This series of Designers’ Guides to the Eurocodes provides comprehensive guidance in the form of design aids, indications for the most convenient design procedures and worked examples. The books also include background information to aid the designer in understanding the reasoning behind and the objectives of the code. All individual guides work in conjunction with the Designers’ Guide to EN1990 (Eurocode: Basis of structural design).

Designers’ Guide to EN1991-1-2, EN1992-1-2, EN1993-1-2 and EN1994-1-2 differs from the other Eurocode guides available in that it is not concerned with a single design standard. The UK standard for the design of steel structures encompasses the rules for both steelwork alone and for composite steel and concrete construction. The key design procedures for reinforced and prestressed concrete structures are contained in the relevant part of the National code. However, the structural Eurocodes consider steel, composite and concrete construction in isolation and each material therefore has its own corresponding part of the National code. However, the structural Eurocodes consider steel, composite and concrete construction in isolation and each material therefore has its own corresponding part of the National code. However, the structural Eurocodes consider steel, composite and concrete construction in isolation and each material therefore has its own corresponding part of the National code.

The design methodology, as set out in the first parts of the structural Eurocodes, is based on the principles adopted for normal temperature design. One of the aims of this book is to demystify the subject so that it can be readily understood and used by structural engineers used to the underlying principles and assumptions of design for the ambient condition. This present Designers’ Guide provides guidance on the nature of the loading that must first be understood before applying the structural engineering principles set out in the Eurocodes. For this reason the book is meant as a guide to four separate documents: EN1991-1-2, EN1992-1-2, EN1993-1-2 and EN1994-1-2 with reference, where appropriate, to the Eurocode covering basis of design.

This guide is essential reading for:

- Designers
- Consultants
- Code-drafting committees
- Clients
- Structural-design students
- Public authorities

in fact, everyone who will be affected by the Eurocodes.
DESIGNERS' GUIDES TO THE EUROCODES


HANDBOOK FOR THE FIRE DESIGN OF STEEL, COMPOSITE AND CONCRETE STRUCTURES TO THE EUROCODES
Eurocode Designers' Guide Series


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DESIGNERS' GUIDES TO THE EUROCODES


HANDBOOK FOR THE FIRE DESIGN OF STEEL, COMPOSITE AND CONCRETE STRUCTURES TO THE EUROCODES

T. LENNON, D. B. MOORE, Y. C. WANG and C. G. BAILEY

Series editor
H. GULVANESSIAN

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Structural Eurocodes offer the opportunity of harmonized design standards for the European construction market and the rest of the world. To achieve this, the construction industry needs to become acquainted with the Eurocodes so that the maximum advantage can be taken of these opportunities.

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Typeset by Academic + Technical, Bristol
Printed and bound in Great Britain by MPG Books, Bodmin
Preface

Many structural engineers will be unfamiliar with the principles of structural fire engineering design. In recent years a number of specialist consultants have emerged offering fire engineering solutions, largely for prestigious projects where the potential benefits of adopting a fire engineering design approach outweigh the additional design cost to the client. There is a fundamental lack of understanding of the principles of structural fire engineering design. In reality the design methodology, as set out in the fire parts of the structural Eurocodes, is based on the principles adopted for normal temperature design. One of the aims of this book is to demystify the subject so that it can be readily understood and used by structural engineers used to the underlying principles and assumptions of design for the ambient condition.

This book differs from many of the other Eurocode guides available in that it is not concerned with a single design standard. The UK standard for the design of steel structures encompasses the rules for both structural steelwork and for composite steel and concrete construction. The fire design procedures for reinforced and prestressed concrete structures are contained in the relevant part of the National Code. However, the structural Eurocodes consider steel, composite and concrete construction in isolation and each material therefore has its own corresponding fire part. In this case a clause-by-clause examination of the material codes would not be sufficient to allow designers to use these documents. The nature of the loading must first be understood before applying the structural engineering principles set out in the Eurocodes. For this reason the book is meant as a guide to four separate documents – EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2 – with reference where appropriate to the Eurocode covering basis of design.
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CHAPTER 1

Introduction

1.1. Introduction to this handbook
In writing this handbook the authors have been aware that many designers will be unfamiliar
with the principles of structural fire engineering design. In recent years a number of specialist
consultants have emerged offering fire engineering solutions largely for prestigious projects
where the potential benefits of adopting a fire engineering design approach outweigh the
additional design cost to the client. The subject is shrouded in mystery and is viewed by
many engineers as a black art. One of the aims of this book is to demystify the subject so
that it can be readily understood and used by civil and structural engineers familiar with
the underlying principles and assumptions of design for the ambient condition.

