is presented graphically in Figure B.1.

Figure B.1 **Thermal conductivity for various densities of mineral (rock) wool at elevated temperatures**



The thermal conductivity of mineral wool can be approximated at the mean temperature of the insulation, from:

$$\lambda_i = a_0 + a_1 T_i + a_2 T_i^2 \quad \text{(B1)}$$

where:

a_0, a_1, a_2 are constants for a particular product and product density as given in Table B.4;

T_i is the mean temperature of the insulation.

Table B.4 **Constants for calculating the thermal conductivity of mineral wool at elevated temperatures**

Density kg/m^3	**Constants**		
	$a_0 \times 10^{-3}$	$a_1 \times 10^{-6}$	$a_2 \times 10^{-8}$
80	32.04	96.87	36.45
100	32.05	89.59	33.41
128	31.88	96.41	24.23

B.4.3 Cellular glass

The thermal conductivity for two densities of cellular glass is given in Table B.5.

Table B.5 **Thermal conductivity of cellular glass**

Mean temperature °C	**Thermal conductivity, W/mK, at densities of:**	
	120 kg/m^3	**135 kg/m^3**
0	0.038	0.044
10	0.040	0.046

B.4.4 Expanded polystyrene

The thermal conductivity for various densities of expanded polystyrene is given in Table B.6.

Table B.6 **Thermal conductivity of expanded polystyrene**

Mean temperature °C	**Thermal conductivity, W/mK, at densities of:**			
	15 kg/m^3	**20 kg/m^3**	**25 kg/m^3**	**30 kg/m^3**
10	0.038	0.035	0.033	0.033

B.4.5 Extruded polystyrene

The thermal conductivity for various densities of extruded polystyrene is given in Table B.7.

Table B.7 **Thermal conductivity of extruded polystyrene**

Mean temperature °C	Thermal conductivity, W/mK, at densities of:			
	28 kg/m^3	**32 kg/m^3**	**38 kg/m^3**	**45 kg/m^3**
10	0.033	0.028	0.025	0.026

B.4.6 Phenolic foam

The thermal conductivity for various densities of phenolic foam is given in Table B.8.

Table B.8 **Thermal conductivity of phenolic foam**

Mean temperature °C	Thermal conductivity, W/mK, at densities of:			
	35 kg/m^3	**40 kg/m^3**	**60 kg/m^3**	**120 kg/m^3**
10	0.018	0.018	0.018	0.028
50	0.021	0.021	0.021	0.032
100	0.027	0.027	0.027	—

B.4.7 Polyisocyanate foam

The thermal conductivity for various densities of polyisocyanate foam is given in Table B.9.

Table B.9 **Thermal conductivity of polyisocyanate foam**

Mean temperature °C	Thermal conductivity, W/mK, at densities of:		
	32 kg/m^3	**40 kg/m^3**	**50 kg/m^3**
0	0.021	0.021	0.021
20	0.023	0.023	0.023
50	0.026	0.026	0.026

B.4.8 Rigid polyurethane foam

The thermal conductivity for various densities of rigid polyurethane foam is given in Table B.10.

Table B.10 **Thermal conductivity of rigid polyurethane foam**

Mean temperature °C	Thermal conductivity, W/mK, at densities of:		
	35 kg/m^3	**40 kg/m^3**	**50 kg/m^3**
0	0.021	0.021	0.021
10	0.023	0.023	0.023
50	0.026	0.026	0.026
100	0.032	0.032	0.032

The thermal conductivity is the sum of the heat flow for the various gaseous and solid components of the material as follows:

$$\text{Total heat transfer} = G + S + R + J \qquad \text{(B2)}$$

where:

G = heat transfer via conduction through the cell gas;

S = heat transfer via conduction through the solid phase;

R = heat transfer via radiation across the cells;

J = heat transfer via convection through the cell gas.

At a temperature of 10 °C these values are typically:

S = 0.004 to 0.006 W/mK;

R = 0.004 to 0.006 W/mK;

J = zero for cell diameters 0.2 mm to 1.0 mm.

The value of G varies depending on the blowing gas used in production. Typical values are given in Table B.11.

Table B.11 **Thermal conductivity through the cell gas for various blowing gases**

Material	**Thermal conductivity through the cell gas (*G*)** W/mK
CFC 11	0.008
HCFC 141b	0.009
Cyclo-pentane	0.011
Iso-pentane	0.013
N-pentane	0.014
Carbon dioxide	0.015

B.5 Density

The core materials used in composite sandwich panels can vary substantially in density according to the particular manufacturer and the product options available.

Typical densities available are given in Table B.12.

Table B.12 **Typical densities of core materials used in sandwich panels**

Material	**Density** kg/m^3
Mineral (rock) wool	80–140
Rigid polyurethane and polyisocyanurate foam	30–50
Polystyrene	16
Phenolic foam	20–35

Annex C (informative) Fire resistant load bearing structural solutions

C.1 General

A wide range of construction assemblies use steel in a manner that enables much greater levels of fire-resisting performance than those envisaged by basic evaluation.

