PD 7974-3:2019

BSI Standards Publication

Application of fire safety engineering principles to the design of buildings

Part 3: Structural response to fire and fire spread beyond the enclosure of origin (Sub-system 3)

bsi.

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Contents

Summary of pages

This document comprises a front cover, and inside front cover, pages i to vi, pages 1 to 118, an inside back cover and a back cover.

Foreword

Publishing information

This Published Document is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 31 March 2019. 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This Published Document supersedes PD 7474-3:2011, which is withdrawn.

Relationship with other publications

This Published Document is one of a series of documents published under the Fire Standards Policy Committee, and is a supporting document to BS 7974, *Application of fire safety engineering principles to the design of buildings — Code of practice*.

Other documents in the series are:

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*.

Where appropriate, references to relevant standards are provided in order to assist the reader in understanding the design methodologies presented and to compare 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 standards, particularly in relation to additional notes and subclauses describing its application.

Information about this document

This is a full revision of the standard, and introduces the following principal changes:

- the content has been updated to include the recommendations in the latest standards and guidance documents;
- design fires for structural fire engineering have now been integrated with PD 7974-1 and a new section on design fire selection process has been introduced. Additionally, the inherent assumptions and limitations of adopting these design fires have been explicitly stated;
- the information on potential mechanisms for fire spread has been expanded;
- the potential use of risk-based concepts described in PD 7974-7 has been added for the purposes of PD 7974-3;
- information such as material properties that was available in other British Standards has been removed to avoid repetition and reduce the length of the document;
- the layout of the document has been re-arranged so that self-contained technical information is provided as an annex.

This and the other Published Documents (PDs) in this series contain 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 need to 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 and competent 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.

Use of this document

As a guide, this part of PD 7974 takes the form of guidance and recommendations. It should not be quoted as if it were a specification or a code of practice and claims of compliance cannot be made to it.

This publication is not to be regarded as a British Standard.

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.

Where words have alternative spellings, the preferred spelling of the Shorter Oxford English Dictionary is used (e.g. "organization" rather than "organisation").

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.

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. This Published Document considers the following issues:

a) the conditions that lead to fire spread beyond the enclosure of fire origin (see also Annex A);

b) the selection of design fires depending on the objectives of the assessment (see also Annex B);

c) the thermal and mechanical response of the enclosure boundaries and its structure to the fire conditions (see also Annex C and Annex D);

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

e) 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 (see also Annex E and Annex F).

Annex G provides a methodology for establishing the extended application of fire resistance test results.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes provisions of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS EN ISO 13943, *Fire safety — Vocabulary*

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 structural failure has occurred or the time-equivalent period has been 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 can 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 and competent 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 structure in a given period

3.15 localized fire

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

3.16 place of relative safety

predetermined place in which persons are in no immediate danger from the effects of fire

NOTE This may be inside or outside the building depending upon the evacuation strategy.

3.17 probability

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

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

3.18 risk

frequency × probability × consequence

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 rate of change of output as a function of rate of change of an input parameter of input

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 in standardized fire that would result in an equivalent structural response to that of the design fire in question

4 Symbols

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 Boltzmann 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 conductivity
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)

$d_{\mathrm{char,0}}$ charring depth for one dimensional charring (mm)

d_{door} thickness of a door leaf (mm)

d_{ef} 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 integrity criteria

$E_{\mathrm{d,t}}$ design load created by the fire situation at time t

$E_{\mathrm{fi,d}}$ design effect of actions for the fire situation

E_{m} Young's modulus (kN/mm^2)

E_{p} plastic modulus (kN/mm^2)

$E_{\mathrm{p},\theta}$ 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)

$F_{\mathrm{e\text{-}R}}$ 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 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	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_w	height above the top of the opening (m)
Z_y	elastic modulus about the minor axis (cm^4)

5 Design approach to PD 7974-3

5.1 General

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

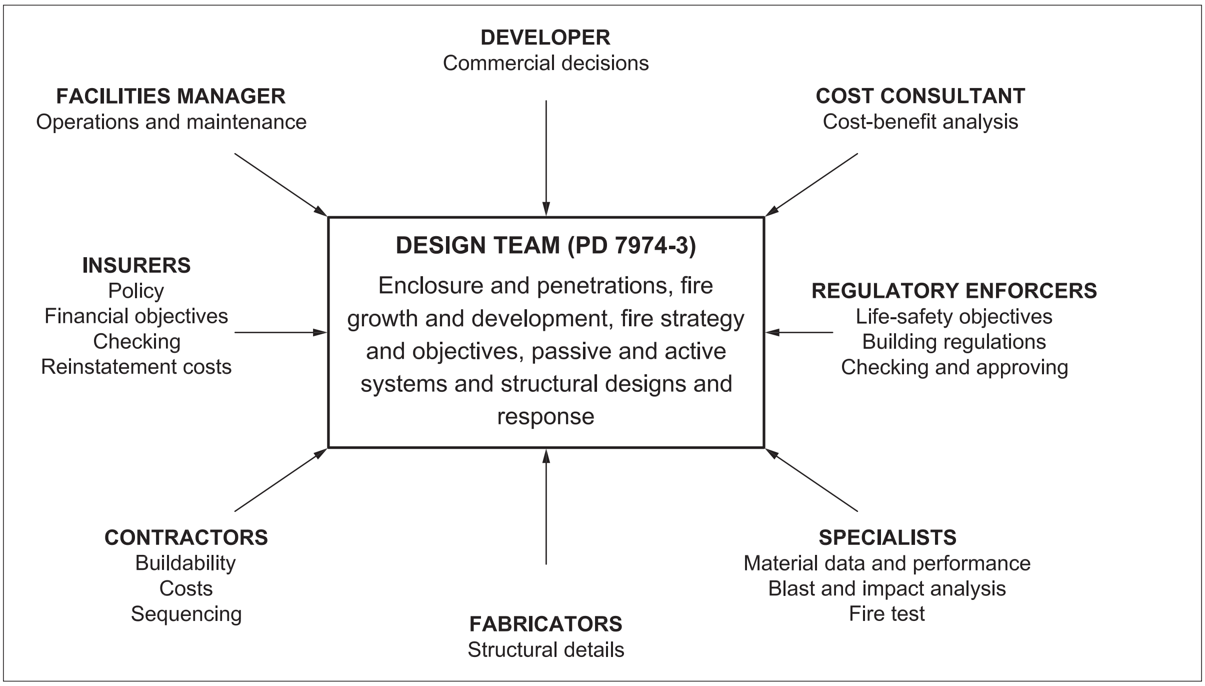
BS 7974 assists with the design process. The quantitative design analysis is divided into a number of separate sub-systems. Each sub-system can be used in isolation when analysing a particular aspect of design, or can be used in combination as part of an overall fire evaluation of a building. The parameters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

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.

5.2 Interaction with BS 7974 framework

The BS 7974 framework is shown in BS 7974:2019, Figure 1. The preliminary part of the framework is to conduct a qualitative design review (QDR) comprising several components, defined in BS 7974, during which the relevant sub-systems should be considered. The parties that might be involved in PD 7974-3 are listed in Figure 1. 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 1 — *Interaction between the various professions and the design team in addressing PD 7974-3 factors*



5.3 Functional objectives

The functional objectives for the assessment should be defined (see BS 7974).

5.4 Identification of fire hazards and possible consequences

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, all potential mechanisms of fire spread and potential routes for fire spread that would prevent the functional objectives being met should be identified. See Annex A for further information on fire spread mechanisms.

5.5 Identification of acceptance criteria and appropriate methods of analysis

5.5.1 General

The acceptance criteria, as defined in the QDR, should be applicable to the fire safety objective. The basis of any assessment or sub-assessment can be empirical or theoretical, the accuracy can be approximate or realistic, the analysis can be deterministic or risk-based and the measure can be qualitative or quantitative.

Regardless of what combination is adopted, the acceptance criteria should be compatible with the functional objectives, hazards and consequences, trial fire safety design and analysis method.

The acceptance criteria may 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.

5.5.2 Factors influencing acceptance criteria

Mechanisms of failure for loadbearing elements can result from material degradation, loss of material, thermally induced stresses and strains, delamination or debonding of composite materials, connection failure, etc. The acceptance criteria should be selected to adequately protect against the relevant mechanisms of failure and might include:

a) minimum permissible strength and/or stiffness;

b) minimum permissible loss of material;

c) maximum allowable stress or strain or deflection;

d) maximum allowable rate of change of stress or strain or deflection;

e) maximum allowable material temperature;

f) maximum allowable rate of change of material temperature; and

g) maximum allowable increase of axial forces.

The following factors should 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 compartments other than the compartment of fire origin;

  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 places of relative safety.

  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, for common situations it can be assumed that there is an adequate level of redundancy in the solution derived if they are used.

5.6 Establishing trial fire safety designs

5.6.1 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 (within the context of this Published Document). Typically, 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, smoke control;
- fire service intervention and effectiveness; and
- reliance upon future management.

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

5.6.2 Redundancy

Redundancy or diversity ensures that an alternative is available if a particular feature or system becomes compromised. Examples of redundancy include:

- alternative structural load paths; or
- multiple means of protection (e.g. passive fire protection and sprinklers).

5.6.3 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.

5.6.4 Reliability

Reliable systems are those likely to perform as required. Reliability can be improved through the use of reliable components, and these are subjected to appropriate maintenance and testing.

5.7 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.

a) 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 as a localized or travelling fire can also lead to compartmentation failure.

b) 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.

c) It is often assumed that all elements being heated simultaneously represents the worst case, which is not necessarily true. A travelling fire can induce different structural responses to a uniform fire and can be more severe.

d) 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 shorter and hotter than 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.

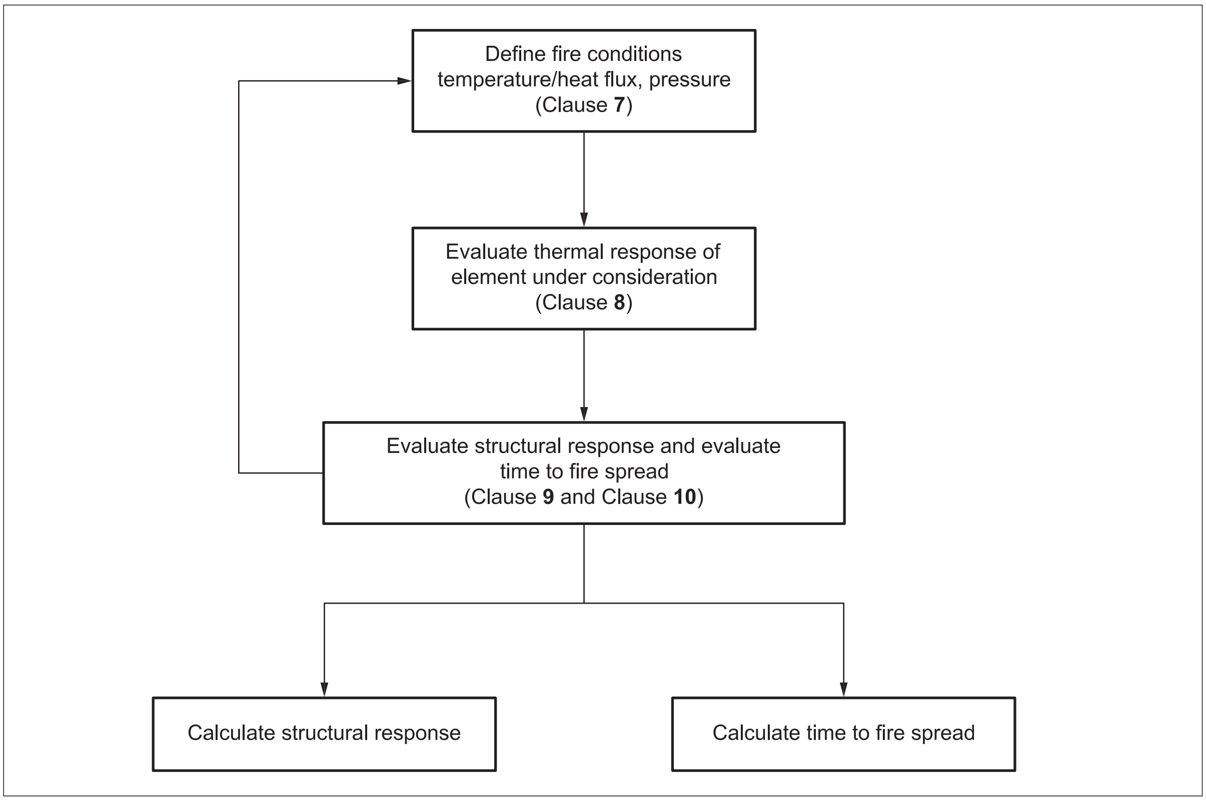
5.8 Analysis

5.8.1 General

The primary focus of each of the BS 7974 sub-systems is the 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 analysis that should be conducted as part of PD 7974-3 (within the limitations described in Clause 7) is defined in Figure 2. Each step can be conducted sequentially or combinations of the steps can be conducted simultaneously.

Figure 2 — *Procedure for analysis within PD 7974-3*



5.8.2 Defining analysis methods

The science behind each of the parts of the 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.

The analysis method should be compatible with the functional objectives, mechanisms of fire spread and the acceptance criteria.

5.8.3 Defining fire conditions

PD 7974-1 provides information on defining fire conditions depending on the compartment characteristics.

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. Turbulence in the fire could make the results of some fire tests inappropriate if the construction materials are not robust enough to withstand this. When defining the fire conditions, the designer should consider the temperature, pressure conditions and turbulence.

5.8.4 Heat transfer and thermal response

The fire conditions should be used to evaluate the heat transferred into and throughout the elements of the structure and/or enclosure boundaries.

5.8.5 Structural response

5.8.5.1 General

The structural response assessment should consider material degradation (e.g. strength and stiffness), thermal expansion and structural boundary conditions (e.g. connections surrounding structure).

5.8.5.2 Changes in building characteristics

Analysis undertaken in accordance with PD 7974-3 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.

If the structural response analysis identifies changes in building characteristics, these should be assessed to determine whether the design fire characteristics need to be revisited.

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6 Analysis methods

6.1 General

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). There are many forms of analysis that can be used to evaluate the thermal and/or structural response to fire. The selected method should be compatible with the functional objectives, hazards and consequences, trial fire safety design and acceptance criteria.

6.2 Basis of analysis

The basis of analysis can be empirical, theoretical or semi-empirical.

Empirical methods include standard fire tests, experiments and experiential data where real fire behaviour, heat transfer and/or structural response are estimated by comparison with empirical data. Fire, heat transfer and structural response can be complex, and in certain circumstances, empirical methods have to be used because theoretical methods do not exist. Similarly, a key advantage of empirical methods is the associated physical verification. However, care should be taken to ensure that the empirical method is adequately conservative and representative of the real fire, heat transfer and structural response such that comparisons of behaviour are valid.

Theoretical methods include analytical calculations and numerical simulations. They might comprise simple approximations to a complex series of sub-models. The fire, heat transfer and structural response might be considered separately or coupled. In certain circumstances, theoretical methods have to be used because real enclosures and structures are often too large or complex to be represented adequately by empirical methods. Typically, theoretical models are sufficiently quick and cost-effective to allow multiple assessments and consideration of the impact of variables or uncertainty. As with empirical methods, care should be taken to ensure that the theoretical model is adequately conservative and representative of the real fire, heat transfer and structural response such that prediction of behaviour is valid. Furthermore, it should be ensured that the theoretical models and sub-models are adequately verified and validated for the intended use.

Appropriate fire and structural models should be selected for the scenario under consideration. For example:

a) 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.

b) Standard furnace tests might not be suitable for exposed structural timber whereby the fire curve is influenced by the timber itself being a fuel.

The designer should select an analysis method and level of sophistication to assess the trial fire safety design against the functional objectives.

6.3 Accuracy

The analysis can range from a simple approximation to an accurate representation of reality.

The accuracy should be sufficient for the analysis in question and should be compatible with the functional objectives. For example, in certain circumstances, the overall structural response can be sufficiently accurately represented by the isolated response of its constituent structural elements; whereas, in other circumstances, the overall structural response can only be sufficiently accurately represented by the assembly of all or multiple constituent structural elements.

6.4 Means

Deterministic assessments have non-probabilistic inputs and variables and give the same answer for any given set of input. Inevitably fire, heat transfer and structural response involve probabilistic data and variables. Therefore, for deterministic assessments to be valid, it is typically necessary to ensure adequate conservative inputs and variables and/or adequate margins/factors of safety against the acceptance criteria.

Risk-based assessments include at least one probabilistic input or variable.