This book differs from many other Eurocode guides in that it is not concerned with a single
design standard. The UK standard for the design of steel structures encompasses the design
rules for both structural steelwork and for composite steel and concrete construction, albeit
in different parts.1–3 The fire design procedures for reinforced and prestressed concrete
structures are contained in the relevant parts of the national code, BS 8110.4,5 However,
the structural Eurocodes consider steel, composite and concrete construction in isolation
and each material therefore has its own corresponding fire part. In this case a clause-by-
clause examination of the fire parts of the material codes would not be sufficient to allow
designers to use these documents. The nature of the loading must first be understood
before applying the structural engineering principles set out in the Eurocodes. For this
reason this book is meant as a guide to four separate documents, namely EN 1991 Part
appropriate to the Eurocode covering the basis of design, namely EN 1990.6

The guide will take the form of an introduction to the procedures required to achieve
design solutions for a typical range of structural elements and assemblies. Worked examples
will be included along with the text where appropriate to illustrate the use of the Eurocodes
for specific design scenarios. As a way of setting the scene for those unfamiliar with the basic
principles of structural fire engineering design, the next section provides an overview of
the regulatory framework and a description of the commonly used methods for ensuring
compliance with the regulations in the UK.

1.2. Introduction to structural fire design
In the UK the fire resistance requirements for buildings are specified in the Building Regula-
tions for England and Wales7 with regional differences covered in separate documents for
construction in Scotland8 and Northern Ireland.9 All buildings must meet certain functional
requirements covering means of escape, internal fire spread, external fire spread and access
and facilities for the Fire Service as laid down in the regulations. It is important to note that
the Building Regulations are only intended to ensure reasonable standards of health and
safety for those in and around the building. They are not designed to limit structural damage other than to achieve this aim and they are not designed to minimize financial losses arising from a fire. This has important implications for the fire engineering design of buildings where the requirements of the regulations may not be sufficient to meet the needs of the client.

For present purposes the most important requirement is that dealing with internal fire spread as related to structural elements which states:

*The building shall be designed and constructed so that, in the event of a fire, its stability will be maintained for a reasonable period.*

The Approved Document provides detailed guidance on ways to demonstrate compliance with the functional requirements. However, it is important to emphasize that it is the requirement which is mandatory NOT the guidance. This allows for alternative approaches to meeting the requirements, which can be developed in collaboration with the regulatory authorities. In general, the most common route to demonstrate compliance has been to follow the guidance in the Approved Document. The development of the Eurocodes provides for alternative methods for satisfying the regulatory requirement through performance-based calculation.

For steel and concrete members both strength and stiffness decrease with increasing temperature and this reduction is particularly significant between 400°C and 700°C. The most common method of designing steel structures for the fire condition is to design the building for the ambient temperature loading condition and then to cover the steel members with proprietary fire protection materials to ensure that a specific temperature is not exceeded or, put another way, that a specified percentage of the ambient temperature loading capacity is maintained. For concrete structures the required performance in fire is usually achieved by reference to tabulated values for minimum dimensions and minimum cover to the reinforcement. For concrete structures the dimensions adopted for the ambient temperature condition and for durability will often be sufficient to achieve the required fire resistance.

The requirement for the building to maintain stability for a reasonable period has traditionally been related to a required time for survival in a standard fire test. The detailed drawbacks and advantages of the standard test are discussed in Section 3.3 of this handbook. The fire resistance requirements contained in the guidance to the Approved Document relate directly to fire resistance time and it is often incorrectly assumed that there is a one-to-one relationship between survival in a fire resistance test and survival in a fire. This is clearly not the case. Real fires may be more or less severe (either in the time or temperature domain) than the standard fire curve, depending on the particular characteristics of the fire enclosure. The traditional design criterion is that the fire resistance is greater than the time required by the regulations based on the assessment of the building as belonging to a particular purpose group. Fire resistance is defined relative to three failure criteria: insulation, integrity and load-bearing capacity. These terms are explained in greater detail in Chapter 3. This is the method by which the vast majority of buildings are designed. The prescriptive nature of the regulations has hindered the development of a more rational approach to the design of buildings for fire.