C.2 Special forms of steel construction

C.2.1 General

The following subclauses provide basic information on the different design philosophies for special forms of steel construction.

C.2.2 Steel portal frames

Steel portal frames can be designed to maintain their stability under fire conditions for reasonable periods without additional protection to the rafter beams. The enhanced performance is achieved by detailing suitable fixing at the base of the portal columns to resist the overturning moments due to collapsing rafters (see Newman [58]).

C.2.3 Blocked-in columns

Placing concrete blocks between the flanges of universal columns can increase the fire resistance to at least 30 minutes. The blocks are not designed to be load bearing and are used solely to provide shielding to the inside flanges and web (see BRE Digest 317 [59]).

C.2.4 Shelf angle floor beams

Shelf angle floor beams using pre-cast slabs have been used for many years as a means of reducing construction depths. Since the slab shields the upper part of the main beam, this type of construction also provides enhanced fire resistance without the need for additional protection. Using slightly heavier angles and positioning these with the short leg upwards can achieve 30 min to 60 min fire resistance.

C.2.5 Water filled columns

Filling hollow section columns with water is a method of maintaining the temperature of the steel members at acceptable levels by removing heat from the system. There are two principle design approaches:

a) a replenishment system in which water lost through evaporation is replaced; and

b) a non-replenishment system in which water is permitted to evaporate but is not replaced.

Theoretically, a) can provide infinite fire resistance if the right balance between the circulation rates and heat transfer can be achieved. There are only a few buildings, principally in North America, that have used this type of fire protection, partly due to the difficulty in circulating water any way other than vertically and the cost of maintenance in anti-algae and anti-freeze treatments (see Bond [60]).

C.2.6 Columns in walls

Several types of systems have been evaluated on an individual basis and exceptionally high fire resistance periods can be achieved (see Kirby and Wainman [27]). However, the behaviour measured in these tests was specific to the construction detailed and careful consideration should be given if using the information outside the test parameters evaluated. For example, as the linear dimensions increase, thermal bowing effects are exaggerated due to the large temperature differential between the exposed and unexposed portions of the steel members.

C.2.7 Fabricated slim floor beams

A fabricated slim floor beam is formed by welding a plate (normally 15 mm thick) to the bottom flange of a universal column section to extend its width 100 mm beyond each of the flange tips. The outstands then support either pre-cast concrete hollow core units or deep deck composite slabs. The advantage of the system is that it reduces storey height and only part of the section is exposed to fire. Typical floor spans are in the range of 6 m to 9 m with structural depths between 250 mm to 450 mm. This type of floor construction can also be designed as non-composite. Design guidance is given by Mullett and Lawson [61].

C.2.8 Slim floor system using an asymmetric beam (ASB) section

The ASB is a specially rolled range of steel beams designed for use with deep steel decking in which the bottom flange is rolled wider than the top flange in order to support the floor slab. Fire resistance periods up to 60 minutes are possible. A key feature of the section is a thick web that is generally thicker than the flanges. This is particularly important in fire when the exposed bottom flange loses much of its strength. Typical spans are in the range 6 m to 9 m with total structural depths between 280 mm and 400 mm. Design guidance is given by Mullett and Lawson [62].

C.2.9 Rectangular hollow section (RHS) slim floor edge beams

This type of section is fabricated by welding a plate to a RHS section and is designed to form edge beams. They are often used around the perimeter of buildings designed with ASB or fabricated slim floor and offer a high level of torsional stiffness. This is also particularly helpful during the construction stage. Typical spans are in the range of 5 m to 7 m with structural depths between 380 mm and 400 mm. Design guidance is given by Mullett [63].

C.2.10 Web in-filled columns

By filling between the flanges of a universal column section with un-reinforced concrete, 60 minutes fire resistance can be achieved without additional protection. Although at ambient temperature the concrete is not intended to contribute to the normal strength of the column, it is effective at the fire limit state. To ensure composite action, shear connectors are shot fired along the web at 500 mm intervals. Web stiffeners should also be welded to the top of the column. Design guidance is given by Newman [64].

C.2.11 Active cooling

Active cooling/drenching systems can be employed to spray steelwork with water to maintain them at an appropriate temperature during a fire. Drencher systems need specialist design. Particular consideration should be given to their actuation, water distribution, water delivery rate, maintenance and their reliability of operation.

Annex D Methodology for establishing the extended application of fire resistance test results

D.1 General

There are a number of practical limitations on the size and design of elements that can be tested by the standard methods of test for fire resistance. When the elements are to be used at a different size (normally larger), receive different levels of restraint, or are of a modified design, there is a need to be able to confirm their performance, i.e. whether the classification(s) given in the classification report in relation to the relevant criteria identified in the Interpretative Document [2] are maintained. In a life safety strategy a designer should not assume that the classification granted to an element under the idealized conditions that the European tests provide, applies to the as-built construction with a completely prescriptive specification.