6.5 Measures

Measures of assessment are typically described as being qualitative or quantitative. In practice, it is not a binary situation and the measure can range from little quantification to fully quantified justifications. The amount of quantification should be commensurate with the reliance being placed on it.

Assessments can be qualified through interpretation of data and evidence, expert judgement and/or logic and reasoning.

7 Evaluation of fire conditions

7.1 Design fire characterization

The characterization of design fires and the associated thermal load to the structure is discussed in PD 7974-1. More information on design fires is given in Annex B.

The characterization requires definition of temperature (or heat flux) and potentially pressure, both spatially and with respect to time, and can range from homogeneous, steady state to spatially and time-dependent heating and cooling. Fire characteristics can be derived from a range of sources including expert judgement, experimentation, standardized fires, simplified temperature–time curves, and advanced modelling (CFD, zone, etc.). For the use of time equivalence, see Clause **10**.

7.2 Selection of design fires

7.2.1 General

Structural response can be sensitive to both spatial and time-based variations in a heating regime [3], [4], [5]. For example:

a) Non-homogeneous heating can result in greater thermally induced stress and/or strain than homogeneous heating and consequently higher restrained thermal expansion.

b) Homogeneous heating can result in a higher proportion of structure at reduced strength than non-homogeneous heating.

c) Rapid heating can induce a higher thermal gradient in partially exposed elements than steady state or gradual heating.

d) Cooling can result in restrained thermal contraction.

The designer should select the design fires for a structural response assessment such that the temperature/heat flux/pressure conditions as relevant with respect to space and time are sufficiently accurate or onerous to test the structural response (or reliability where a risk-based assessment is undertaken) against the acceptance criteria and functional objectives.

7.2.2 Applicability and limitations

In some instances (for example, determining the limiting temperature of simple beams or columns, benchmarking trial safety designs that are not susceptible to non-homogeneous heating or rate of heating, or where heating is likely to be steady state and homogeneous), steady state, homogeneous fire conditions might be sufficient. Additionally, the value of temperature or heat flux used, particularly in comparative studies, might not be critical. In other situations, detailed spatial variations with respect to time might be necessary.

Standardized fires were typically developed as a means of benchmarking structural response to a sufficiently onerous representation of reality. They can be very useful for testing structural response and potential failure mechanisms (particularly for comparative assessments of different trial fire safety designs), but they are unlikely to be applicable where an accurate assessment of structural response is required or for situations where structural response is likely to be significantly impacted by restrained thermal expansion or contraction or rapid or differential heating. This would also apply in comparative studies.

Homogeneous, time-based heating conditions, e.g. parametric fires, can be used as a more accurate characterization of reality and/or to test the impact of rate of heating and/or cooling. Both might require more than one design fire. As with standardized fires, they are only applicable when fire conditions are sufficiently homogeneous, homogeneous fire conditions are conservative in respect of structural response or the structural response is not sensitive to differential spatial heating.

Travelling fires allow for fire conditions to be characterized spatially with respect to time. Whilst this might be the most realistic and/or onerous characterization of a fire, there is an increased number of variables: shape and dimensions; rate; direction; path of travel; and temperature or heat flux in three dimensions. Therefore, it might be necessary to consider multiple fire scenarios.

7.2.3 Changing characteristics

The initial characterization of fires might have to be changed iteratively through the analysis process. For example, where openings are provided between compartment floors (atriums, open stairs, etc.) and no passive/active measures apart from suppression systems are provided to restrict fire spread, and it is assumed that suppression systems fail, a multi-storey or vertically travelling fire might need to be considered. Similarly, when calculating the fire severity within a compartment, if that fire severity is higher than the fire resistance of the surrounding construction, it might have to be assumed that the surrounding construction fails and the domain of analysis (both for design fires and structural response) changes.

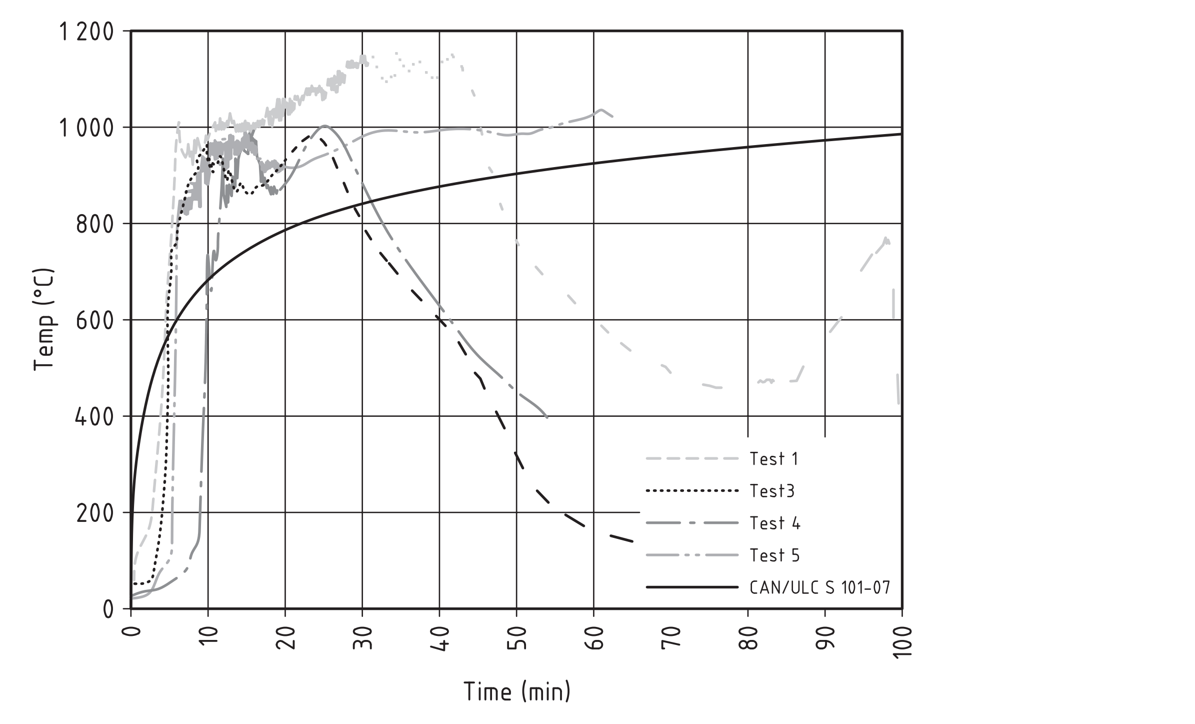
7.2.4 Combustible enclosures

Combustible materials, such as timber or bamboo, are sometimes used over more traditional construction materials, such as steel or concrete. When combustible structural members are exposed to a fire (i.e. not protected with fire-rated construction for the whole duration of the expected fire), they can alter the fire dynamics leading to increased fire durations and increased external flaming from the openings [6]. See Figure 3. This is due to the increase in fuel load, the potential for secondary flashover (in the event that the structure does not experience self-extinguishment, where relevant) and the reduction in the geometry of the structural members as a result of the consumption of the combustible material by the fire [7].

Combustible elements can possess fire resistance even when they are left exposed. However, traditional methods of specifying fire resistance by separating the design fire selection process and the subsequent heat transfer and mechanical response assessment (similarly to the principles described in this Published Document) might not always be appropriate for combustible buildings.

Additionally, the structural performance of timber buildings is not based on a maximum temperature, but rather on a temperature profile and the charring (of the wood) which can continue to affect the capacity of the material even during the decay stage of a fire. Delamination needs to be prevented for laminated members such as CLT through appropriate use of fire-resting adhesives [6].

Figure 3 — *Gas temperature in non-combustible and combustible compartments [8]*



8 Evaluation of thermal response

8.1 Thermal response of elements within enclosure

The thermal response of any construction element can influence:

a) the heat balance within the enclosure and, accordingly, the ongoing fire conditions;

b) their mechanical response and their separating function; and

c) the structural response of structural elements.

The thermal response of structural members can be determined from:

1) empirical data based upon data and observations from fire tests (**10.3.2**);

2) simplistic calculations of the temperature response based on first principles (**10.3.4**); or

3) advanced calculations (**10.3.5**).

More information on heat transfer and thermal response of specific materials is given in Annex C. The thermal properties for non-loadbearing construction systems are given in Annex D.

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 vary widely amongst the different mechanisms. This reflects up-to-date understanding and the level of analysis typically required for practical design purposes.

8.2 Empirical data

Several materials have been extensively evaluated under the standard fire 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 various standard fire heating periods and temperature profiles/ gradients through the structural element in the form of design charts or monograms for use by non-specialist designers.

8.3 Simplistic calculations

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

The energy from the fire is imparted to any exposed surfaces of an element of the structure within, or outside, the enclosure through conduction and radiation. The net heat flux to which an element's surface is exposed, q_{net}, is the sum of the convective and radiative incident fluxes:

$$q_{\mathrm{net}} = q_{\mathrm{net,c}} + q_{\mathrm{net,r}} \quad (1)$$

where:

q_{net} is the net incident heat flux (kW/m^2);
$q_{\mathrm{net,c}}$ is the convective heat flux (kW/m^2);
$q_{\mathrm{net,r}}$ is the radiative heat flux (kW/m^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 surface of the element, $T_{surface}$, and the impinging hot gases, T_g. The convective heat flux per unit surface area, q_c, of the element is given by:

$$q_{\mathrm{c}} = \alpha_{\mathrm{c}}(T_{\mathrm{g}} - T_{\mathrm{surface}}) \quad (2)$$

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}K is more appropriate. At the non-exposed face of an element, where some cooling is expected to occur through convection and radiation, α_{c} should be assigned a value of 4 kW/m^2 or 9 kW/m^2 where the effects of radiation are considered. Further guidance is provided in BS EN 1991-1-2.

When evaluating the radiative heat transfer between fires and non-combustible solids in the enclosure, 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 can be modelled by the relationship:

$$q_{\mathrm{r}} = \phi \varepsilon_{\mathrm{m}} \varepsilon_{\mathrm{f}} \sigma [(T_{\mathrm{g}})^4 - (T_{\mathrm{surface}})^4] \quad (3)$$

where:

q_{r} is the radiative heat flux of the receiver (W/m^2)
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 C.1.

ε_{m} is the surface emissivity of the member;

ε_{f} is the emissivity of the fire.

8.4 Advanced calculations

The thermal 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 thermal response of the element itself requires solving the governing formula of transient conduction subject to the appropriate boundary conditions, as described by Formula (4) within homogeneous and isotropic solids.

$$\frac{\delta}{\delta x}\left(K\frac{\delta T}{\delta x}\right)+\frac{\delta}{\delta y}\left(K\frac{\delta T}{\delta y}\right)+\frac{\delta}{\delta z}\left(K\frac{\delta T}{\delta z}\right)+\frac{Q}{\rho C}=\frac{\delta T}{\delta t} \quad (4)$$

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 in Formula (4) 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 Formula (4). 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 the simulation 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. This is typically achieved by conducting appropriate sensitivity studies.

8.5 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:

$$q_{\mathrm{cond}}=-KA_{\mathrm{a}}\frac{dT}{dN} \quad (5)$$

where:

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

– 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:

$$\frac{d^2T}{dx^2} \times \frac{d^2T}{dy^2} \times \frac{d^2T}{dz^2} = 0 \qquad (6)$$

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

The general formulae 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, solving the governing formulae 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 the 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 the analysis of transient conduction into solid members [9].

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.

8.6 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 for describing heat flow from post-flashover fires or from small and irregularly shaped openings, such as those created during a fire.

In general terms, the rate of heat flow from the enclosure of fire origin can be calculated using Formula (7):

$$q_{\mathrm{conv}} = MC_{\mathrm{g}}(T_{\mathrm{g}} - T_{\mathrm{o}}) \quad (7)$$

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:

$$M = 0.5A\sqrt{h_{\mathrm{W}}} \quad (8)$$

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 **8.8**.

8.7 Quantitative analysis of heat flow by radiation

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

$$q_{\mathrm{rad}} = \varepsilon_{\mathrm{g}}\sigma T^{4}{}_{\mathrm{g}} A_{\mathrm{rad}} \quad (9)$$

Alternatively, fires can be represented as a surface or multiple surfaces with a surface emissive power:

Surface emissive power = radiative heat release rate divided by surface area.

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

a) the relative positions of emitting and receiving surfaces;

b) the emissivity of the emitting and receiving surfaces;

c) transmissivity and absorptivity of any fluids or solids through which radiation passes;

d) the temperature of the receiving surface; and

e) re-radiation and associated change in temperature of surrounding surfaces.

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 **8.3** 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:

$$q_{\mathrm{rec}} = F_{\text{e-R}} q_{\mathrm{rad}} \tag{10}$$

where:

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

Complex surfaces can be approximated by summing the configuration factors of multiple, more simple surfaces and time-based assessments can be conducted by changing size and shape and temperature of emitting surfaces with respect to time.

Commonly used configuration factors can be found in [10] or alternative references in heat transfer theory.

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.

A range of convenient methods has been developed to allow evaluation of the potential for radiative fire spread between openings from an enclosure and a parallel plane some distance away.

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 Formula (9) should be replaced by T_{surface}, the temperature of the unexposed surface (in kelvins).

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

$$T(x,t) = T_0 + \frac{\varepsilon_{\mathrm{rec}} q_{\mathrm{r}}}{\alpha_{\mathrm{c}}} \left[erfc\left\{ \frac{0.5x_{\mathrm{s}}}{\sqrt{Kt}} - \exp(\alpha_{\mathrm{c}} x_{\mathrm{s}} + K\alpha_c^2 t) \right\} erfc\left(\frac{0.5x_{\mathrm{s}}}{\sqrt{Kt}} + \frac{\alpha_{\mathrm{c}}}{K}\sqrt{Kt} \right) \right] \tag{11}$$

where:

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

Formula (11) 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.

8.8 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 or heat flux not less than the maximum 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.

The flow of hot gases from an enclosure entrains air and forms a smoke plume in the adjacent space. The recommended mass entrainment [11] into the plume above an opening, $M_{opening}$, is given by:

$$M_{opening} = 0.68(A_W \sqrt{h_W})^{\frac{1}{3}}(Z_W + a)^{\frac{5}{3}} + 1.59 A_W \sqrt{h_W} \quad (12)$$

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) = $2.4 A_W^{2/5} h_W^{1/5} - 2.1 h_W$

Entrainment Formula (12) assumes a convective heat release, q_{conv} (kW), from the enclosure of:

$$q_{conv} = 1260 A_W \sqrt{h_W} \quad (13)$$

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

9 Behaviour of separating elements in fire

9.1 Behaviour of fire-resisting separating elements

9.1.1 General

Predicting the behaviour of some materials, e.g. glass (see **9.1.6**) and polymers (see Clause **8**) 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 **8**. 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 beyond the deflection criteria limits used in standard fire testing. Deflections beyond these limits 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 [12]. This should be reviewed by the designer.

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.

9.1.2 Elements primarily composed of concrete or masonry

9.1.2.1 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.

9.1.2.2 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" (negative bending moment) 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 lightweight 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.

Un-reinforced 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.

9.1.2.3 Integrity performance of concrete

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.

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. Spalling is likely to be more of a problem in high-strength concrete rather than in normal or low-strength/low-density concrete. See BS EN 1992-1-2 for more information on spalling of 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. Where proprietary fixings have been tested and issued with a classification under European Technical Approvals, a fixing type/depth/concrete grade may be provided. 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.

9.1.2.4 Insulation performance of concrete

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 provides a method for making corrections.

9.1.2.5 Masonry and brick walls

Blockwork walls can be loadbearing or non-loadbearing. BS EN 1996-1-2 provides tabulated data for the fire resisting performance of both loadbearing and non-loadbearing walls.

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 BS EN 1996-1-2 and LPCB guidance [13]. Additional information is given in the ASFP publication on fire stopping and penetration seals [14]. 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 (see Annex G), 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.

9.1.2.6 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 **10**), 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.

9.1.3 Elements primarily composed of metal

9.1.3.1 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 **9.1.5.1**):

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 [12]. 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 or where collapse of the roof would lead to progressive failure due to dynamic loading on other building elements such as fire resisting walls or elements of structure, or where the roof structure provides support to floors below that are 'hung off' the roof. External cladding might require fire resistance from inside or outside depending upon the fire strategy approach taken.

9.1.3.2 Integrity performance of metal

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.

9.1.3.3 Insulation performance of metal

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 to the effects of radiation and smoke leakage.

9.1.3.4 Radiation of metal

Metals, particularly steel, reach high temperatures when exposed to fire and radiate heat readily into any place of relative safety (see Clause **8**).

9.1.4 Elements primarily composed of timber

9.1.4.1 General

Timber is used extensively in the construction of floors and walls as loadbearing members.

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.

9.1.4.2 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 [15].

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 [16].