As mentioned above, the traditional means of achieving specified periods of fire resistance for steel-framed buildings is to apply passive fire protection to the structural elements. This passive fire protection may be in the form of traditional construction materials such as concrete or brickwork. Up until the late 1970s, concrete was the most common form of fire protection for structural steelwork. However, the high cost of this form of protection together with the problem of spalling in fires led to the development of alternative methods. More frequently, insulation is provided by spray or board fire protection or some combination of the two. Intumescent coatings may be preferred to the more traditional methods. Sprayed systems are popular where the steelwork is not visible, such as floor soffits hidden by a suspended ceiling. Board protection is preferred where the protection is to be left exposed. In modern steel-framed offices the most common form of protection is to spray the beams and to protect the columns with boards. A useful source of information on commonly used fire protection materials is ‘The Yellow Book’ published jointly by the Association for
Specialist Fire Protection and the Steel Construction Institute (SCI). The Yellow Book provides information on the thickness of protection for specified periods of fire resistance.

A major change in the design methodology for steel structures in fire came about with the publication in 1990 of BS 5950 Part 8\textsuperscript{12} (subsequently revised in 2003\textsuperscript{13}). Although this code is still based on an evaluation of the performance of structural steel and composite members in the standard fire test, it allows architects and engineers an alternative approach of designing for fire resistance by calculation. Unlike the Approved Document, the code is concerned only with restricting the spread of fire and minimizing the risk of structural collapse. It recognizes that there is no single ‘failure temperature’ for steel members and that structural failure is influenced not only by temperature but also by load level, support conditions and the presence or otherwise of a thermal gradient through and/or along the member. The code allows for consideration of natural fires but does not provide any detailed information or guidance. Load factors and material strength factors specific to the fire limit state are given. These are partial safety factors which deal with the uncertainties inherent in probabilistic distributions for loading and material properties and represent reductions from ambient temperature design in recognition of the small probability of excessive loads being present at the same time as a fire occurs.

The most common method of achieving the specified fire resistance for steel structures remains the application of passive fire protection. The thickness of fire protection is derived from a consideration of the Section Factor \((H_p/A \text{ or } A_m/V)\). This is the ratio of the heated perimeter to the gross cross-sectional area to allow for the varying rates at which different steel sections heat up during a fire. The thickness of fire protection is then selected by reference to The Yellow Book for specified periods of fire resistance and section factors or, alternatively, calculated according to the formula given in Appendix D of BS 5950 Part 8\textsuperscript{13}.

Calculation methods included in BS 5950 Part 8 include the limiting temperature method. This simple but effective procedure uses the concept of load ratio – that is, the ratio of the load carried during the fire to the ambient temperature load capacity – to derive a limiting temperature, which is then compared with the design temperature to assess the need for passive protection. The design temperature may be determined either from tests or from tabulated data published in the code. A reduction in this value is allowed for I or H sections with low aspect \((D/B)\) ratios to account for shielding effects. It should be borne in mind that the limiting temperature method is not applicable to beams with high shear load. This method utilizes the reduced load factors for the fire limit state.

An alternative option is to use the moment capacity method. This method cannot be used for slender sections. It is not widely used, as knowledge of the temperature profile of the beam is required. The moment capacity is based on the known temperature of the critical element with the relevant strength reduction factor used. If the moment capacity does not exceed that applied at the fire limit state then the beam does not require protection.

Composite slabs are included in the code through the use of simple look-up tables for the critical dimensions and temperature data, which can be incorporated in a more extensive fire engineering analysis. A useful flow chart is included in the original version of the code, summarizing the options and procedures available; however, this was removed from the 2003 revision.

The SCI has produced an informative handbook\textsuperscript{14} to be used in conjunction with the code. This includes tabulated values of section factors for commonly used sections and methods of protection as well as separate chapters dealing with the limiting temperature and moment capacity methods of calculation. It also provides design examples to illustrate the use of the code.