Such prescriptive guidance invariably incorporates "safety margins" in the performance requirements that take into account the probability that, in practice, the performance is not identical to that indicated by the classification. In any deviation from that prescription, no such assumption is valid.

Even in prescriptive guidance with classified elements, such as a small door assembly, which can be tested at the "in-use" size, and with representative levels of fixings and restraint, there are so many variations in hardware, sizes, apertures, frames and restraint levels that the economics of testing rule out the possibility of proving every case. A method is needed by which the classification given, based upon the test result achieved on a full sized assembly, could be maintained or extended to cover these manufacturing variations without resorting to additional tests.

This annex compliments the validated rules used in support of direct application.

For simple loadbearing elements, i.e. those constructed wholly from a single material, European material design codes can sometimes provide guidance on extending the application of test results.

This annex gives the methodology for making extended application statements/reports both generally or for specific elements. Methodology to be used for establishing the appropriate parameters, and factors that need to be taken into account when determining the extended application for the various elements are listed in the bibliography.

The annex does not cover the predication of performance as a result of the interaction of elements on site as that is the function of design guides, where they exist. Where design guides do not recognize the problem of such interactions, the principles given in this Published Document may be utilized by the approving authorities to determine whether the as-built construction continues to satisfy the classification given.

For the extended application standards covering the individual elements that are to be produced in conformity with this annex, the scope(s) only refers to the element under consideration. The scope of the individual standard should state:

"This standard is designed for use by "recognized" fire experts when preparing a report on the extended application of a specific construction that has been tested in accordance with BS EN (number of the standard for which the guidance standard has been prepared)".

NOTE Fire experts are normally those persons who can demonstrate adequate knowledge of the high temperature behaviour of materials or constructions, and who may normally be expected to be a "corporate" member of a relevant learned institution. Initially, member states need to recognize such persons, but there is a need for the Commission to recognize suitable fire experts who are adequately qualified to perform this service on a pan-European basis.

D.2 Principles of establishing the field of application

D.2.1 Types of field of application

Following a classification as a result of a test, there are two fields of application that need to be derived from the result. These are:

a) direct application;

b) extended application.

The rules governing the direct application are given in Clause **13** of the individual EN standards. There is no need for any special fire knowledge when applying these rules as the granting of the resulting increase in the field of application is automatic. In some cases, the field of direct application is dependent upon the result of the test, e.g. BS EN 1634-1. For any variation that is not listed in Clause **13**, an extended field of application analysis should be undertaken.

The extended application is an additional process that has to be applied for and is not granted automatically following a fire resistance test. An extended application analysis is needed when the application of the element differs from the construction that was tested and for which classification was achieved, and which is not covered by the field of direct application. The field of application report can take the form of a global report where all predicted variations are considered and the new limits on application are included. It can also address a change in a specific parameter, e.g. thickness of the element. An extended application can cover a number of forms as identified in **D.2.2**.

D.2.2 Variations to be considered when performing an extended application analysis

The common variations that are likely to be encountered are as follows.

a) Thermal and mechanical parameters.

These are the parameters that relate to the conditions the element is subjected to in the intended use, i.e. the exposure, which vary from those used in the test:

1) the load on loadbearing elements, both magnitude and distribution;

2) the boundary conditions applied to the element at its ends or edges;

3) the thermal action, i.e. BS EN 1363-2, or change in the number of faces exposed;

NOTE 1 There is no test within the European standards for a two or three sided exposed duct.

4) the pressure differential experienced by the construction due to its height;

5) the mechanical impact (if appropriate);

6) the orientation of an element with respect to the fire, e.g. a change from vertical to sloping.

b) Constructional parameters.

These are the parameters, which relate to changes in the construction of the element in its intended use, which vary from those of the tested construction:

1) changes in the construction method or the materials used in the construction of the element, not warranting a further test;

2) any change in the primary dimensions of the element (normally larger), from that tested to that under consideration;

NOTE 2 The use of a free edge(s) during the testing of separating elements infers unrestricted increases in the width of the element in use. An analysis of the "to be built" construction might indicate that this is not valid in all cases.

3) the introduction of, or any variations to, an aperture in a separating element;

4) the orientation of an asymmetric element tested in one direction only in respect of the fire exposure.

c) Fire resistance rating.

Any upward or downward changes in the fire resistance performance as a result of applying one or more of the possible changes resulting from a) or b), above.

An extended application analysis should consider each variation individually, as appropriate, but the analysis should then consider the effect of combining all of the relevant variations. In many situations, it might be necessary to consider the introduction of compensatory measures to change one or more parameters in order to permit the variation, e.g. an increase in the level of restraint [a2)], in order to compensate for a change in a dimension [b2)].