9.1.4.3 Integrity performance 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 with 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.

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.

9.1.4.4 Insulation performance of timber

Timber has a very low thermal conductivity 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.

9.1.5 Elements constructed from composite panels

9.1.5.1 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 D. 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 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) [17].

9.1.5.2 Integrity performance 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 **9.1.3.2**, 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.

9.1.5.3 Insulation performance 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 change 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.

9.1.5.4 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 place of relative safety. 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.

9.1.6 Elements primarily composed of glass

9.1.6.1 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 **9.3**).

9.1.6.2 General consideration of the response of glass elements

There are many different types of fire-resisting glass, which broadly fall into three major categories:

a) monolithic glasses, wired and unwired, which do not provide any significant level of insulation;

b) composite glasses consisting of glass in combination with intumescent/ablative materials, which control temperature rise and provide protection from radiation; and

c) 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.

9.1.6.3 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:

a) using glasses with lower coefficients of thermal expansion;

b) reducing the edge cover to the glass pane to a minimum; or

c) 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 approximately 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.

9.1.6.4 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.

9.1.6.5 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.

9.1.6.6 Integrity performance of glass

Monolithic clear glasses can produce an integrity failure as a result of cracking (see 9.1.6.3). 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) 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.

9.1.6.7 Insulation performance of glass

Conventional soda/lime composition annealed glass cannot satisfy the integrity criteria for more than a few minutes 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.

9.1.6.8 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.

9.2 Maintaining the separating capability of elements or constructions

9.2.1 Fire stopping and linear gap sealing between separating elements

9.2.1.1 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.

9.2.1.2 General consideration of the behaviour of gaps

Any sealant applied around imperfections of fit or functional discontinuity should provide an appropriate level of performance in containing a fire for the structure or elements to which it is applied. 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 place of relative safety.

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.

9.2.1.3 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 [14].

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.

9.2.1.4 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 D.

9.2.2 Service penetrations passing through elements or protection systems

9.2.2.1 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.

9.2.2.2 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

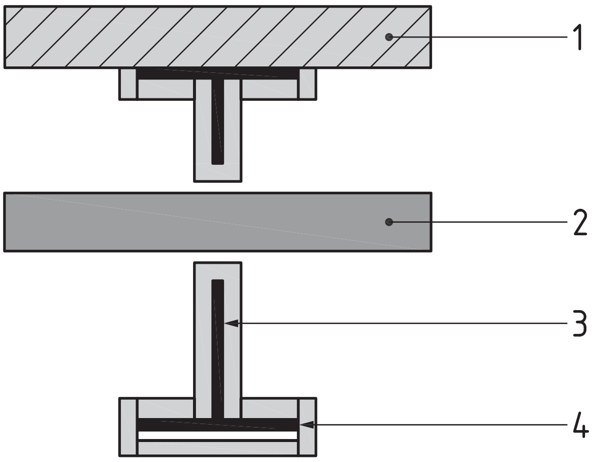
b) building a strict inspection and maintenance regime into the management section of the fire manual to prevent unprotected apertures remaining unsealed.

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) (see ASTM E119 - 09c and [14]).

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 4.

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 4 — *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

9.2.2.3 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; or

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.

9.2.2.4 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.

9.3 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 1.

Table 1 — *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

Table 1 *(continued)*

Boundary type	Notional period of fire endurance min
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 1, it might be possible for competent persons to quantify the time of penetration by fire more accurately using numerical engineering tools.

10 Analysis of structural response of loadbearing structural elements and frames

10.1 Concepts

10.1.1 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:

a) materials lose stiffness and strength with increasing temperature;

b) 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.

c) 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. This clause provides guidance on how to assess the performance of structural elements and assemblies at elevated temperatures.

10.1.2 Actions and effects on structure in fire situation

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 ($E_{d,t}$):

$$E_{d,t} = \gamma_G G_k + \psi_{1,1} Q_{k,1} + \sum \psi_{2,i} Q_{k,i} + \sum F_d(t) \quad (14)$$

where:

$F_d(t)$ is the design value of an action (e.g. applied force or moment) at time t;

Ψ_1 is the partial safety factor for frequent value of an imposed load;

Ψ_2 is the partial safety factor for quasi-permanent value of an imposed load; and

γ_G is the partial safety factor for permanent actions (may be taken as 1.0 in fire situation).

The coincidental application of accidental loads is very unlikely and, accordingly, the fire design case attracts differing partial safety factors. Table 2 shows the values that are included in Formula (14). 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 situations should not be confused with "fire load", which refers to the combustible contents within an enclosure (see BS 7974).

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 2 — *Partial safety factors for loads (PD 6688-1-2:2007, Table 1)*

Action		ψ_1	ψ_2
Imposed loads in buildings	Domestic, residential areas, office areas	0.5	0.3
	Congregation areas, shopping areas	0.7	0.6
	Storage areas	0.9	0.8
	Roofs		
Snow on buildings for sites located at altitude $H \leq 1\,000$ m above sea level		0.2	0
Wind loads on buildings		0.2	0

*NOTE Additional information on how to use the partial safety factors for loads in a fire situation is provided in PD 6688-1-2:2007, Clause **2**.*

10.1.3 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 8 Pa per metre of height on the exposed face;
- impact loads from collapsing fire-affected elements, including service plant;
- impact of firefighting 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; and
- loads applied due to failure of building contents to remain self-supporting, e.g. ducts and services.

10.1.4 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 [18].

10.2 Acceptance criteria

10.2.1 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:

$$R_{\mathrm{d,t}} \geq E_{\mathrm{d,t}} \tag{15}$$

10.2.2 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.

10.2.3 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.

10.2.4 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; and

- appropriate acceptance criteria are imposed to ensure that the response of the structural elements does not adversely affect the performance of the non-structural element.

10.3 Methods for determining structural response

10.3.1 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. Many of the methods and considerations given in 10.3.2 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.

10.3.2 Empirical data from testing

10.3.2.1 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.

10.3.2.2 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 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.

10.3.2.3 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 should not 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 or floor, additional protection might be required to achieve the insulation requirements of the overall wall/floor structure.

10.3.3 Equivalent time of fire exposure

The anticipated fire conditions within the enclosure can be characterized with reference to a set duration of a standardized gas temperature/time relationship (see BS 476-20).

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

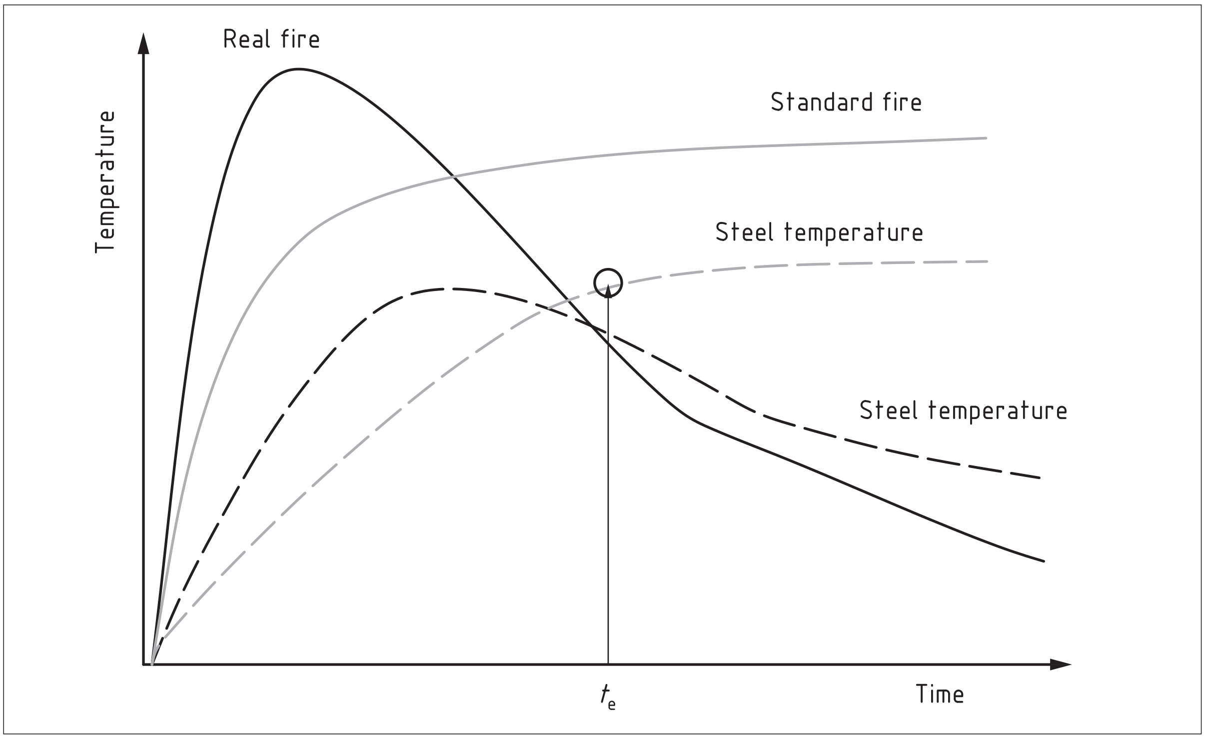
It should also be noted that given that the fire severity is measured as a comparison to the standard fire, the same limitations associated with the standard fire test apply. Thermal expansion (failure is assumed to be due to material response alone) and load re-distribution are neglected.

A number of researchers have proposed methods for correlating durations of exposure in the standard test to real fires using different metrics such as the maximum steel temperature or the energy load. The most notable have been developed by Law [19], Harmathy [20] and Pettersen [21]. The approach originates from Inberg where the area under the temperature time curve is used as a metric of equivalent severity (subsequently shown not to be appropriate) [22].

A typical method of time equivalence is that known as the “graphical method” based on the maximum steel temperature concept. This approach assumes that the failure of a steel member in a “real fire” can be represented based on its critical temperature and that it can be compared to that derived for the “standard fire”.

Figure 5 illustrates the concept graphically.

Figure 5 — *Maximum steel temperature concept*



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 [23].

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

$$t_e = k_b w_f q \tag{16}$$

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 the National Annex 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 3.

Table 3 — *Values of k_b*

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

w_f is the ventilation factor and is defined by:

$$w_{\mathrm{f}} = 6.0H^{-0.3}[0.62 + 90(0.4 - \alpha_{\mathrm{v}})^4](1 + b_{\mathrm{v}}\alpha_{\mathrm{h}})^{-1} \geq 0.5 \quad (17)$$

where:

$\alpha_{\mathrm{v}} = A_{\mathrm{v}} / A_{\mathrm{f}}$	is the area of the vertical openings in the façade (A_v) related to the floor area of the compartment (A_f);
$\alpha_{\mathrm{h}} = A_{\mathrm{h}} / A_{\mathrm{f}}$	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:

$$b_v = 12.5[1 + 10(A_v / A_f) - (A_v / A_f)^2] \geq 10 \quad (18)$$

Formula (16) 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. [24]).

For construction materials, reinforced concrete, protected steel and unprotected steel, BS EN 1993-1-2 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 [25] demonstrated that the time-equivalent relationship for unprotected steel can only be validated for fire resistance periods up to 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. As described in PD 6688-1-2, the UK does not consider this approach acceptable in the engineering calculations of 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.

Designers 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 two-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 shown in BS 9999:2017, Table 24.

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

risk = frequency × probability × consequence of failure.

It should be noted that a similar risk-based approach could be adopted for the selection of design fires for thermo-mechanical assessment.

As previously described, BS EN 1991-1-2 takes an implicit approach for accounting risk (presence of sprinklers, fire brigade presence etc.) and uncertainty by appropriately adjusting the fuel load density so that a deterministic calculation can be undertaken. However, both the uncertainty of the input of the design fire calculation as well as the performance metrics can be evaluated explicitly following the procedures described in PD 7974-7.

An example is the application by Kirby et al [26] for the development of BS 9999, where the uncertainty in the input parameters for the BS EN 1991-1-2 parametric fire where sampled probabilistically using through a Monte Carlo analysis process. Similar approaches have been more recently adopted for travelling fires (see [27]).

10.3.4 Simplistic calculation methods

10.3.4.1 Approach

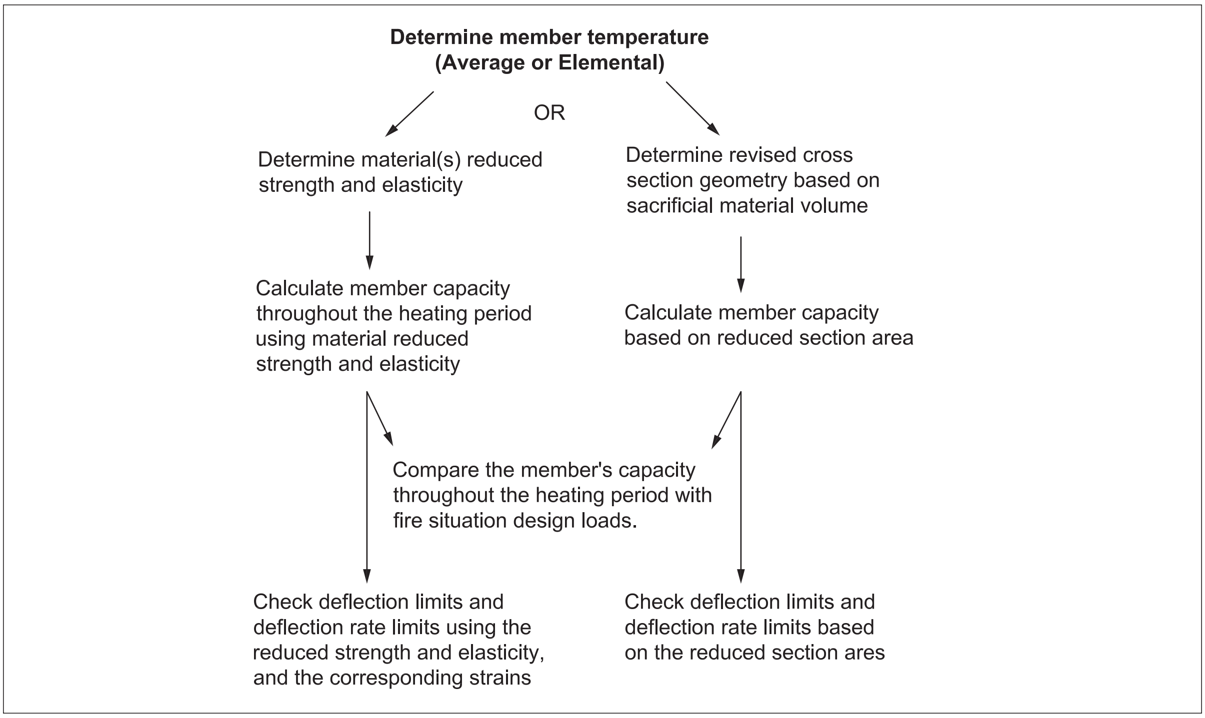
As with design for ambient conditions, many structural assemblies can be considered as a series of individual, isolated members at the fire limit state. This is usually the simplest approach and, in most cases, adopting such an approach provides conservative solutions. The analysis methods vary between 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 6.

There are some cases where the approach is slightly different and these are discussed within the material-specific clauses in Annex E.

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 6 — *General approach to structural fire safety design*



10.3.4.2 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.

10.3.4.3 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 in a steel truss or space frame where a member is subjected to localized heating or for unusual structural features [28], [29].

10.3.5 Advanced calculation methods

10.3.5.1 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:

a) the assembly's ability to redistribute loads via alternative load paths (for example, columns acting in tension);

b) continuity; and/or

c) 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 [28], [29].

10.3.5.2 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.

a) 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.

b) The effects of thermally induced strains and stresses due to temperature rise and temperature differentials should be considered.

c) The model for mechanical response should also take account of:

   1) the combined effects of mechanical actions, geometrical imperfections and thermal actions;

   2) the temperature-dependent mechanical properties of the material;

   3) geometrical non-linear effects;

   4) the effects of non-linear material properties, including the unfavourable effects of loading and unloading on the structural stiffness.

d) Attention should be given to whether thermal creep is explicitly considered.

e) Attention should be given to whether material-specific phenomena, such as concrete spalling or charring of timber are explicitly considered.

f) 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.

10.3.5.3 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* [30].

10.3.5.4 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:

a) the sophistication of the element formulations;

b) the adequacy of the selected sub-frame to represent overall building behaviour;

c) the finite element mesh density; and

d) 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 is sufficiently representative of the scenario and that any differences are justifiable.

10.3.5.5 Considerations

Finite element analysis is a complicated procedure and it is important that the designer has sufficient knowledge and experience to develop the finite element model, conduct the analyses and interpret the results. The designer 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 include maximum allowable strains, maximum allowable deflections or maximum allowable curvatures.