The fire provisions for concrete structures are contained in BS 8110 Part 2\textsuperscript{5} and are based on the results from standard tests\textsuperscript{15}. The current code provisions in BS 8110 refer to data published in \textit{Guidelines for the Construction of Fire Resisting Structural Elements}\textsuperscript{10} and include variation in requirements depending on whether normal-weight or lightweight concrete is used and, for beams and floors, a recognition of the beneficial aspects of continuity on fire resistance. The tabulated provisions for minimum dimensions and minimum cover...
are based around the need to limit the temperature rise on the unexposed face and to maintain stability for a reasonable period. This is achieved by providing a sufficient depth of concrete to limit the temperature rise on the unexposed face to a mean temperature of 140°C and sufficient cover to limit the temperature rise of the reinforcement to 550°C (450°C for prestressing tendons). In general these provisions have proved to provide an acceptable level of safety based on experience of real fires. The development of the structural Eurocodes has provided an opportunity for UK designers to adopt a performance-based approach to designing concrete structures for the effects of real fires, taking into account the beneficial aspects of whole-building behaviour and the inherent continuity and robustness of properly detailed concrete buildings.

The fire parts of the Eurocode set out a new way of approaching structural fire design. To engineers familiar with BS 5950 Part 8, the design procedures in EN 1993-1-2 will be relatively familiar. However, for concrete designers more familiar with a very simple prescriptive approach to the design of structures for fire, based on the use of simple look-up tables, the new philosophy may appear unduly complex. However, the fire design methodology in the Eurocodes affords designers much greater flexibility in their approach to the subject. The options available range from a simple consideration of isolated member behaviour subject to a standard fire, to a consideration of the physical parameters influencing fire development coupled with an analysis of the entire building. The available options are discussed in more detail in Chapter 2. This rather complex process can effectively be simplified into a three-phase procedure consisting of the characterization of the fire model, a consideration of the temperature distribution within the structure and an assessment of the structural response to the fire. What is different about the European system is that all the information required by the designer is no longer available within a single document. Information on thermal actions for temperature analysis is taken from EN 1991-1-2; the method used to calculate the temperature rise of structural steelwork (either protected or unprotected) is found in EN 1993-1-2 and EN 1994-1-2; values for the temperature of concrete members subject to a standard fire exposure are tabulated in EN 1992-1-2. The design procedures to establish structural resistance are set out in the fire parts of EN 1992, EN 1993 and EN 1994 but the actions (or loads) to be used for the assessment are taken from the relevant parts of EN 1991. The simplified procedure and the relationship between the various European Standards is explained in Chapter 2.


EN 1991-1-2 is the fire part of Eurocode 1. It is intended for use in conjunction with the fire design parts of Eurocodes 2 to 9, which are concerned with design for various materials and special circumstances. A more detailed description of the nature and extent of the structural Eurocodes may be found in an earlier book in this series. EN 1991 Part 1.2 provides general principles and actions for the structural design of buildings and civil engineering works and is only valid if the ambient temperature design is carried out in accordance with the relevant structural Eurocodes.

The temperature–time curves (thermal actions in the code) used for structural analysis may be either nominal or physically based fire models. Typical nominal curves would include the ‘standard’ time–temperature response (ISO 834, BS 476 Part 20, BS EN 1363-1) used to determine fire resistance or the more severe hydrocarbon curve used by the offshore and petroleum industries among others. More information is provided in Chapter 3 Section 3.3. Physically based natural fire models include the parametric approach, time equivalent method and advanced calculations such as zone or field models. More information is given in Chapter 3.

The fire part of the structural Eurocode for the design of steel structures, EN 1993 Part 1.2, in common with BS 5950 Part 8, contains information on steel properties at elevated temperature for use in calculation models to determine resistance at elevated temperature.
However, there are a number of significant differences between the European and National documents.

EN 1993 Part 1.2 is not a stand-alone document. In relation to the calculation of the time–temperature response, the code is designed to be used in conjunction with the fire part of Eurocode 1. The fire part of EN 1993 deals with the design of steel structures for the accidental situation of fire exposure with particular reference to the load-bearing function and only identifies differences from, or supplements to, normal temperature design. It is concerned only with passive (as opposed to active) forms of fire protection. Cold-formed members are also covered by the fire part of EN 1993. Composite construction is not dealt with here but in the corresponding fire part of the composite code, EN 1994. There is a change in emphasis from UK practice in that design by calculation is acknowledged as the preferred approach, with design based on the results from tests presented as an alternative option.

The fire part of the Eurocode for composite steel and concrete structures, EN 1994-1-2, is similar in scope to the corresponding part of the steel code. Material properties at elevated temperature are given for structural steel, reinforcing steel and concrete (see Chapter 6). Tabular data are presented for cross-sectional dimensions and area of reinforcement for a range of composite beams and columns. The use of the tabulated data is restricted to the standard fire exposure.