D.2.3 Establishing the influence of a variation in a parameter on the performance of the element

D.2.3.1 Thermal and mechanical parameters

The variation in performance that can arise as a result of a change in an exposure parameter can be significant, but the influence is not always obvious unless the person undertaking the analysis is experienced, especially if a change takes place in more than one parameter. When considering a change in one of the thermal or mechanical parameters it is necessary to consider all of the possible ways, i.e. factors, in which the parameter can vary.

If, for example, the load parameter varies then the following factors can apply to the load, and the relevant one should be considered in order to establish the influence that the variation might have on the performance:

a) the magnitude of the load might increase or decrease;

b) the distribution of the load might become more or less concentrated;

c) the mode of the stress generated by the variation in load might change, e.g. transfer from bending limiting to shear limiting;

d) the direction of the load might change.

Having identified the possible factors resulting from a change in the load, the next stage is to establish what the influence of a change in one of the factors would be on the fire resistance of the construction. Using the example of a change in the load parameter of a solid (non-composite homogeneous) element such as a concrete floor the influences could be as follows.

1) An increase in the load without a corresponding change in the cross-section of the structure carrying the load, would increase the stress in the element. As the increase in stress would mean that less of the section could be eroded or weakened before failure became evident, the fire resistance in respect of the loadbearing capacity (R) would be decreased, and integrity (E) can also decrease if the increased deflection were to result in cracking. Insulation (I) would probably not vary, assuming there was a margin of safety in the test.

2) A decrease in the load without a decrease in the cross-section of the loadbearing member(s) would result in lower stresses and probably an increase in the fire resistance with respect to both loadbearing capacity and integrity (R and E). Again, insulation (I) would probably not vary.

3) An increase in the concentration of load can only be resolved following a full analysis of the deflection resulting from re-distribution of the stresses in the element under consideration. The likely influence is an increase in the maximum deflection, hence a probable reduction in the fire resistance in terms of loadbearing capacity (R). If this change in deflection resulted in further cracking then the integrity (E) also reduces.

4) In extreme cases, this change in the concentration of the load could result in a change in the possible mode of the limiting stress, i.e. from bending to shear [see **D.2.3.2**e) below].

5) Conversely, a reduction in the load concentration is likely to result in lower deflection and maximum stresses and an enhanced fire resistance in terms of loadbearing capacity (R), and integrity (E) would be expected.

6) The influence on the fire resistance of the floor as a result of a change in the mode of the limiting stress load is generally only established by a full load analysis. This can be carried out by reference to the appropriate Eurocode.

7) A change in the direction of the load generally requires a re-calculation of the stress in the members. All criteria, "R, E and I", are likely to be influenced. In composite, loadbearing, separating constructions, e.g. a metal joisted floor or a timber stud wall, changes in the loading parameter which would influence the deflection would have a direct affect on the performance of the linings and any fixings. This is likely to have a more significant influence on integrity and insulation than with a homogeneous element.

D.2.3.2 Constructional parameters

The most important aspect when considering a variation in the construction of a tested and classified element is to establish the parameter(s) that are influenced by the proposed change. These are not necessarily obvious; therefore, the extended application analysis should be performed by an expert with knowledge of fire and materials in the hot state. **D.4** identifies the major parameters that should be addressed in standards produced to provide guidance for extending the field of application for individual elements but, because of the variation that can take place with elements of various constructional materials, they are not meant to be exhaustive. The analysis should then consider all possible factors that result from a change in the parameter(s) under consideration. For the thermal and mechanical parameters, it is easier to explain the principle of the analysis by example. For joisted or studded elements one of the parameters is the lining. The following factors may be varied in respect of the lining:

a) the lining may be as tested but with increased or reduced thickness;

b) the lining fixings may be increased or reduced in number;

c) the lining fixings may have enhanced or reduced resistance to pulling out/through when hot;

d) the boards forming the lining may be larger or smaller, resulting in more or less joints;

e) joints transverse to the studs/joists may be provided with enhanced or reduced support/sealing;

f) the lining may be of the same generic material as tested but with a higher or lower density;

g) the tested lining may be replaced with a lining of a different material.

The next stage is to establish what influence the relevant factor(s) might have on the fire resistance of the construction.

Considering, for example, an increase in the thickness of the lining, the influences that this factor can have include:

1) reduction in the temperature rise on the unexposed face;
2) reduction in the permeability of the assembly (depending upon the type of lining);
3) increase in the thermal inertia of the system;
4) increase in the stress on the lining fixings due to enhanced weight;
5) an enhanced temperature differential between hot and cold faces which can create additional bowing;
6) increase in the mass of the element and hence a change in the safety margins on the support/restraint conditions;
7) a change in the eccentricity of any applied load or self-induced load as a result of expansion which can result in increased bowing.

The next stage is to establish whether there are any rules or calculations available to substantiate the influence of the change and identify those influences that can be quantified. In the example above, it is possible that valid heat flow models might exist to quantify the temperature rise, both behind the lining under consideration and possibly on the unexposed face of the element.