The mechanical response of a structure can be very sensitive to the fire exposure and thermal distribution within the structure [31]. 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.

It is difficult to model localized behaviour, including reinforcement fracture and connection behaviour, in finite element analysis. Therefore care should be taken that:

a) the model is sufficiently sophisticated to predict local behaviour;

b) appropriate acceptance criteria (such as deflection limits) are applied so that local failures do not occur; or

c) local failures do not significantly impact on the overall performance of the structure in terms of its functional requirements.

Annex A (informative) Fire spread mechanisms

A.1 Mechanisms for fire spread

A.1.1 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) fuel transfer; or

e) direct pyrolysis.

A.1.2 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:

a) ignition of the non-exposed surface; or

b) conduction of heat from a non-exposed surface to combustibles with which it has direct contact; or

c) 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.

A.1.3 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, for example, 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.

A.1.4 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.

A.1.5 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.

A.1.6 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.

A.2 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 suppression 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, which 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, which can distort or otherwise react to fire exposure in a manner that allows fire to bypass the fire stopping.

A.3 Routes for fire spread

Once the enclosure has been characterized, the designer needs to identify all possible routes of fire transmission. Figure A.1 illustrates some of the more common direct routes for potential fire spread. In many instances, the designer needs to also consider the potential for fire spread between two adjoining enclosures via independent spaces. These routes of fire spread need to be examined as a series of direct spread mechanisms.

All potential routes for fire spread from the enclosure need to be investigated. 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.

The Association of Specialist Fire Protection (ASFP) provides guidance in their red book on effective fire stopping [14].

Figure A.1 — *Routes for fire transmission*

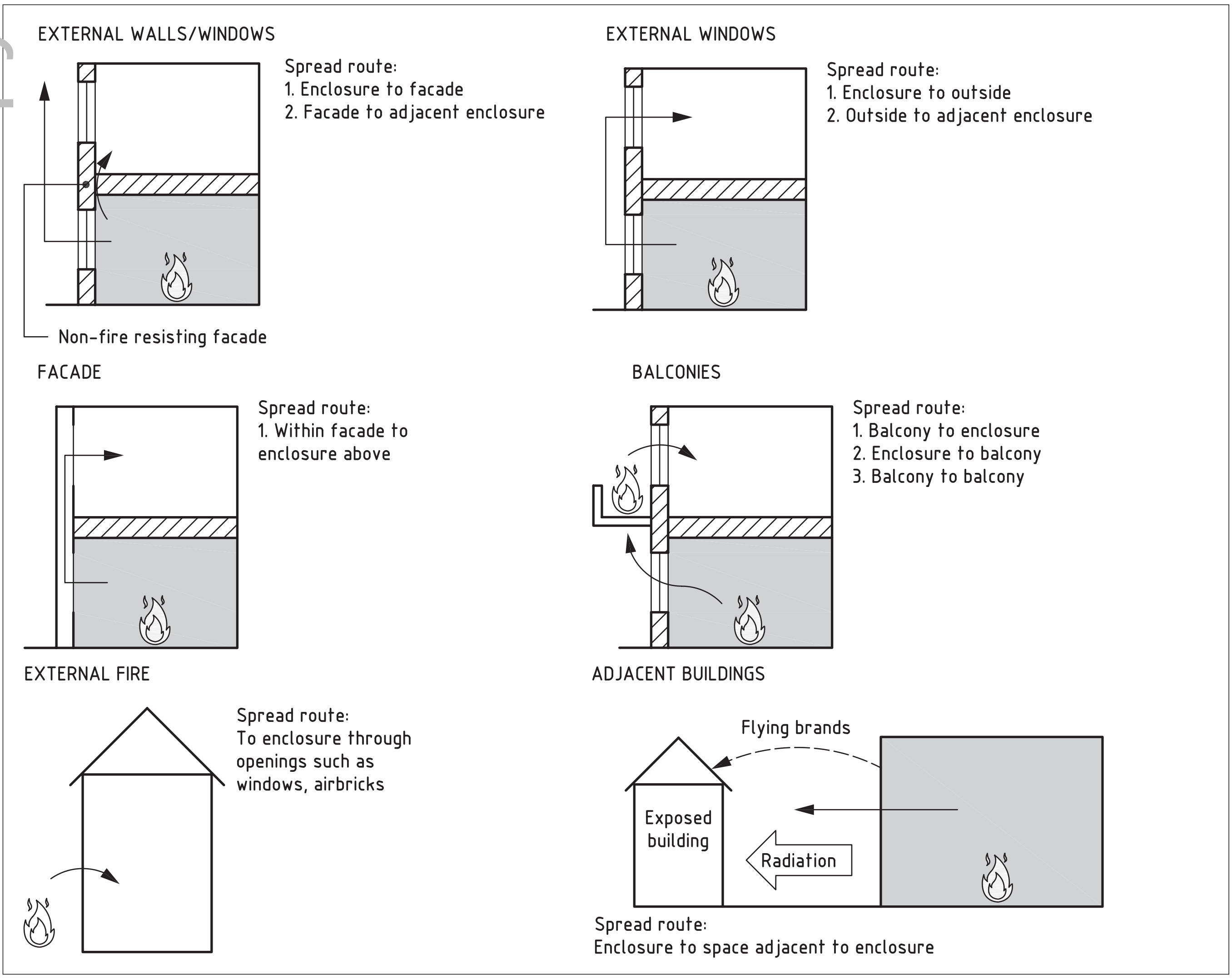


Figure A.1 *(continued)*

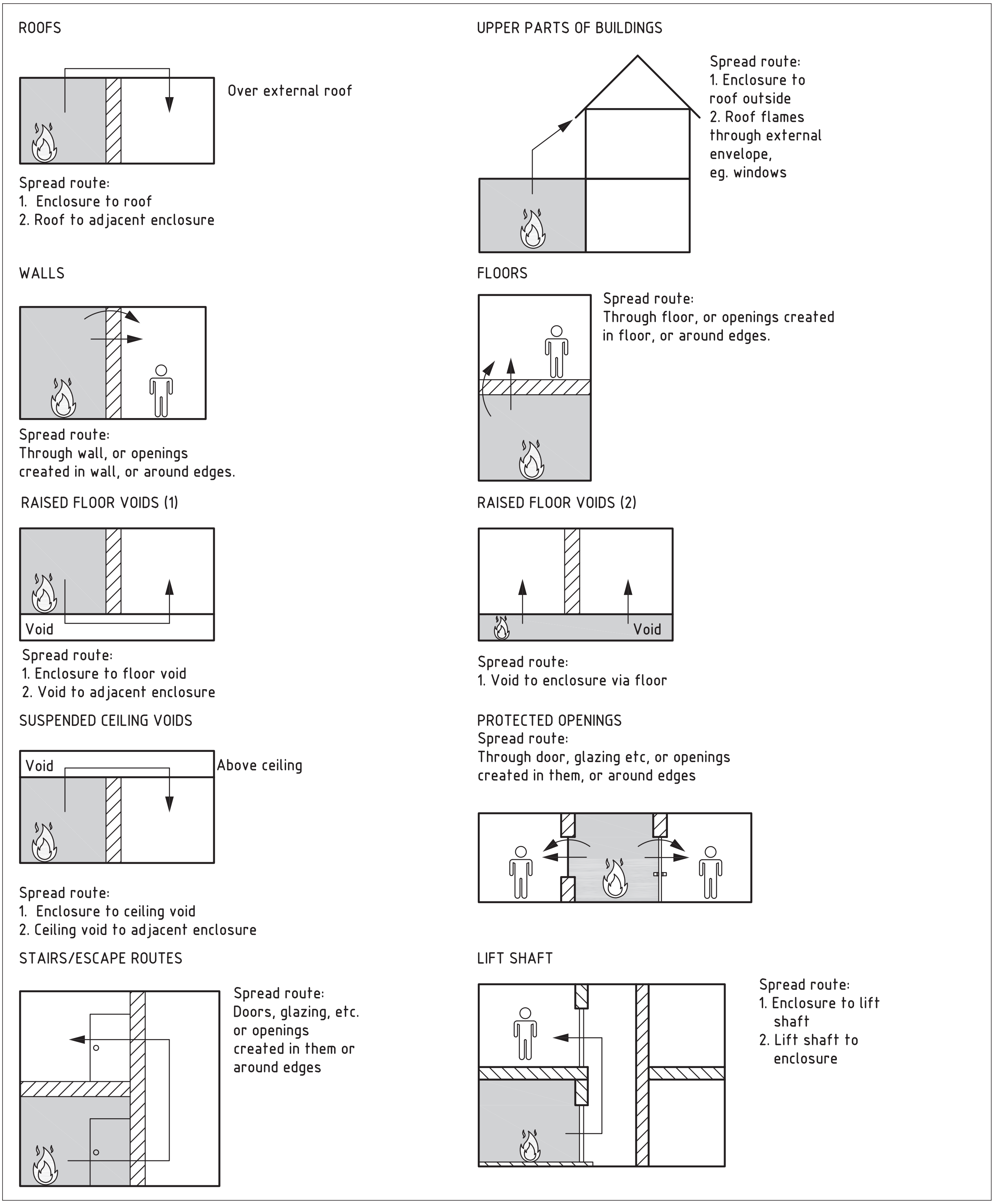
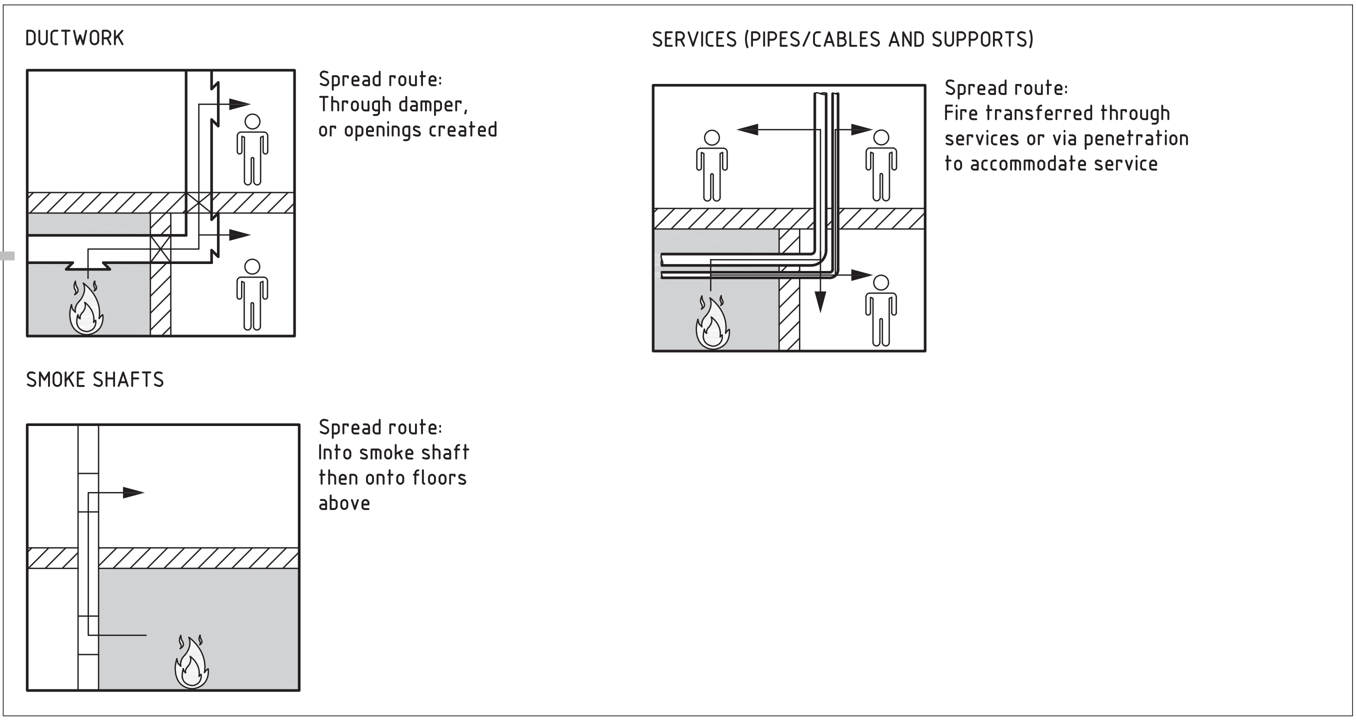


Figure A.1 *(continued)*



Annex B (informative) Design fires

B.1 General

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 needs to 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 configured differently to those submitted for fire testing.

For example, 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 needs to 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.

B.2 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. Care needs to 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. 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 need to 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 needs to describe and justify all relevant parameters.

It is advised that 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 follow the procedures specified in ISO/TR 15658. By following the procedures, the results of the tests are likely to be more comparable with data from other experimental testing, thereby extending their applicability.

Annex C (informative)
Heat transfer (and thermal response) of specific materials

C.1 General

The emissivity of the fire is taken as 1.0. The emissivity of the material varies according to Table C.1. Figure C.1 gives configuration factors for typical scenarios.

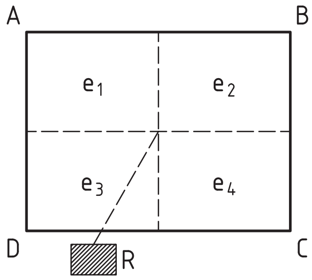
Table C.1 — *Guidance on the material surface emissivity of construction materials*

Material	Surface emissivity ε_m
Concrete	1.0
Steel (carbon)	0.8
Stainless steel	0.63
Timber	1.0

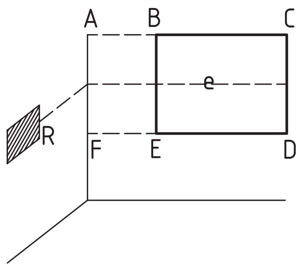
Table C.1 *(continued)*

Material	Surface emissivity ε_m
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

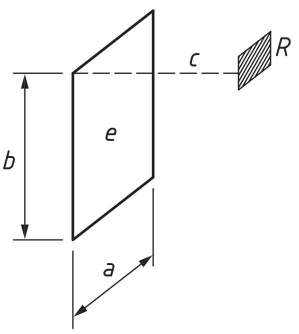
Figure C.1 — *Configuration factors for typical scenarios*



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

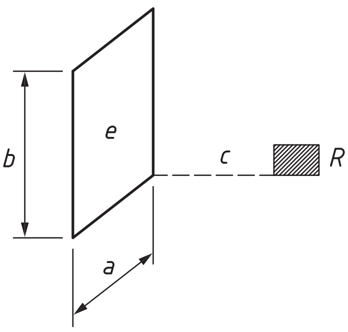


$$F_{\text{e-R}} = F_{\text{ACDF-R}} - F_{\text{ABEF-R}}$$



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

where $X = \frac{a}{c}$ and $Y = \frac{b}{c}$



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

where $X = \frac{a}{c}$ and $Y = \frac{b}{c}$

C.2 Thermal response of reinforced concrete members

C.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 loadbearing members 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 of 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 codes such as BS EN 1992-1-2 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.

C.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 [32];

b) fire resistance tests to evaluate the effect of polypropylene fibres on the performance of high strength columns [33];

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

d) report by the Comité Euro-International du Béton [34].

Information on the development of temperature within concrete members exposed to the standardized fire conditions is given in Formulae (C.1) and (C.2) (see BS 476-20). 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 [34].

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

a) specific heat;

b) moisture content of 1.5%;

c) thermal conductivity at the lower limit;

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

d) an assumed emissivity for concrete of 0.7.

C.2.3 Simplistic calculation of the temperature response of concrete

Wickström [35] 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.

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:

$$T_x = n_x T_s \quad \text{(C.1)}$$

and

$$T_S = n_S T_g \quad \text{(C.2)}$$

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.

$$t_s = (\gamma / \gamma_i)t \quad \text{(C.3)}$$

where:

t_s is the scaled time (hours);

$\gamma = \sqrt{\Gamma}$;

Γ is the compartment time factor ($m^{5/2}K/s^{1/2}W$);

$\gamma_i = \sqrt{b/1550}$;

$$\sqrt{k_c \rho_c C_c \Gamma} = \left(\frac{\left(A_W \sqrt{h_W} \right) / A_t}{b} \right)^2 \left(\frac{1160}{0.04} \right)^2 \quad \text{(C.4)}$$

When predicting the response of normal weight concrete to the standard BS 476-20 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:

$$n_c = 1 - 0.0616 t_s^{-0.88} \quad \text{(C.5)}$$

where:

T_s is the scaled time in house (see Formula C.3).

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

$$n_x = 0.18\ln(U_x) - 0.81 \tag{C.6}$$

where:

$$U_x = \frac{K_c}{4.17\times10^{-7}} - \frac{t_s}{x^2} \tag{C.7}$$

where:

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

x is the depth (m).