The fire part of the Eurocode for concrete structures, EN 1992 Part 1.2, deals with the design of concrete structures for the accidental situation of fire exposure and is intended to be used in conjunction with the main part of EN 1992 and the fire part of the Eurocode for Actions. It deals with the avoidance of premature collapse of the structure and the limiting of fire spread beyond the compartment of origin. Design methods and tabulated data are provided for reinforced and prestressed concrete columns, walls (load-bearing and non-load-bearing), tension members, reinforced and prestressed beams, and reinforced and prestressed slabs. EN 1992 Part 1.2 does not cover structures with prestressing by external tendons, shell structures and active fire protection. The fire design standard provides data on material properties at elevated temperature of concrete (normal strength, lightweight and high strength), reinforcing steel and prestressing steel (see Chapter 6). It is important to note that the thermal conductivity for concrete specified in this document differs from the corresponding value in EN 1994. The reasons for this are explained in Chapter 6.

1.4. Distinction between principles and application rules
In common with all Eurocodes, the four documents described above make a distinction between principles and application rules. The principles are general statements and definitions for which there is no alternative. Application rules are generally accepted methods, which follow the principles and satisfy their requirements. It is permissible to use alternatives to the application rules but the designer must demonstrate that the chosen alternative also satisfies the principles with at least the same degree of reliability.

To take an analogy with a document likely to be familiar to readers of this handbook, principles may be compared to the functional requirements of Approved Document B. Application rules could then be viewed in the same light as the prescriptive guidance provided on meeting the requirements. It is possible for designers to develop alternative performance-based solutions to meet the functional requirements but such alternatives must be shown to be at least as reliable as the prescriptive solution. Procedures in current national codes will, in many cases, satisfy the principles in the Eurocodes.

1.5. National annexes and Nationally Determined Parameters
The National standards implementing the Eurocodes will comprise the full text of the Eurocode (including any technical annexes), as published by CEN preceded by a National title page and National Foreword followed by a National Annex (NA). The NA only contains information on those parameters which are left open in the Eurocode for national
choice, known as Nationally Determined Parameters (NDPs), to be used for the design of buildings and civil engineering works to be constructed in the country concerned. A list of the clauses for which national choice is permitted together with the recommended value from the Eurocode and the NA value (where available) is given in Table 1.1 below. For those familiar with the ENV version of the Eurocodes, the National Annex is analogous to the National Application Document and the Nationally Determined Parameters to the boxed values.

Table 1.1. Summary of Nationally Determined Parameters. (Note: At the time of writing, many of these issues have not been finalized. Readers are asked to consult the final National Annexes published by BSI)