However, models of the mechanical behaviour are be needed if list item a), above, is to be quantified. The influence of load on the fixings can be supported by an empirically derived rule depending upon whether the material is generic and whether research has been carried out to identify what the influence would be on the fixings, particularly if an improved fixing were to be used or the fixings were to be at closer centres. The influence of other factors in the list can only be established by means of expert judgment.

The factors to be considered when determining the influence of the variation should be listed in the guidance standard for the particular element, if one exists. Taking a) from the list of factors in respect of a change in the lining, the following factor influences should be considered: a change in the integrity (E) and loadbearing capacity (R), a failure in the shear resistance of the fixings between the proposed and tested conditions which produces a change in the tensile strength and/or pull out resistance of the fixings proposed compared with those used in test. This influences the contribution that the lining makes to integrity (E) and insulation (I). A change in the moment created between the centre line of the increased thickness lining and the centre line of the construction, influences the deflection of the element and, therefore, integrity (E).

As stated in **D.2.3.1** in conjunction with the thermal and mechanical parameters, this method of analysis can be carried out for any of the constructional factors that apply to the element under consideration. There may not be sufficient information available for the applicant to justify an extended application by such an analysis, in which case further testing is justified. In many cases, an *ad hoc* test, possibly even at reduced scale, might provide more information in support of the extended application process than a repeat of the classification test to the appropriate European norm. Such *ad hoc* tests benefit from the use of enhanced instrumentation.

D.3 Explanation of the expert analysis process

The following list summarizes the processes to be undertaken by a fire expert preparing the field of the extended application report on an element.

a) Identify all thermal and mechanical parameters that can vary as a result of the proposed application and/or use, if any.

b) Identify the components of a construction that can vary, either directly or as a result of the proposed changes.

c) Identify all constructional parameters that can change as a result of a change in that component.

d) Identify the factors that may change for each parameter.

e) For each factor, determine the factor influences on the relevant criteria by calculations, validated rules or expert judgment, as appropriate.

The specific extended application standard for the element under consideration should be referred to, if applicable, in order to identify the relevant parameters.

The extended application standard might suggest the primary factors, but because each construction differs it is virtually impossible for all factors to be pre-identified.

The obvious influences should be listed, but it is unlikely that every influence can be identified in advance of the analysis being started and, in this case, the process might need to be repeated as other influences are recognized.

For each influence listed it is necessary to consider whether there is evidence available to quantify the variation by means of secondary test evidence, historic data, *ad hoc* tests (at full or reduced scale), calculation or whether a qualitative analysis should be made by means of an expert judgment. As with most applications, the output should be the product of at least two experts with the necessary fire and high temperature material response knowledge. The reasoning process should be incorporated into the extended application report in a transparent manner.

The extended application standards (under preparation) should not only provide guidance as to whether a rule, calculation or expert judgment is appropriate for establishing the influence of the variation, but also recommend the appropriate calculation or source of any rule and their limits on applicability.

Expert analysis of the influence on the result should be performed:

1) initially on a factor by factor basis; and then

2) on a global basis where the interaction between the influence of factors needs to be taken into account, i.e. do they complement or contradict each other?

It is difficult to give guidance on the interaction between factors in the individual extended application standards under preparation, but the interaction should be considered to give some idea as to which factors could cancel each other out and which are additive. When performing the analysis in the absence of any such report, consideration of the relative influence of each variation in a multiple factor application should be carried out using expert judgment, as described above.

Once the field of extended application has been established in accordance with the standards (under preparation), if applicable, or from first principles, an extended application report should be prepared.

D.4 Contents of the extended application report

The extended application report should be used in conjunction with the test report as it affects the classification achieved. The report presenting the findings of a field of extended application analysis should contain:

a) name of the sponsor;

b) the type of element being subjected to analysis, including a general description of the element, e.g. a floor carrying a UDL of "*x*" kN/m^2.

c) a complete characterization of the assembly tested, including any trade names of the products involved;

d) description of any variations not conforming to the tested and classified construction, incorporating a clear statement of the proposed variations considered in this document, including previously analyzed changes;

e) summary of fire testing evidence upon which the analysis is to be made;

   NOTE This is a specially prepared synopsis of the relevant test evidence identifying in detail the performance of components relevant to the analysis and not necessarily the brief summary sometimes given as part of the report. Alternatively, it is acceptable to append full copies of the relevant documents.

f) identification of the relevant parameter(s) and the list of the factors to be considered in the analysis;

g) the relevance of each parameter can be stated using a box system with a tick or a cross for each of the relevant factors);

h) for each relevant factor, the influence of the proposed change on the fire resistance of the element is either:

   1) favourable;

   2) unfavourable; or

   3) no influence;

For each influence the report should give the justification behind the above conclusion, especially where expert judgement has been used.