Formula (C.5) 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 Annex D. Formulae (C.1) to (C.7) can be simplified for applications considering the temperature development in normal weight concrete heated under conditions specified in BS 476-20. In this case the temperature at a depth x metres (T_x) beneath the surface at time t hours is given by:

$$T_x = 345\log t(480+1)(1-0.0616t^{-0.88})(0.18\ln\frac{t}{x^2} - 0.18) \tag{C.8}$$

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:

$$T_{xy} = \left\{n_s(n_x + n_y - 2n_x n_y) + \mathrm{n}_x\, n_y\right\}T_s \tag{C.9}$$

C.2.4 Fire protection for concrete

Due to the relatively low thermal conductivity of concrete structures fire protection is not normally required for reinforced concrete 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 concrete members to prevent spalling or to make up for deficiencies in the concrete cover to the steel reinforcements.

In BS 8110 (now superseded), 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. For concrete structures designed to BS EN 1992-1-2, a quantified justification might be necessary.

By carrying out calculations, the thickness of other types of insulation can be substituted, provided that they achieve adequate adhesion with the substrate (in this case with the concrete surface) and that appropriate and verified fixing methods are followed to avoid debonding due to possible pore pressures generated at the interface of the concrete and the selected protection material for the full duration of the fire.

C.3 Thermal response of iron and steel members

C.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.

C.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 [36] for a range of steel section sizes and configurations. The data are supplemented by computer simulations reported in Wainman et al [37].

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 eight-storey steel frame building. In each test programme the temperatures of both unprotected and protected steel members were extensively monitored ([21] and [38]). In addition, FRS (BRE) also conducted two fire tests on the eight-storey steel frame building (see [39]).

C.3.3 Simplistic calculations of the temperature response of steel members

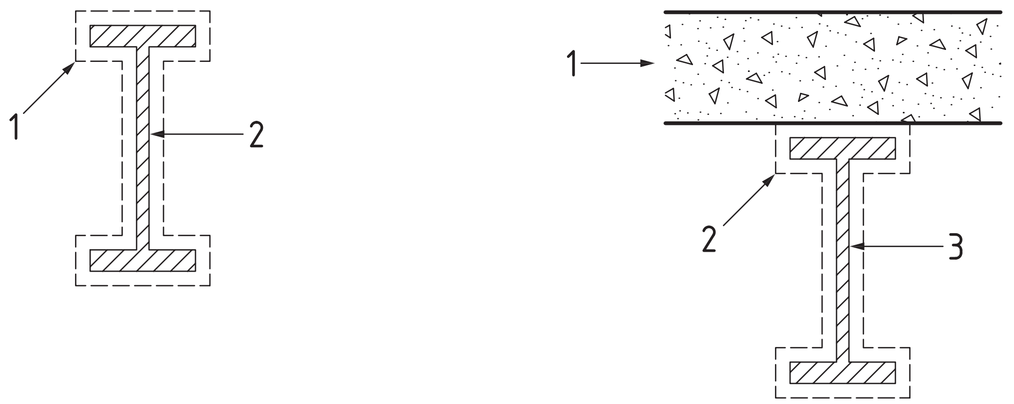
C.3.3.1 Unprotected steel

C.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 resistance so as to not require any additional fire protection. However, the lower the section factor the higher the internal temperature gradient in the steel member and this is something that needs to be considered when evaluating the mechanical response of the steel member.

The section factor, by definition, requires knowledge of the geometry and configuration of the member used in the building. This is illustrated in Figure C.2 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 C.2 — *Calculation of section factors*



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

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

Key

1 H_p
2 254 mm × 102 mm × 28 kg/m UB

1 Concrete slab
2 H_p
3 254 mm × 102 mm × 28 kg/m UB

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) [40] and are also available from Tata Steel [41].

Empirical calculations on the temperatures attained by unprotected steel members are reported in Wainman et al [37].

C.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:

$$\Delta\theta_{a,t} = k_{sh}\frac{h_{net,d}}{\rho_a C_a}(A_m / V)\Delta t \qquad (C.10)$$

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});

$h_{net,d}$ 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:

$$k_{sh} = 0.9(A_m / V)_b / (A_m / V) \qquad (C.11)$$

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.

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, is given by:

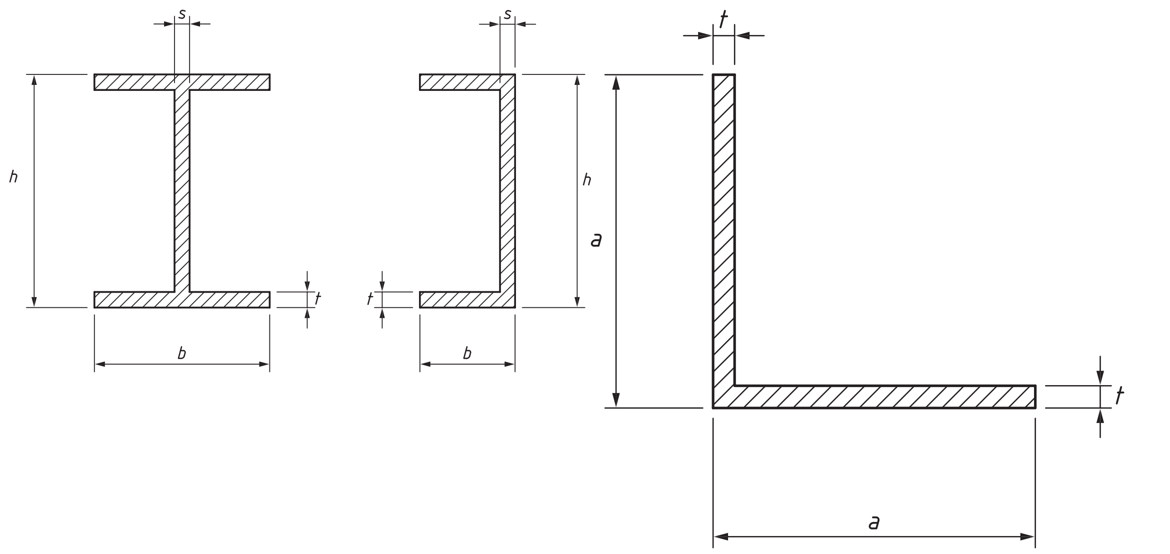
$$\Delta\theta_{a,t} = \frac{h_{net,d}}{\rho_a C_a}(EF)\Delta t \qquad (C.12)$$

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 C.2 and Figure C.3.

Table C.2 — *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 C.3 — *Calculation of element factors*



a) Beams, channels and columns b) Angles

C.3.3.2 Protected steel

C.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;

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 F.

C.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.

BS EN 13381-4 describes the analysis methods for determining the non-reactive 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 850 °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. 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.

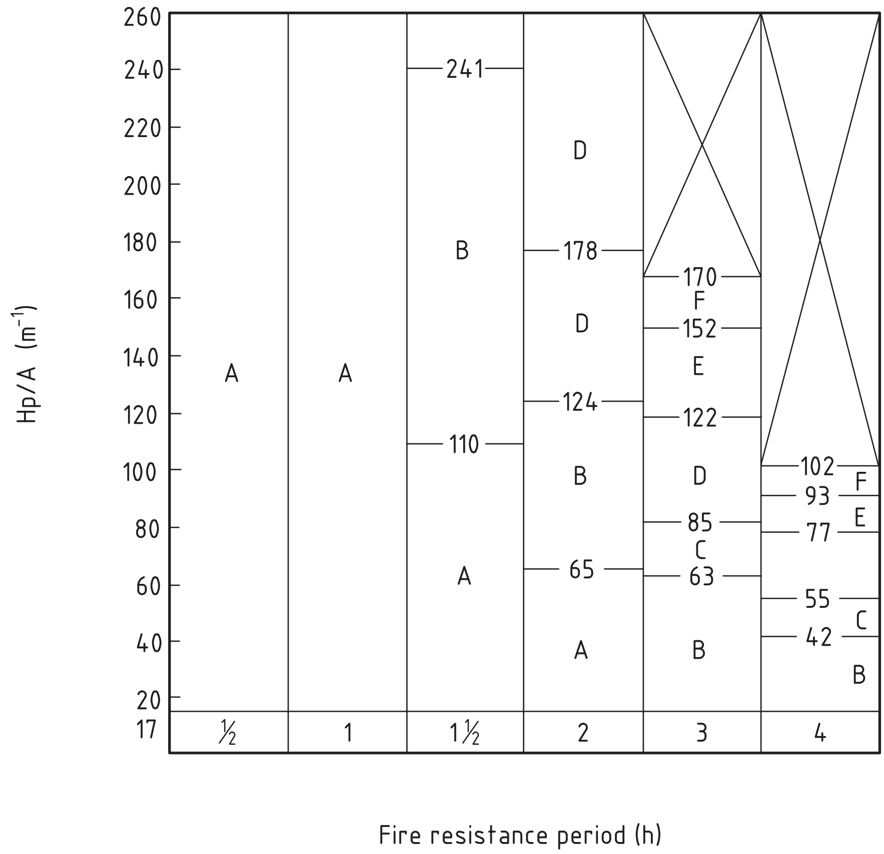
Table C.3 illustrates the relationship between section factor and protection thickness for different fire resistance periods for a non-reactive spray-applied system.

Table C.3 — *Typical set of coating thicknesses for a profile non-reactive 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 C.4 illustrates an example of the relationship between section factor and protection thickness for different fire resistance periods for a box encasement system.

Figure C.4 — *Typical set of board thicknesses for a box encasement fire protection system*



C.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 member's limiting 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 [40]. For loadbearing capacity, many DFT values are based on recommended limiting carbon steel temperatures of 550 °C and 620 °C for columns and beams respectively. These are understood to be based on permissible working stress design assumptions with respect to utilization of steel. They assume a fully loaded member at ambient design. Alternate default limiting temperatures for designs in accordance with the Eurocodes are given in the UK National Annex to BS EN 1993-1-2 and the Association of Specialist Fire Protection [40]. In fact, detailed analysis might conclude that limiting temperatures in excess of these values could also be appropriate. 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 reactive protection materials such as intumescent coatings are given BS 476-20, BS 476-21 and also in BS EN 13381-8.

The designer needs to conduct proper preparation, priming and sealing of intumescent coatings and 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 needs to 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 needs to also be satisfied that the intumescent coating system is suitable for the application and environmental conditions. The stickability of the coating needs to 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.

C.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. Beams with web openings (Cellular beams) can also be fabricated from steel plate.

The rules governing the determination of the protection thickness for protected solid-web steel members do not apply to cellular and castellated sections.

A cellular beam will typically not fail in the same structural mode as a solid-web beam. Generally they will be governed by behaviour of the web, e.g. web-post buckling. As such, the failure temperature of a cellular beam might be less than that of a solid-web beam. It is advised that designers seek specialist advice on the performance and protection of these types of beam. A structural assessment calculation is available in accordance with SCI guidance, RT1356 [42].

It is important to also note that protection materials ought to have been tested for performance on beams with web openings. Further information on this is available from specialist manufacturers, BS EN 13381-9 and the Association of Specialist Fire Protection [40].

For reactive fire protection (intumescent coatings), the size of the post between the holes has a major influence on the structural performance of the beam (see [40] and [43]). These examples are limited to cellular 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 might 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 are only to be viewed as valid in the context of particular calculation methods and are not to 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.

C.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:

$$\Delta T_{\mathrm{m}} = \frac{1}{\rho_{\mathrm{m}} C_{\mathrm{m}}} \left(\frac{H_{\mathrm{p}}}{A} \right) \frac{k_{\mathrm{i}}}{d_{\mathrm{i}}} (T_{\mathrm{g}} - T_{\mathrm{m}}) \Delta t \qquad \text{(C.13)}$$

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).

Formula (C.13) 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:

$$\Delta T_m = \left[\frac{\lambda_i / d_i A_i}{C_m \rho_m V_i}\left(\frac{1}{1+\Phi/3}\right)(T_g - T_m)\right] - \left[(e^{\Phi/10} - 1)\Delta T_g\right] \quad \text{(C.14)}$$

where:

$$\Phi = \left(\frac{C_i \rho_i}{C_m \rho_m}\right) d_i \left(\frac{A_i}{V_i}\right) \quad \text{(C.15)}$$

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).

Formulae (C.13) to (C.15) 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.

Formula (C.14) 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:

$$t_d = \frac{P_W \rho_i d_i^2}{5k_i} \quad \text{(C.16)}$$

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 Formula (C.16).

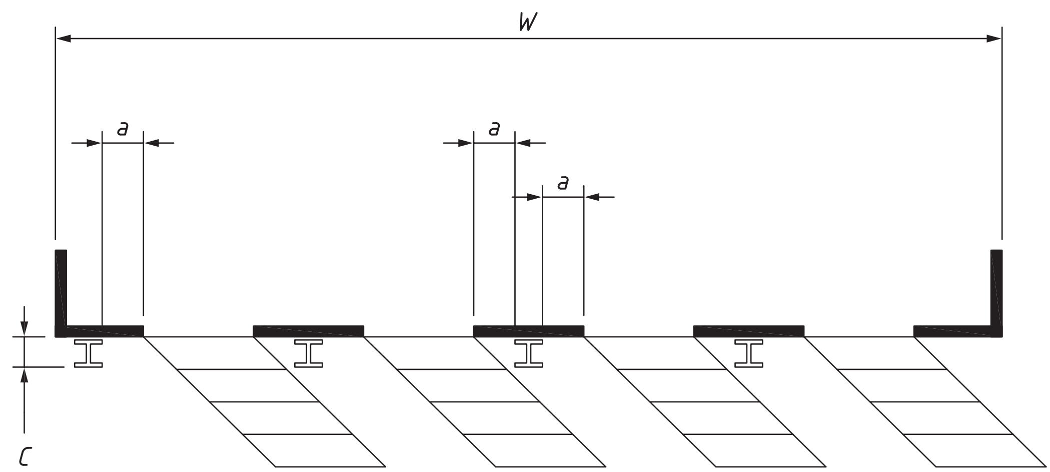
C.3.3.3 Temperatures attained by external members

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 [44], and has 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 [44] 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 C.5 and Table C.4 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. The methodology has not been developed taking into account the fire dynamics in large compartments (see travelling fires) therefore the designer needs to be careful that the outcome is conservative.

Figure C.5 — *Compartment parameters*



Key

W Width

C Lateral position of column relative to the opening

a Side position of column relative to the opening

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

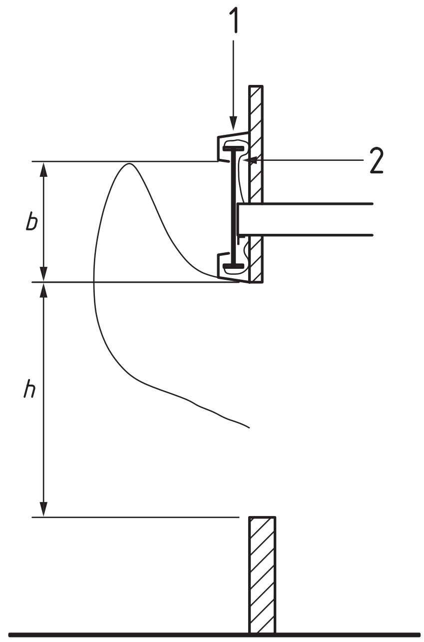
Table C.4 — *Location of columns between windows to avoid direct flame impingement*

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 C.6 and Table C.5).

Figure C.6 — *Spandrel beam with shielded flanges*



Key

1 Shielding to flanges

2 Spandrel beam

b Maximum exposed web height

h Window height

Table C.5 — *Spandrel beams*

Dimensions in metres

Window height	**Maximum exposed web height**
h	*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.

C.4 Thermal response of timber

C.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.

C.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 [45]).

Some limited data are also available from the Timber Research and Development Association (TRADA) [15] on fire tests carried out in real buildings.

C.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. For soft wood, the char line occurs at approximately 300 °C.

Calculations on the depth of char are also given in Annex E 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.

C.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 chooses 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 $t_{ch} < t_f$.

C.5 Masonry

C.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, resulting in a temperature gradient being developed. This can generate thermal stresses which cause bowing of the wall towards the fire.

C.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 [46].

C.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 C.7.