<table>
<thead>
<tr>
<th>Clause</th>
<th>Description</th>
<th>Recommended value</th>
<th>National Annex value</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.4(4)</td>
<td>Periods of fire resistance</td>
<td>National regulations of Technical Annex F</td>
<td>National regulations and fire engineering approach</td>
</tr>
<tr>
<td>3.1(10)</td>
<td>Choice of nominal or natural fire models</td>
<td>Either nominal or natural fire models</td>
<td>Either nominal or natural fire models</td>
</tr>
<tr>
<td>3.3.1.1(1)</td>
<td>Simple fire models – calculation of fire load density</td>
<td>Annex E for calculation of fire load density</td>
<td>Simple models allowed – fire load density from BS 7974 PD1</td>
</tr>
<tr>
<td>3.3.1.2(1)</td>
<td>Procedure for calculating the heating conditions for internal members in compartment fires</td>
<td>At least fire load density and ventilation conditions</td>
<td>Use of parametric approach (Technical Annex A) permitted subject to complementary information</td>
</tr>
<tr>
<td>3.3.1.2(2)</td>
<td>Procedures for calculating the heating condition for external members in compartment fires</td>
<td>Use of Annex B</td>
<td>Annex B allowed subject to complementary information</td>
</tr>
<tr>
<td>3.3.1.3(1)</td>
<td>Procedure for calculating the heating condition where fire remains localized</td>
<td>Use of Annex C</td>
<td>Alternative procedure in BS 7974 PD1 to be used</td>
</tr>
<tr>
<td>3.3.2(1)</td>
<td>Procedures for calculating fire load density and heat release using advanced fire models</td>
<td>Calculation of fire load density and heat release using Annex E</td>
<td>Fire load density and heat release from BS 7974</td>
</tr>
<tr>
<td>3.3.2(2)</td>
<td>Selection of advanced fire models</td>
<td>One-zone, two-zone or computational fluid dynamics (CFD) models to be used</td>
<td>One-zone, two-zone or CFD models to be used</td>
</tr>
<tr>
<td>4.2.2(2)</td>
<td>Type of additional actions to be considered</td>
<td>Choice of additional actions</td>
<td>No additional actions to be considered</td>
</tr>
<tr>
<td>4.3.1(2)</td>
<td>Combination rules for actions</td>
<td>Quasi-permanent value $\psi_{2,1}$ recommended</td>
<td>Frequent value $\psi_{1,1}$ to be used</td>
</tr>
<tr>
<td>2.1.3(2)</td>
<td>Temperature rise for decay phase</td>
<td>$\Delta \theta_1 = 200 , K$, $\Delta \theta_2 = 200 , K$</td>
<td>No change</td>
</tr>
<tr>
<td>2.3(2)</td>
<td>Partial factor $\gamma_M$</td>
<td>1.0 thermal and mechanical for all materials</td>
<td>No change</td>
</tr>
<tr>
<td>3.2.3(5)</td>
<td>Parameters for stress–strain relationship of reinforcement at elevated temperature</td>
<td>Choice of Class N or Class X</td>
<td>Class N</td>
</tr>
<tr>
<td>3.2.4(2)</td>
<td>Parameters for the stress–strain relationship of prestressing steel at elevated temperature</td>
<td>None</td>
<td>Class A</td>
</tr>
<tr>
<td>Clause</td>
<td>Description</td>
<td>Recommended value</td>
<td>National Annex value</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>-------------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>3.3.3(1)</td>
<td>Thermal conductivity</td>
<td>Value between upper and lower limit</td>
<td>Lower limit</td>
</tr>
<tr>
<td>4.1(1)P</td>
<td>Use of advanced calculation methods to satisfy 2.4.1(2)P</td>
<td>None</td>
<td>Properly validated advanced calculation methods may be used</td>
</tr>
<tr>
<td>4.5.1(2)</td>
<td>Value of moisture content k% below which spalling is unlikely to occur</td>
<td>3%</td>
<td>No change</td>
</tr>
<tr>
<td>5.2(3)</td>
<td>Reference load level for use of tabulated data ( \eta_b )</td>
<td>0.7</td>
<td>No change</td>
</tr>
<tr>
<td>5.3.1(1)</td>
<td>Tabulated data for unbraced structures</td>
<td>None</td>
<td>None given</td>
</tr>
<tr>
<td>5.3.2(2)</td>
<td>Value of ( e_{\text{max}} )</td>
<td>0.15h (or b)</td>
<td>No change</td>
</tr>
<tr>
<td>5.6.1(1)</td>
<td>Web thickness</td>
<td>Choice of Class WA, WB or WC</td>
<td>No change</td>
</tr>
<tr>
<td>5.7.3(2)</td>
<td>Additional rules on rotation capacity at supports for continuous solid slabs</td>
<td>—</td>
<td>No additional rules</td>
</tr>
<tr>
<td>6.1(5)</td>
<td>Reduction of strength at elevated temperature for high-strength concrete</td>
<td>For C55/67 and C60/75 Class 1 of Table 6.1N, For C70/85 and C80/95 Class 2 of Table 6.1N, For C90/105 Class 3 of Table 6.1N</td>
<td>Normally the recommended classes should be used. Alternative values may be used only if satisfactory test evidence is available</td>
</tr>
<tr>
<td>6.2(2)</td>
<td>Measures to control spalling of high-strength concrete</td>
<td>Methods A, B, C and D</td>
<td>Any single method or combination of methods may be used</td>
</tr>
<tr>
<td>6.3.1(1)</td>
<td>Thermal conductivity for high-strength concrete</td>
<td>Within the limits given in clause 3.3.3</td>
<td>Upper limit</td>
</tr>
<tr>
<td>6.4.2.1(3)</td>
<td>Value of factor ( k )</td>
<td>1.1 for Class 1, 1.3 for Class 2</td>
<td>No change</td>
</tr>
<tr>
<td>6.4.2.2(2)</td>
<td>Value of factor ( k_m )</td>
<td>Table 6.2N</td>
<td>No change</td>
</tr>
<tr>
<td>2.3(1)</td>
<td>Partial factor ( \gamma_{Mk} ) for mechanical properties</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2.3(2)</td>
<td>Partial factor ( \gamma_{Mk} ) for thermal properties</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2.4.2(3)</td>
<td>Reduction factor ( \eta_k )</td>
<td>0.65 or 0.7 for category E</td>
<td>—</td>
</tr>
<tr>
<td>4.1(2)</td>
<td>Choice of design methods for fire resistance</td>
<td>Simplified, calculation, advanced calculation, testing</td>
<td>Any suitable design method is permitted</td>
</tr>
<tr>
<td>4.2.3.6(1)</td>
<td>Choice of critical temperature for Class 4 sections ( \theta_{\text{crit}} )</td>
<td>350°C</td>
<td>350°C</td>
</tr>
<tr>
<td>4.2.4(2)</td>
<td>Critical temperature for carbon steel ( \theta_{\text{cr}} )</td>
<td>Formula in code</td>
<td>Revised version of Table 4.1 based on UK experience</td>
</tr>
</tbody>
</table>
1.6. Definitions and symbols
A number of definitions are common to the four Eurocodes considered in this book. The most important definitions together with the associated symbols and units where appropriate are summarized in Table 1.2 below. The terminology in the Eurocodes will be unfamiliar to UK designers; however, the system adopted is common throughout the Eurocodes and the use of subscripts such as $\theta$ (temperature), $a$ (steel) and $d$ (design) is relatively straightforward. The Eurocodes operate a hierarchical system whereby definitions found in higher versions of the code are generally not repeated. For some of the more common design terms and definitions, reference will therefore be required to the main (Part 1.1) versions of the material codes. Also in some documents, different terms are given for the same thing; for example, the definition for reduced cross-section (EN 1992-1-2) and effective cross-section (EN 1994-1-2) are identical.