Identification of the influence on performance resulting from interactions between elements has an influence on the relevant criteria and the report should state the revised field of extended application resulting from this.

The fire resistance rating and the field of extended application of the varied construction should be expressed in the report without ambiguity.

Bibliography

For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Standards publications

BS 15, *Structural steel for bridge and general building construction*

BS 476-23:1987, *Fire tests on building materials and structures – Part 23: Methods for determination of the contribution of components to the fire resistance of a structure*

BS 476-24, ISO 6944, *Fire tests on building materials and structures – Part 24: Method for determination of the fire resistance of ventilation ducts*

BS 476-33, *Fire tests on building materials and structures – Part 33: Full-scale room test for surface products*

BS 6336:1998, *Guide to the development of fire tests, the presentation of test data and the role of tests in hazard assessment*

BS 8214, *Code of practice for fire door assemblies*

BS 9999:2008, *Code of practice for fire safety in the design, management and use of buildings*

BS EN 1634-1, *Fire resistance and smoke control tests for door, shutter and, openable window assemblies and elements of building hardware – Part 1: Fire resistance tests for doors, shutters and openable windows*

BS EN 1993-1-3:2006, *Eurocode 3 – Design of steel structures – Part 1: General rules – Supplementary rules for cold-formed members and sheeting*

BS EN 10218, *Steel wire and wire products – General*

BS EN 12602, *Prefabricated reinforced components of autoclaved aerated concrete*

BS EN 13381-8, *Test methods for determining the contribution to the fire resistance of structural members – Part 8: Applied reactive protection to steel members*

BS EN 13501-1, *Fire classification of construction products and building elements – Part 1: Classification using data from reaction to fire tests*

prEN 10138, *Prestressing steels*

prEN 10138-2, *Prestressing steels – Part 2: Wire*

prEN 10138-3, *Prestressing steels – Part 3: Strand*

prEN 10138-4, *Prestressing steels –Part 4: Bars*

DD ENV 1991-2-2:1996, *Eurocode 1 – Basis of design and actions on structures – Part 2.2: Actions on structures exposed to fire (together with United Kingdom National Application Document)*

DD ENV 13381-4, *Test methods for determining the contribution to the fire resistance of structural members – Part 4: Applied protection to steel members*

ISO 13784-2, *Reaction-to-fire tests for sandwich panel building systems – Part 2: Test method for large rooms*

ISO/TR 834-2, *Fire resistance tests – Elements of building construction – Part 2: Guidance on measuring uniformity of furnace exposure on test samples*

ISO/TR 834-3, *Fire resistance tests – Elements of building construction – Part 3: Commentary on test method and test data application*

ISO/TR 22898, *Review of outputs for fire containment tests for buildings in the context of fire safety engineering*

DIN 18230-1, *Structural fire protection in industrial buildings– Part 1: Analytically required fire resistance time*

Other documents

[1] GREAT BRITAIN. The Building Regulations (England and Wales) Approved Document B, *Fire Safety*, Vols. 2 and 11, London: The Stationery Office, 2006.

[2] EUROPEAN COMMISSION. Interpretative Document 2 – *Safety in case of fire.* European Commission – Council Directive 89/106/EEC of 21 December 1988. Available from: http://ec.europa.eu/enterprise/sectors/construction/documents/legislation/interpretative-documents/doc2_en.htm

[3] CORUS GROUP. *The behaviour of a Multi-storey Steel Framed Building* – A European Joint Research Programme, Corus Group, Swinden Technology Centre, Rotherham, 1999.

[4] SULLIVAN, P.J.E., TERRO, M.J., and MORRIS, W.A. *Critical review of fire dedicated thermal and structural computer programs*, Applied Fire Science, Vol. 3,113-135, 1993.

[5] READ, R.E.H. *External fire spread: building separation and boundary distances*. BRE Report 187, BREPress, 1991.

[6] LAWSON, D.I., and SIMMS, D.L. *The ignition of wood by radiation*. British Journal of applied Physics, Vol. 3, 288-292, 1952.

[7] KANURAY, A.M. *Ignition of cellulosic materials – a review*. Fire Research Abstracts and Reviews, Vol.14, 24-52, 1972.

[8] PETTERSEN, O., MAGNUSSON, S.E., and THOR, J. *Fire engineering design of steel structures*. Swedish Institute of Steel Construction, 1976.

[9] INGBERG, S. H. *Tests of the severity of building fires*. National Fire Protection Association, Quarterly 22, (43-61) 1928.

[10] SCOTLAND. The Building Standards (Scotland) Amendment Regulations 2001, Scottish Office. London: The Stationery Office.

[11] GREAT BRITAIN. The Building (Amendment) Regulations (Northern Ireland) 2005, London: The Stationery Office.

[12] LAW, M. *A relationship between fire grading and building design and contents.* Fire Research Station, Note No. 877, Borehamwood, England, 1971.