Figure C.7 — *Calculation methods for determining the temperature profiles though masonry elements*

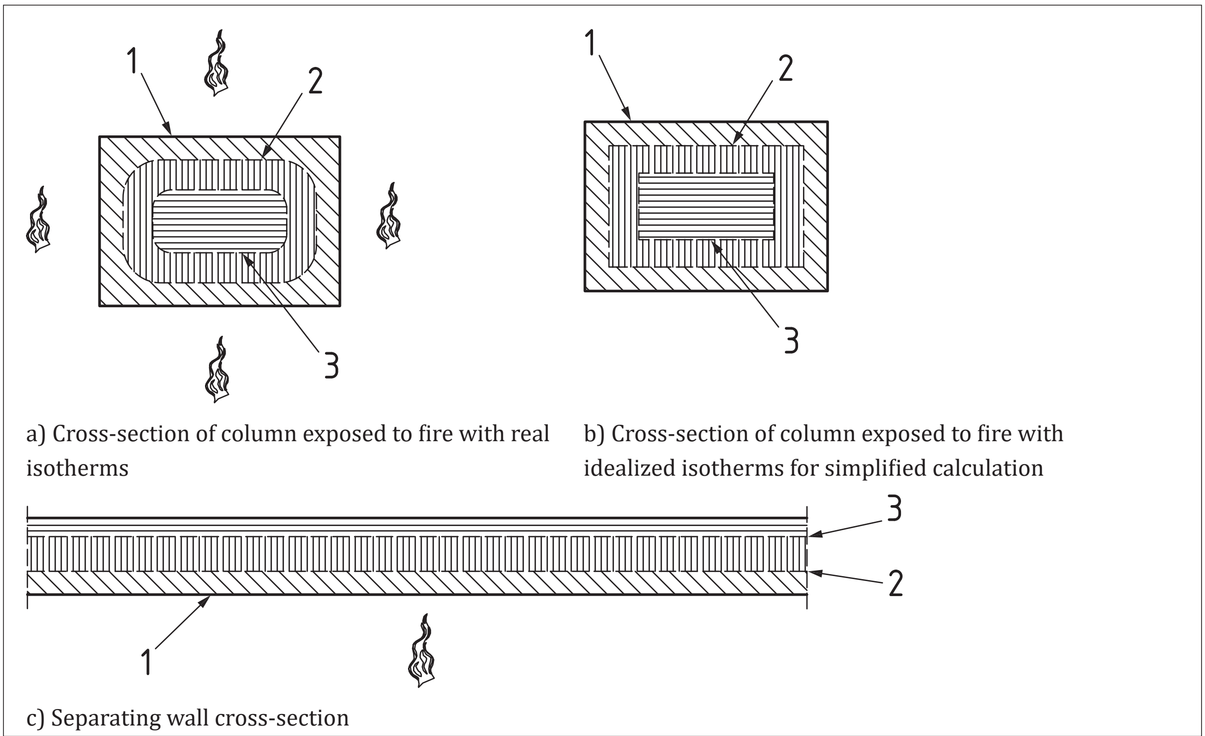


Figure C.7 *(continued)*

Key

1 Boundary of original cross-section

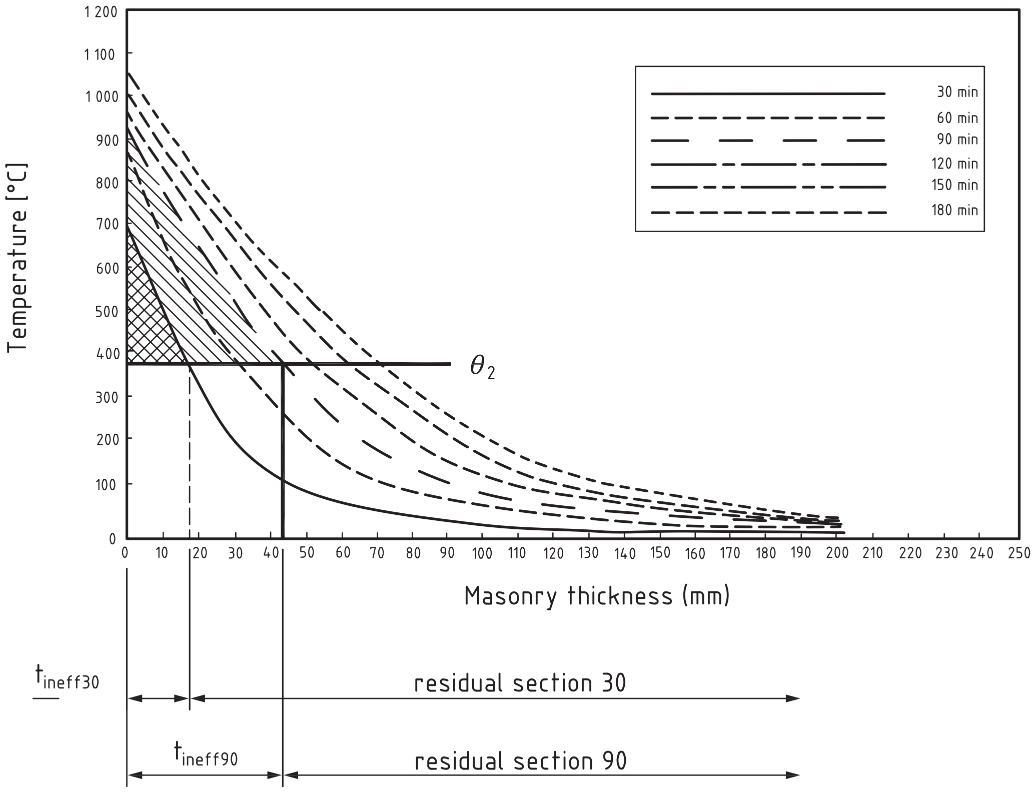
2 Isotherm for $\theta = \theta_2$

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 C.8 which is an example for autoclaved concrete masonry.

Figure C.8 — *Temperature gradient through autoclaved concrete masonry with a density of 400 kg/m³ to 800 kg/m³*



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

C.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.

C.6 Thermal response of aluminium

C.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.

C.6.2 Empirical data based upon fire test results

There is little information on the performance of aluminium loadbearing members in fire and, therefore, it is advisable to refer to individual manufacturers.

C.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 are to be discounted in deriving the surface area exposed to fire.

C.7 Thermal response of glass

C.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.

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.

C.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.

C.8 Thermal response of plastics

C.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. It is recommended that selection is always carried out in consultation with the manufacturer/supplier.

C.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, which has the highest strength;
- C glass, which has good chemical resistance but is not as strong as E glass;
- ECR glass, which is boron-free glass with similar properties to E glass;
- carbon fibres, which have a wide range of properties in strength and stiffness;
- aramid, which are organic fibres that include Kevlar®[1].

C.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:

a) polyester resin: a general purpose thermosetting orthophthalic resin, which has a good combination of mechanical properties and moderate elevated temperature performance:

   1) isophthalic acid (IPA);

   2) bisphenol-A (BPA);

   3) chlorendic;

1 Kevlar is the trademark of a product supplied by DuPont. This information is given for the convenience of users of this document and does not constitute an endorsement by BSI of the product named. Equivalent products may be used if they can be shown to lead to the same results.

b) vinyl ester resin: thermosetting resin derived from the components of polyester and urethane resins:

   1) bisphenol-A;

   2) vovalic;

c) modified acrylic resins: thermosetting resins which can have good flammability characteristics;

d) phenolic resins: most suitable where heat is a primary consideration, such as fire resistance;

e) 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.

C.8.4 Empirical data based upon fire test results

Data on the behaviour of fibre composites is held by individual manufacturers.

C.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:

$$C_{\mathrm{com}} = (C_{\mathrm{f}}\rho_{\mathrm{f}}V_{\mathrm{f}} + C_{\mathrm{x}}\rho_{\mathrm{x}}V_{\mathrm{x}}) / (\rho_{\mathrm{f}}V_{\mathrm{f}} + \rho_{\mathrm{x}}V_{\mathrm{x}}) \tag{C.17}$$

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:

$$1 / k_{\mathrm{com}} = V_{\mathrm{f}} / k_{\mathrm{f}} + V_{\mathrm{x}} / k_{\mathrm{x}} \tag{C.18}$$

Annex D (informative) Temperature-dependent properties of non-loadbearing construction systems — Thermal properties of materials used in composite sandwich panels

D.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 needs to be given to where and how panels are used.

The following thermal data might be helpful, but specific formulations can have slightly different performances.

D.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 D.1.

Table D.1 — *Comparison of expansion of materials used in composite sandwich panels*

Material	**Expansion × 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

D.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 D.2.

Table D.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

D.4 Thermal conductivity

D.4.1 General

Data on the thermal conductivity of materials used in composite sandwich panels are given in D.4.2 to D.4.8.

D.4.2 Mineral (rock) wool — typical values

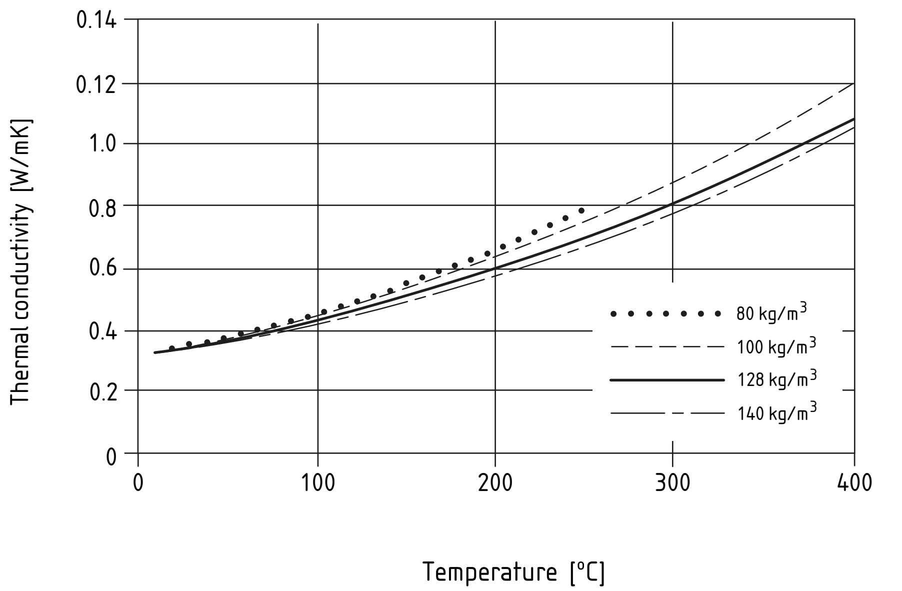
The thermal conductivity of mineral wool at elevated temperatures is given in Table D.3 for various densities.

Table D.3 — *Thermal conductivity for various densities of mineral (rock) wool at elevated temperatures*

Mean temperature	Thermal conductivity, W/mK, at densities of:			
°C	80 kg/m³	100 kg/m³	128 kg/m³	140 kg/m³
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 D.3 is presented graphically in Figure D.1.

Figure D.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:

$$\lambda_i = a_0 + a_1 T_i + a_2 T_i^2 \qquad \text{(D.1)}$$

where:

a_0, a_1, a_2 are constants for a particular product and product density as given in Table D.4;

T_i is the mean temperature of the insulation.

Table D.4 — *Constants for calculating the thermal conductivity of mineral wool at elevated temperatures*

Density	**Constants**		
kg/m³	$a_0 \times 10^{-3}$	$a_1 \times 10^{-6}$	$a_2 \times 10^{-8}$
80	32.04	96.87	36.45
100	32.05	89.59	33.41
128	31.88	96.41	24.23

D.4.3 Cellular glass

The thermal conductivity for two densities of cellular glass is given in Table D.5.

Table D.5 — *Thermal conductivity of cellular glass*

Mean temperature	**Thermal conductivity, W/mK, at densities of:**	
°C	120 kg/m³	135 kg/m³
0	0.038	0.044
10	0.040	0.046

D.4.4 Expanded polystyrene

The thermal conductivity for various densities of expanded polystyrene is given in Table D.6.

Table D.6 — *Thermal conductivity of expanded polystyrene*

Mean temperature	Thermal conductivity, W/mK, at densities of:			
°C	15 kg/m³	20 kg/m³	25 kg/m³	30 kg/m³
10	0.038	0.035	0.033	0.033

D.4.5 Extruded polystyrene

The thermal conductivity for various densities of extruded polystyrene is given in Table D.7.

Table D.7 — *Thermal conductivity of extruded polystyrene*

Mean temperature	Thermal conductivity, W/mK, at densities of:			
°C	28 kg/m³	32 kg/m³	38 kg/m³	45 kg/m³
10	0.033	0.028	0.025	0.026

D.4.6 Phenolic foam

The thermal conductivity for various densities of phenolic foam is given in Table D.8.

Table D.8 — *Thermal conductivity of phenolic foam*

Mean temperature	Thermal conductivity, W/mK, at densities of:			
°C	35 kg/m³	40 kg/m³	60 kg/m³	120 kg/m³
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	—

D.4.7 Polyisocyanate foam

The thermal conductivity for various densities of polyisocyanate foam is given in Table D.9.

Table D.9 — *Thermal conductivity of polyisocyanate foam*

Mean temperature	Thermal conductivity, W/mK, at densities of:		
°C	32 kg/m³	40 kg/m³	50 kg/m³
0	0.021	0.021	0.021
20	0.023	0.023	0.023
50	0.026	0.026	0.026

D.4.8 Rigid polyurethane foam

The thermal conductivity for various densities of rigid polyurethane foam is given in Table D.10.

Table D.10 — *Thermal conductivity of rigid polyurethane foam*

Mean temperature	Thermal conductivity, W/mK, at densities of:		
°C	35 kg/m³	40 kg/m³	50 kg/m³
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:

$$\text{Total heat transfer} = G + S + R + J \qquad \text{(D.2)}$$

where:

G is the heat transfer via conduction through the cell gas;

S is the heat transfer via conduction through the solid phase;

R is the heat transfer via radiation across the cells;

J is the heat transfer via convection through the cell gas.

At a temperature of 10 °C these values are typically:

S = 0.004 W/mK to 0.006 W/mK;

R = 0.004 W/mK 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 D.11.

Table D.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

D.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 D.12.

Table D.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 E (informative) Structural response of specific materials

E.1 Concrete

E.1.1 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 needs to either be shown that the likelihood of spalling is negligible or the analysis method adopted needs to 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 needs to 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 need to be carefully considered for high strength concretes, i.e. cube strengths in excess of 60 MPa. A summary of spalling is given in Connolly [47].

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.

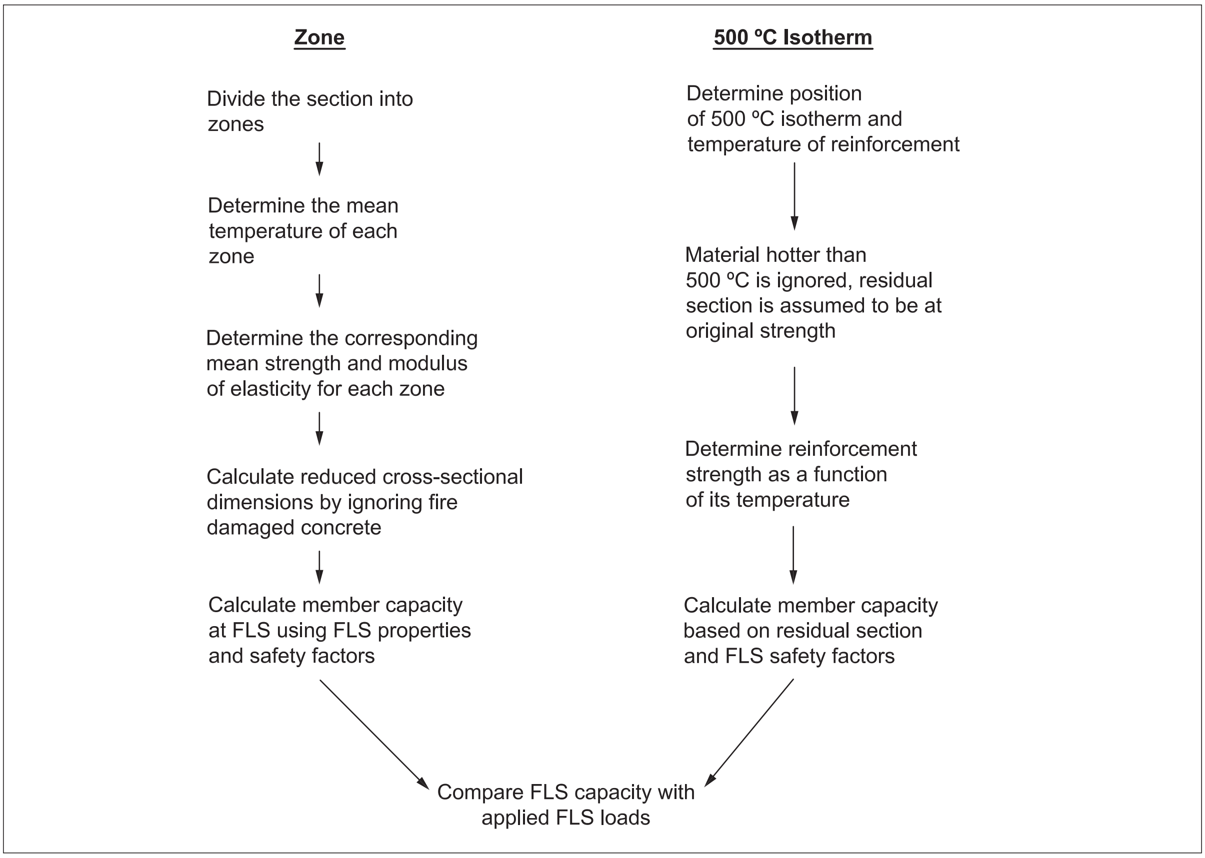
E.1.2 Empirical data from testing

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.