### Table 1.1. Continued

<table>
<thead>
<tr>
<th>Clause</th>
<th>Description</th>
<th>Recommended value</th>
<th>National Annex value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1(16)</td>
<td>Inclusion of high-strength concrete design</td>
<td>Use information in EN 1992-1-2</td>
<td>—</td>
</tr>
<tr>
<td>2.1.3(2)</td>
<td>Temperature rise for decay phase</td>
<td>$\Delta \theta_1 = 200 \text{K}, \Delta \theta_2 = 200 \text{K}$</td>
<td>—</td>
</tr>
<tr>
<td>2.3(1)</td>
<td>Design values of mechanical material properties</td>
<td>1.0 for all cases</td>
<td>May be modified in NA for EN 1992-1-2 and EN 1993-1-2</td>
</tr>
<tr>
<td>2.3(2)</td>
<td>Design values for thermal material properties</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2.4.2(3)</td>
<td>Reduction factor $\gamma_h$</td>
<td>0.65 or 0.7 for category E</td>
<td>Dependent on partial factors from loading codes and basis of design</td>
</tr>
<tr>
<td>3.3.2(9)</td>
<td>Thermal conductivity</td>
<td>Value between upper and lower limit</td>
<td>—</td>
</tr>
<tr>
<td>4.1(1)</td>
<td>Use of advanced calculation models</td>
<td>Any suitable design method permitted</td>
<td>Any suitable design method permitted</td>
</tr>
<tr>
<td>4.3.5.1(10)</td>
<td>Buckling length of composite columns</td>
<td>0.5 for intermediate column, 0.7 for top storey</td>
<td>—</td>
</tr>
</tbody>
</table>