[13] HARMATHY, T.Z. *On the equivalent fire exposure.* Fire and materials. Vol.11, No. 2, pp. 95-104, 1987.

[14] PETTERSEN, O. *The connection between a real fire exposure and the heating conditions according to standard fire resistance tests*. European Convention for Constructional Steelwork, 1974, Chapter II, CECM-III-74-2E. Elsevier B.V.

[15] FIRE SAFETY JOURNAL. CIB W14 *Workshop report, Design Guide: Structural fire safety*, Fire Safety Journal, Vol. 10, No.1, 1986.

[16] KIRBY, B.R., WAINMAN, D.E., TOMLINSON, L.N., KAY, T.R., and PEACOCK, B.N. *Natural Fires in Large Scale Compartments*. British Steel Technical/Fire Research Station, British Steel Technical, 1994.

[17] KIRBY, B.R., and TOMLINSON, L.N. *Development of the UK and European design codes,* Corus Group, Swinden Technology Centre, Final Report to the Department of the Environment, Transport and Regions (DETR), September 2000.

[18] LAW, M. and O'BRIEN, T. *Fire safety of bare external structural steel*. The Steel Construction Institute, Series 009, 1989.

[19] LENNON, T. Natural fire tests on the multi-storey concrete framed building at Cardington, Building Research Establishment, 1997.

[20] NATIONAL FIRE PROTECTION ASSOCIATION. *Standard for smoke management systems in malls, atria, and large spaces*. NFPA 92B, 2009 Edition.

[21] DAVEY, N. and ASHTON, L.A. *Investigations on building fires – Part V – Fire tests on structural elements*. National Building Studies Research Paper No. 12, Department of Scientific and Industrial Research, HMSO, London: The Stationery Office, 1953.

[22] CLAYTON, N. and LENNON, T. *Effect of polypropylene fibres on performance in fire of high grade concrete.* BRE report 395, BRE Bookshop, 2000.

[23] LENNON, T. *Natural fire tests on pre-cast hollow core slabs in fire*. Building Research Establishment.

[24] COMITÉ EURO-INTERNATIONAL DU BÉTON (CEB). *Fire design of concrete structures in accordance with CEB-FIP Model Code 90*. Bulletin d'Information No. 208. Lausanne, 1991.

[25] GREAT BRITAIN. *Investigation on building fires, Part 1 Estimation of maximum temperatures*, National Building Studies Technical paper No. 4, London: HMSO, 1950.

[26] WICKSTRÖM, U. *A very simple method for estimating temperature in fire exposed concrete structures.* Fire Technology Technical Report SP-RAPP 1986. 46, Swedish National Testing Institute, 1986.

[27] KIRBY, B.R. and WAINMAN D.E. *Compendium of UK standard fire test data – unprotected structural steel*. British Steel Technical, Swinden Laboratories. Rotherham. Vol. 2, 1989.

[28] WAINMAN D.E, KIRBY, B.R., TOMLINSON, L.N., KAY, T.R. and PRESTON, R.R. *The behaviour of unprotected steel floor beams in the standard fire resistance test – compendium of predicted temperature profiles*. British Steel Technical, Swinden Laboratories. Rotherham 1990.

[29] LATHAM, D.J., KIRBY, B.R. and THOMSON, G. *The temperature attained by unprotected steel in compartment fires.* Fire Safety Journal 12, 139-152, 1987.

[30] ASSOCIATION FOR SPECIALIST FIRE PROTECTION. *Fire protection of structural steel in buildings*. Volume 1, 4th Edition, ASFP, Association House, 99 West Street, Farnham, Surrey GU9 7EN, 2009.

[31] CORUS UK LTD. *Advance sections – CE marked structural sections*, Corus Construction & Industrial product manual, Corus UK Ltd, 2007.

[32] STEEL CONSTRUCTION INSTITUTE. *Fire protection of beams with web openings*, Technical Report RT 1187, Steel Construction Institute, 2008.

[33] HARTL, H. *Fire resistance of joints*. In Blass, H.J. et al. Timber engineering STEP 1. Centrum Hout, Almere, The Netherlands, Düsseldorf, 1995.

[34] HALL, G.S., SAUNDERS, R.G., ALLCORN, R.T., JACKMAN, P.E., HICKLEY, M.W. and FITT, R. *Fire performance of timber – a literature survey*. Timber Research and Development Association, High Wycombe, England, 1972.

[35] READ, R.E.H., MORRIS, W.A. and COOKE, G.M.E. *Guidelines for the construction of fire-resisting structural elements*. BRE Report 128, BREPress, 1988.

[36] BURGAN, B. *Development on the use of stainless steel in construction*. Final report by The Steel Construction Institute. ECSE Contract No. 7210-SA/842, 2000.

[37] ASTM E119 – 09c, *Standard test methods for fire tests of building construction and materials*.

[38] INTUMESCENT FIRE SEALS ASSOCIATION. *The IFSA Code – Sealing apertures and service penetrations to maintain fire resistance.* Intumescent Fire Seals Association (IFSA), 1999.