E.1.3 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 E.1.

Figure E.1 — *Design methods for fire limit state (FLS) design adopted in BS EN 1992-1-2*



Second order effects can be included in both models. The two methods are applicable to structures subjected to any fire exposure.

E.1.4 Advanced calculation methods

Advanced calculation 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; and
- 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. Appropriate sensitivity studies need to be conducted to mitigate the unreliability of concrete properties at elevated temperature.

E.2 Steel and cast and wrought iron

E.2.1 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 C.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-dependent 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.

E.2.2 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.

E.2.3 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 designers with simplistic calculation models for members in pure bending, compression and tension, and combined bending and compression (see Figure E.2).

Figure E.2 — *Principle design methodologies adopted in BS EN 1993-1-2*

BS EN 1993-1-2 - moment capacity

Calculate element temperature

↓

Determine the corresponding material reduced strength and elasticity

↓

Calculate member capacity at FLS using the reduced strength and elasticity of the material and FLS safety factors

↓

Compare FLS capacity with applied FLS

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.

E.2.4 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:

$$N_{\mathrm{b,fi,t,Rd}} = \chi_{fi} A k_{\mathrm{y},,} f_{\mathrm{y}} / \gamma_{\mathrm{M,fi}} \tag{E.1}$$

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, Clause 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 $\chi_{\mathrm{y,fi}}$ and $\chi_{\mathrm{z,fi}}$ determined according to:

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

with:

$$\varphi_\theta = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right] \tag{E.3}$$

and:

$$\alpha = 0.65\sqrt{235 / f_{\mathrm{y}}} \tag{E.4}$$

The non-dimensional slenderness $\bar{\lambda}_\theta$ for the temperature θ_a is given by:

$$\bar{\lambda}_\theta = \bar{\lambda}\left[k_{y,\theta} / k_{E,\theta}\right]^{0.5} \tag{E.5}$$

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.

E.2.5 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 needs to be established and compared with the design temperature. This enables the determination of the steel member's capacity to sustain its function. 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 often 50 °C to 100 °C below the equivalent values for hot rolled sections. The relationship between the strength reduction of cold formed steel and the steel temperature is given in BS EN 1993-1-2:2005, Table E.1.

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.

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 always be appropriate and need to 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 needs to be allocated partial safety factors.

The load ratio for columns exposed on up to four sides is determined as follows.

a) For columns in simple construction designed in accordance with BS EN 1993-1-2:

$$R = \frac{F_{\mathrm{f}}}{A_{\mathrm{g}} p_{\mathrm{c}}} + \frac{M_{\mathrm{fx}}}{M_{\mathrm{b}}} + \frac{M_{\mathrm{fy}}}{p_{\mathrm{y}} Z_{\mathrm{y}}} \tag{E.6}$$

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:

$$R = \frac{F_f}{A_g p_y} + \frac{M_{fx}}{M_{cx}} + \frac{M_{fy}}{M_{cy}} \quad \text{or} \tag{E.7}$$

$$R = \frac{F_f}{A_g p_c} + \frac{mM_{fx}}{M_b} + \frac{mM_{fy}}{p_y Z_y} \tag{E.8}$$

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 e.g. a typical compression member with differing slenderness.

E.2.6 Advanced calculation methods

The effects of thermally induced strains and stresses due to temperature rise and temperature differentials need to be considered.

In addition, the model for mechanical response needs to 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; 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 BS EN 1993-1-2 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 needs to be limited to ensure that compatibility is maintained between all parts of the structure.

The design has to 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, it is recommended a sinusoidal initial imperfection with a maximum value of $h/1\,000$ at mid-height is used when not specified by the relevant product standards.

E.2.7 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.

E.3 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 [48].

E.4 Concrete and steel composite floors, beams and columns

E.4.1 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 E.4.2, E.4.3 and E.4.4 assume that the composite section comprises steel and concrete, but could be suitable for other materials.

In all cases, the sections need to be designed to act compositely and, where necessary, the mechanical connection between the different materials maintained throughout the fire.

E.4.2 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 also provided in BS EN 1994-1-2.

E.4.3 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 to 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 needs to be ensured that:

- the difference in rates of thermal expansion is negligible; or
- different rates of thermal expansion do not have an adverse effect 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).

E.4.4 Advanced calculation methods

The mechanical response model need to 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 BS EN 1994-1-2 are used, the effect of high temperature creep need not be explicitly considered.

The deformations at ultimate limit state given by the selected advanced calculation model need to be limited as necessary to ensure that compatibility is maintained between all parts of the structure.

E.5 Composite floors systems — sub-frames

E.5.1 General

E.5 relates to composite steel-framed buildings consisting of steel beams acting in combination with a composite concrete and metal deck floor system.

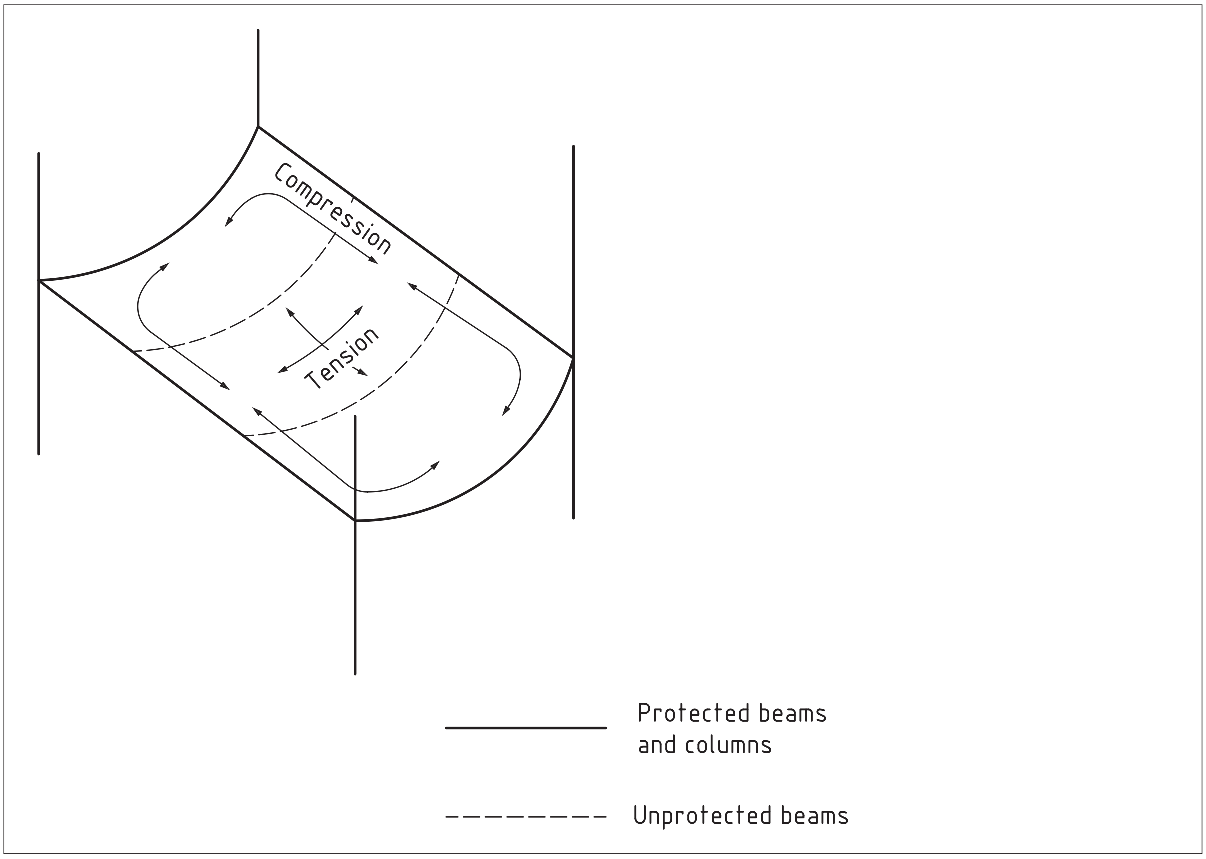
In 1990, a serious fire occurred in the Broadgate Phase 8 Building in London. The building was still under construction and the fireproofing had not yet been applied to the steel frame. The fire spread over a significant area of the floors, yet the structure remained stable and repairs were made. Traditional 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 needs to 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 E.3).

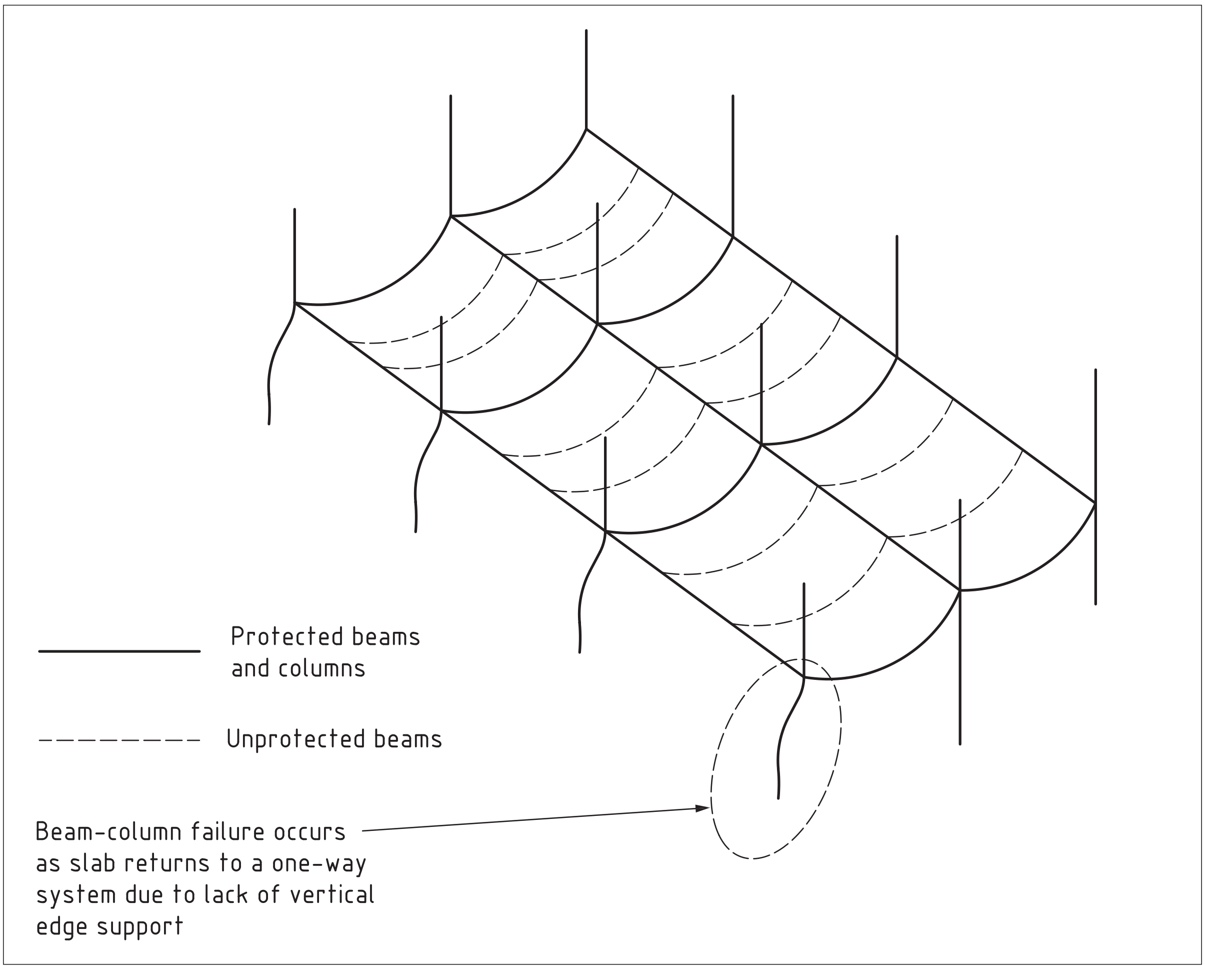
Figure E.3 — *Schematic representation of the compressive and tensile forces of a floor zone during fire*



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.

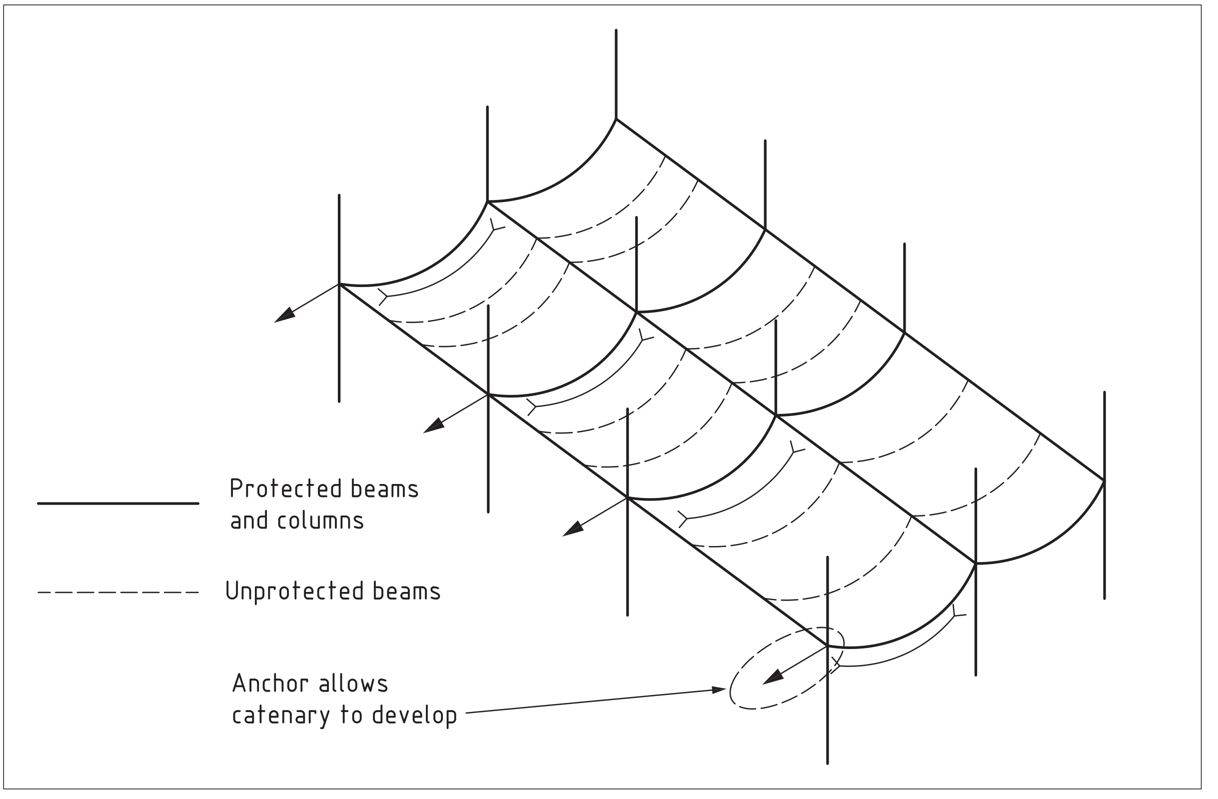
As the temperature of the exposed beams increases, they begin to lose strength and deflect. At this point, the performance of the slab to which they are attached plays an increasingly important role in supporting the floor loads. The characteristics of the slab, together with the way it is supported, 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 E.4).

Figure E.4 — *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 E.5).

Figure E.5 — *Illustration of catenary action developed in a multi–zone composite floor system*



There are different methods available for determining the effects of tensile membrane action. Whichever method is adopted a number of checks need to be made, for example, the capacity of the slab to support the applied load under tensile membrane action. 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 also needs to 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.

If the floor is a compartment floor its integrity needs to 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.

E.5.2 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 needs to ensure that this additional capacity is realistic for slabs with a high amount of reinforcement.