[39] ASSOCIATION FOR SPECIALIST FIRE PROTECTION. *Fire stopping and penetration seals for the construction industry*. ASFP, Association House, 99 West Street, Farnham, Surrey GU9 7EN. ISBN 1 870409 07 8, 2000.

[40] *LPC design guide for the fire protection of buildings*. Loss Prevention Council. 2000.

[41] ASSOCIATION FOR SPECIALIST FIRE PROTECTION. *Fire rated non-loadbearing partitions*. ASFP, Association House, 99 West Street, Farnham, Surrey GU9 7EN, 2003.

[42] SCHAFFER, E.L. *Charring rate of selected woods – transverse to grain*, FPL 69, US Department of Agriculture, Forest Service, Wisconsin, 1967.

[43] INTERNATIONAL ASSOCIATION FOR COLD STORAGE CONSTRUCTION. *Guidelines for the design, construction, maintenance, specification and fire management of insulated envelopes for temperature controlled environments*. International Association for Cold Storage Construction (IACSC), 1999.

[44] PURKISS, J.A. *Fire safety engineering design of structures*. Second Edition, Butterworth Heinemann, 2007.

[45] INSTITUTE OF STRUCTURAL ENGINEERS. *Guide to the advanced fire safety engineering of structures*, Institute of Structural Engineers, publication no. 433, 2007. Available from: http://www.mace.manchester.ac.uk/project/research/structures/strucfire/

[46] CONNOLLY, R.J. *Spalling of concrete.* Fire Engineers Journal, Vol. 57, No. 186. Institution of Fire Engineers, January 1997.

[47] STEEL CONSTRUCTION INSTITUTE. *Appraisal of existing iron and steel structures*, Steel Construction Institute, Report No. 128, 1997.

[48] COOKE, G.M.E. *Fire engineering of tall separating fire walls (Parts 1 and 2).* Fire Surveyor. Vol. 16, 1987.

[49] STEEL CONSTRUCTION INSTITUTE. *Fire Safe Design – A new approach to multi-storey steel framed buildings*, Steel Construction Institute, 2000.

[50] KODUR, V. K. R. and SULTAN, M. A. *Effect of temperature on the thermal properties of high-strength concrete,* Journal of Materials in Civil Engineering, Vol. 15, No.2, 2003.

[51] KIRBY, B.R. and PRESTON, R.R. High temperature properties of hot rolled structural steel for use in fire engineering design studies. *Fire Safety Journal*, 1998, Vol. 13, 27-37.

[52] LAWSON, M.L. and NEWMAN, G.M. *Structural fire design to EC3 and EC4 and comparison with BS 5950*. Steel Construction Institute, Report 159, 1996.

[53] KIRBY, B.R. The application of BS 5950-8 on fire limit state design to the performance of "old" structural mild steels. *Fire Safety Journal*. 1993, Vol. 20, 353-376.

[54] KIRBY, B.R, LAPWOOD, D.G. and THOMSON, G. *The reinstatement of fire damaged steel and iron framed structure*s. British Steel Corporation, Swinden Laboratories, 1993.

[55] SIDEY, M.P., and TEAGUE, D.B. *Elevated temperature data for structural grades of galvanised steel*. British Steel, Welsh Laboratories. Report No. WL/PM/TEN/251/88/D, January 1988.

[56] KIRBY, B.R. *The behaviour of grade 8.8 bolts in fire*. Journal of Constructional Steel Research, Vol. 3, 1995.

[57] LATHAM, D.J., and KIRBY, B.R. *Elevated temperature behaviour of welded joints in structural steel.* DISC Final Report, EUR 17855, 1998.

[58] NEWMAN, G.M., *Fire and Steel Construction: The behaviour of steel portal frames in boundary conditions* (2nd Edition). Steel Construction Institute, 1990.

[59] BUILDING RESEARCH ESTABLISHMENT. *Fire resistant steel structures: free standing blockwork-filled column and stanchions*. BRE Digest 317, Building Research Establishment, 1986.

[60] BOND, G.V.L. *Fire and steel construction – water cooled hollow columns*. Constrado, 1975, (available from the Steel Construction Institute).

[61] MULLETT, D.L. and LAWSON R.M. *Design of Slimflor fabricated beams using deep composite decking*. The Steel Construction Institute, Report SCI-P-248, 1999.

[62] MULLETT, D.L. and LAWSON R.M. *Design of asymmetric Slimflor beams using deep composite decking*. The Steel Construction Institute, Report SCI-P-175, 1997.

[63] MULLETT, D.L. *Design of RHS Slim floor edge beams.* The Steel Construction Institute, Report SCI-P-169, 1997.

[64] NEWMAN, G.M., *The fire resistance of web in-filled steel columns*. The Steel Construction Institute, Report SCI-P-124, 1992.

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