E.5.3 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 needs to 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 needs to ensure that this additional capacity is realistic for slabs with a high amount of reinforcement.

E.5.4 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.

Guidance for conducting finite element analysis is given in **10.3.5.3**.

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 plane sections remain plane. Therefore, effects such as localization and bonding or debonding cannot be predicted, so reinforcement fracture cannot be predicted. As a result, the designer needs to ensure that:

- local behaviour of reinforcement is not important in the context of the analyses;
- local failures of the slab and/or reinforcement do not occur; and
- appropriate acceptance criteria are imposed to protect against local failures.

E.6 Aluminium alloy loadbearing elements

Guidance on the design of aluminium 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 need to be appropriate. If protective coatings are employed, the dry film thicknesses recommended by manufacturers need to be 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 need to be confirmed. Aluminium is widely used in applications where hydrocarbon fire conditions ought to be assumed, e.g. off-shore drilling platforms.

E.7 Timber

E.7.1 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. 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 (arrises); 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. For laminated members the potential for delamination might need to be assessed and a fire-resisting adhesive might be necessary. The guidance provided in the BS EN 1995-1-2:2004 was not developed specifically for CLT members and therefore might not be appropriate. Future revisions of the standard propose to include CLT members specifically. The designer ought to develop appropriate engineered solutions where necessary.

E.7.2 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:

$$t_{\mathrm{fr}} = 0.1 f b_{\mathrm{t}} \left(4 - \frac{b_{\mathrm{t}}}{d_{\mathrm{t}}} \right) \tag{E.9}$$

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).

Formula (E.9) 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 E.1.

Table E.1 — *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 mm	Depth of char in 60 min mm
(a) All structural species with a density greater than 420 kg/m³ 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/m³ at 18% moisture content	15	30

E.7.3 Simplistic calculation methods

In BS EN 1995-1-2 the design mechanical resistance of timber for strength and stiffness is determined from the following:

$$f_{\mathrm{d,fi}} = k_{\mathrm{mod,fi}} \frac{f_{20}}{\gamma_{\mathrm{M,fi}}} \tag{E.10}$$

$$S_{d,fi} = k_{\mathrm{mod},fi} \frac{S_{20}}{\gamma_{M,fi}} \tag{E.11}$$

where:

$f_{\mathrm{d,fi}}$ is the design strength in fire;
$S_{\mathrm{d,fi}}$ 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:

$$R_{d,fi} = \eta \frac{R_{20}}{\gamma_{M,fi}} \qquad (E.12)$$

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:

$$f_{20} = k_{fi} f_k \qquad (E.13)$$

$$S_{20} = k_{fi} S_{05} \qquad (E.14)$$

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 E.2 for different components/elements.

Table E.2 — *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:

$$R_{20} = k_{fi} R_k \qquad (E.15)$$

where:

k_{fi} is taken from Table E.2;

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 formula (see Figure E.6A):

$$d_{\mathrm{ef}} = d_{\mathrm{char,n}} + k_0 d_0 \tag{E.16}$$

where:

d_0 = 7 mm;

$d_{char,n}$ is the depth of char and is determined according to BS EN 1995-1-2

k_0 is as follows:

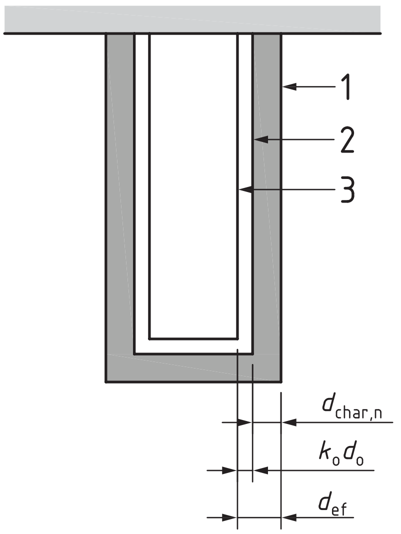
- for unprotected surfaces, k_0 is as given in Table E.3 (see Figure E.6B);
- for protected surfaces where time to start of charring (t_{ch}) > 20 min, k_0 varies linearly from 0 to 1 during the time interval $t = 0$ to $t = t_{ch}$ (see Figure E.6C);
- for protected surfaces where $t_{ch} \leq 20$ min, k_0 is as given in Table E.3 (see Figure E.6B):

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 E.6A — *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 E.6B — *Relationship between k_0 and time of fire exposure for unprotected surfaces, and for protected surfaces where $t_{ch} \leq 20$ min*

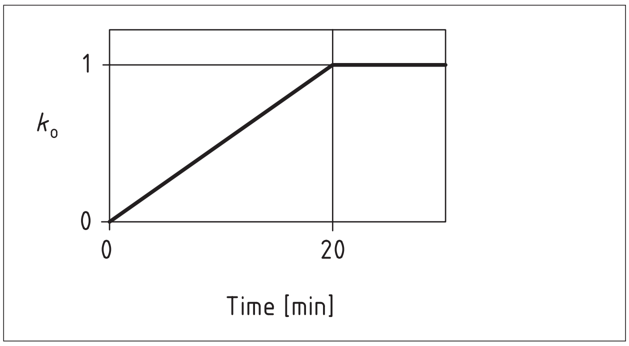


Figure E.6C — *Relationship between k_0 and time of fire exposure for protected surfaces where t_{ch} >20 min*

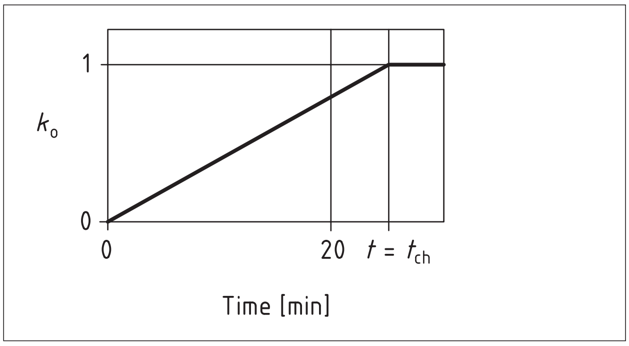


Table E.3 — *Determination of k_0*

t min	$\boldsymbol{k_0}$
<20	t/20
≥20	1.0

The design strength and stiffness properties of the effective cross-section is calculated with $k_{\text{mod,fi}} = 1.0$.

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 $k_{\text{mod,fi}}$ is modified for the following parameters:

For strength in bending:

$$k_{\text{mod,fi}} = 1.0 - \frac{1}{200}\frac{p}{A_{\text{r}}} \tag{E.17}$$

For strength in compression:

$$k_{\text{mod,fi}} = 1.0 - \frac{1}{125}\frac{p}{A_{\text{r}}} \tag{E.18}$$

For strength in tension and elastic modulus:

$$k_{\text{mod,fi}} = 1.0 - \frac{1}{330}\frac{p}{A_{\text{r}}} \tag{E.19}$$

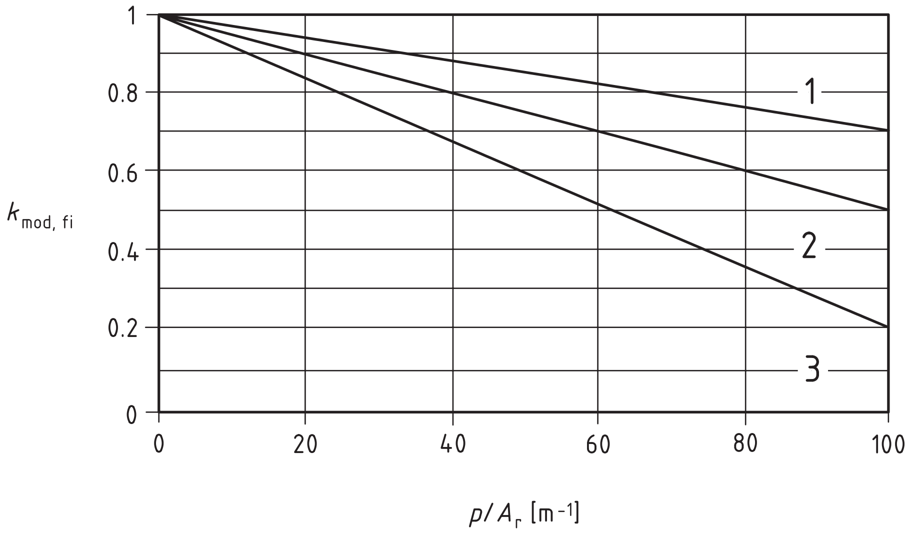
where:

p is the perimeter of the fire exposed residual cross section, m;

A_r is the area of the residual cross section, m².

Formula (E.17) to (E.19) are illustrated in Figure E.7.

Figure E.7 — *Equations (E.17) to (E.19) illustrated*



Key

1 Tensile strength, elastic modulus

2 Bending strength

3 Compressive strength

E.7.4 Advanced calculations for thermal response

The deformation at ultimate limit state, as implied by the calculation methods, needs to be limited as necessary to ensure that compatibility is maintained between all parts of the structure. Where relevant, the mechanical response of the model needs to also take account of geometrical non-linear effects. In the analysis of individual members or sub-assemblies the boundary conditions needs to be checked and detailed in order to avoid failure due to the loss of adequate support for the members.

It needs to be verified that:

$$E_{\text{fi,d}}(t) \leq R_{\text{fi,t,d}} \tag{E.20}$$

where:

$E_{\text{fi,d}}$ 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;

$R_{\text{fi,t,d}}$ 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-dependent material properties, e.g. stiffness and the effect of thermal strain and deformation, need to be assessed.

E.8 Masonry

E.8.1 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.

E.8.2 Empirical data from testing

Careful consideration needs to 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 are 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 than 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.

E.8.3 Simplistic calculation methods

E.8.3.1 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 [50]:

$$\Delta_{\text{head}} = \frac{\alpha h^2 (T_{\text{exp}} - T_{\text{unexp}})}{2d_{\text{w}}} \qquad \text{(E.21)}$$

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 needs to 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:

$$\Delta_{\text{bow}} = \frac{\alpha h^2 (T_{\text{exp}} - T_{\text{unexp}})}{8 d_{\text{w}}} \qquad \text{(E.22)}$$

Formula (E.21) and (E.22) assume elastic behaviour and linear temperature gradients. The formulae 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. Formula (E.22) 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.

E.8.3.2 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 needs to be less than or equal to the design value of the vertical resistance of the wall or column such that:

$$N_{Ed} \le N_{Rd,fi\theta 2} \qquad \text{(E.23)}$$

The design value of the vertical resistance of the wall or column is given by:

$$N_{Rd,fi\theta 2} = \Phi (f_{d\theta 1} A_{\theta 1} + f_{d\theta 2} A_{\theta 2}) \qquad \text{(E.24)}$$

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;
$N_{Rd,fi\theta 2}$	is the design value of the resistance in fire;
$f_{d\theta 1}$	is the design compressive strength of masonry up to θ_1;
$f_{d\theta 2}$	is the design strength of masonry in compression between θ_1 °C and θ_2 °C, taken as $cf_{d\theta 1}$;
Φ	is the capacity reduction factor in the middle of the wall obtained from BS EN 1996-1-1:2005+A1:2012, **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, needs to 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, Figure C.2 with accompanying table, 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 Formula (E.25), (see also BS EN 1996-1-2:2005).

$$e_{\Delta\theta} = \frac{1}{8} h_{\mathrm{ef}}^2 \frac{\alpha_t(\theta_2 - 20)}{t_{\mathrm{Fr}}} \leq h_{\mathrm{ef}} / 20 \quad \text{(E.25)}$$

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 1996-1-2:2005, **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.

E.8.4 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 needs to also take account of geometrical non-linear effects.

In the analysis of individual members or sub-assemblies, the boundary conditions need to be checked and detailed in order to avoid failure due to the loss of adequate support for the members.

It needs to be verified that:

$$E_{\mathrm{fi,d}}(t) \leq R_{\mathrm{fi,t,d}} \quad \text{(E.26)}$$

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-dependent material properties including stiffness, as well as the effect of thermal strain and deformation (indirect fire impact), need to be assessed.

E.9 Glass

E.9.1 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.

E.9.2 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.

E.9.3 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.

E.9.4 Advanced calculation methods

The deformation at ultimate limit state implied by the calculation methods needs to be limited as necessary to ensure that compatibility is maintained between all parts of the structure.

Where relevant, the mechanical response of the model needs to also take account of geometrical non-linear effects. In the analysis of individual members or sub-assemblies, the boundary conditions need to be checked and detailed in order to avoid failure due to the loss of adequate support for the members.

It needs to be verified that:

$$E_{\mathrm{fi,d}}(t) \leq R_{\mathrm{fi,t,d}} \qquad \text{(E.27)}$$

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-dependent material properties, including stiffness and the effect of thermal strain and deformation (indirect fire impact), need to be assessed.

E.10 Plastics

E.10.1 General

Plastic is a combustible material; the effects of this are not covered in detail in the PD 7974 series. More information can be found in the SFPE Handbook [51].

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.

E.10.2 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.

E.10.3 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 needs to 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:

$$P = \frac{P_{\mathrm{m}}\left[P_{\mathrm{f}} + \zeta P_{\mathrm{m}} + \zeta V_{\mathrm{f}}(P_{\mathrm{f}} - P_{\mathrm{m}})\right]}{\left[P_{\mathrm{f}} + \zeta P_{\mathrm{m}} - V_{\mathrm{f}}(P_{\mathrm{f}} - P_{\mathrm{m}})\right]} \tag{E.28}$$

where:

P is the effective property of the composite (elastic and shear moduli);

P_{m} is the effective property of the matrix (elastic and shear moduli);

P_{f} is the effective property of the fibres (elastic and shear moduli);

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.

E.10.4 Advanced calculation methods

The deformation at ultimate limit state implied by the calculation methods need to be limited as necessary to ensure compatibility is maintained between all parts of the structure. Where relevant, the mechanical response of the model needs to also take account of geometrical non-linear effects.

In the analysis of individual members or sub-assemblies, the boundary conditions need to be checked and detailed to avoid failure due to the loss of adequate support for the members.

It needs to be verified that:

$$E_{\mathrm{fi,d}}(t) \le R_{\mathrm{fi,t,d}} \tag{E.29}$$

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.

Annex F (informative) Fire resistant load bearing structural solutions

F.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.

F.2 Special forms of steel construction

F.2.1 General

The following subclauses provide basic information on the different design philosophies for special forms of steel construction.

F.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 [52]).

F.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 [53]).

F.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.

F.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 [54]).

F.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 [36]). However, the behaviour measured in these tests was specific to the construction. Detailed and careful consideration needs to 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.

F.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 [55].

F.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 [56].

F.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 [57].

F.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 [58].

F.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 G (informative) Methodology for establishing the extended application of fire resistance test results

G.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 cannot 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 may 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 might 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.

G.2 Principles of establishing the field of application

G.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** of the individual EN standards, an extended field of application analysis needs to 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 G.2.2.

G.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); and

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; and

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 needs to consider each variation individually, as appropriate, but the analysis has to 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 [a)2)], in order to compensate for a change in a dimension [b)2)].

G.2.3 Establishing the influence of a variation in a parameter on the performance of the element

G.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 needs to 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 **G.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 effect 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.

G.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 needs to be performed by an expert with knowledge of fire and materials in the hot state. G.4 identifies the major parameters to 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 has to 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 can be varied in respect of the lining:

a) the lining might be as tested but with increased or reduced thickness;

b) the lining fixings might be increased or reduced in number;

c) the lining fixings might have enhanced or reduced resistance to pulling out/through when hot;

d) the boards forming the lining might be larger or smaller, resulting in more or less joints;

e) joints transverse to the studs/joists might be provided with enhanced or reduced support/sealing;

f) the lining might be of the same generic material as tested but with a higher or lower density;

g) the tested lining might 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 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 need to 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 influences need to 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 **G.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 might 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.

G.3 Explanation of the expert analysis process

The following list summarizes the processes to be undertaken by a fire expert preparing 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 can change for each parameter.

e) For each factor, determine the influences on the relevant criteria by calculations, validated rules or expert judgment, as appropriate.

The specific extended application standard for the element under consideration ought to 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 need to 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 needs to be made by means of an expert judgment. As with most applications, the output ought to be the product of at least two experts with the necessary fire and high temperature material response knowledge. The reasoning process needs to be incorporated into the extended application report in a transparent manner.

The extended application standards (under preparation) ought to 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 needs to 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 has to 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 needs to 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 needs to be prepared.

G.4 Contents of the extended application report

The extended application report is to 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 needs to 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 analysed 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 ought to 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 needs to state the revised field of extended application resulting from this.

The fire resistance rating and the field of extended application of the varied construction ought to be expressed in the report without ambiguity.

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