### Table 1.2. Definitions from EN 1991-1-2, EN 1992-1-2, EN 1993-1-2 and EN 1994-1-2

<table>
<thead>
<tr>
<th>Parameter/term</th>
<th>Definition</th>
<th>Symbol</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Advanced fire model</td>
<td>Design fire based on mass conservation and energy conservation</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Axis distance</td>
<td>Distance between the axis of the reinforcing bar and the surface of the concrete</td>
<td>$a$</td>
<td>mm</td>
</tr>
<tr>
<td>Box value of section factor</td>
<td>Ratio between the exposed surface area of a notional bounding box to the section and the volume of steel</td>
<td>$(A_m/V)_b$</td>
<td>m$^{-1}$</td>
</tr>
<tr>
<td>Carbon steel</td>
<td>Steel grades referred to in EN 1993, except stainless steels</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Combustion factor</td>
<td>Combustion factor represents the efficiency of combustion, varying between 1 for complete combustion to 0 for combustion completely inhibited</td>
<td>$m$</td>
<td>—</td>
</tr>
<tr>
<td>Parameter/term</td>
<td>Definition</td>
<td>Symbol</td>
<td>Units</td>
</tr>
<tr>
<td>----------------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>--------</td>
<td>---------</td>
</tr>
<tr>
<td>Computational fluid dynamic model</td>
<td>Fire model able to solve numerically the partial differential equations giving, in all points of the compartment, the thermodynamic and aerodynamic variables</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Configuration factor</td>
<td>The fraction of diffusely radiated energy leaving surface A that is incident on surface B</td>
<td>$\Phi$</td>
<td>—</td>
</tr>
<tr>
<td>Convective heat transfer coefficient</td>
<td>Convective heat flux to the member related to the difference between the bulk temperature of gas bordering the relevant surface of the member and the temperature of that member</td>
<td>$\alpha_c$</td>
<td>W/m$^2$K</td>
</tr>
<tr>
<td>Critical temperature of reinforcement</td>
<td>The temperature of reinforcement at which failure of the member in a fire situation is expected to occur at a given steel stress level</td>
<td>$\Theta_{s,cr}$</td>
<td>°C</td>
</tr>
<tr>
<td>Critical temperature of structural steel element</td>
<td>For a given load level, the temperature at which failure is expected to occur in a structural steel element for a uniform temperature distribution</td>
<td>$\Theta_{a,cr}$</td>
<td>°C</td>
</tr>
<tr>
<td>Design fire</td>
<td>Specified fire development assumed for design purposes</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Design fire load density</td>
<td>Fire load density considered for determining thermal actions in fire design; its value makes allowance for uncertainties</td>
<td>$q_{f,d}$ or $q_{t,d}$</td>
<td>MJ/m$^2$</td>
</tr>
<tr>
<td>Design fire scenario</td>
<td>Specific fire scenario for which an analysis will be carried out</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Effective cross-section</td>
<td>Cross-section of the member in structural fire design used in the effective cross-section method. It is obtained by removing parts with zero strength and stiffness</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Effective yield strength</td>
<td>For a given temperature, the stress level at which the stress–strain relationship of steel is truncated to provide a yield plateau</td>
<td>$f_{y,\beta}$</td>
<td>N/mm$^2$</td>
</tr>
<tr>
<td>Emissivity</td>
<td>Equal to absorptivity of a surface, i.e. the ratio between the radiative heat absorbed by a given surface, and that of a black body surface</td>
<td>$\varepsilon$</td>
<td>—</td>
</tr>
<tr>
<td>Equivalent time of fire exposure</td>
<td>Time of exposure to the standard time–temperature curve deemed to have the same heating effect as a real fire in a real compartment</td>
<td>$t_{e,d}$</td>
<td>min</td>
</tr>
<tr>
<td>External fire curve</td>
<td>Nominal time–temperature curve intended for the outside of separating external walls which can be exposed to fire from different parts of the façade, i.e. directly from the inside of the respective fire compartment or from a compartment situated below or adjacent to the respective external wall</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>External member</td>
<td>Structural member located outside the building that can be exposed to fire through openings in the building enclosure</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fire activation risk</td>
<td>Parameter taking into account the probability of ignition, function of the compartment area and the occupancy</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fire compartment</td>
<td>Space within a building, extending over one or several floors, which is enclosed by separating elements such that fire spread beyond the compartment is prevented for a specified fire exposure and for a specified period of time</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fire load</td>
<td>Sum of thermal energies which are released by combustion of all combustible materials in a space</td>
<td>$Q_{hk}$</td>
<td>MJ</td>
</tr>
<tr>
<td>Fire load density</td>
<td>Fire load per unit area related to the floor area $q_f$ or the surface area of the total enclosure, including openings, $q_t$</td>
<td>$q_f$, $q_t$</td>
<td>MJ/m$^2$</td>
</tr>
<tr>
<td>Fire protection material</td>
<td>Any material or combination of materials applied to a structural member for the purpose of increasing its fire resistance</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>
### Table 1.2. Continued

<table>
<thead>
<tr>
<th>Parameter/term</th>
<th>Definition</th>
<th>Symbol</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fire resistance</td>
<td>Ability of a structure or a member to fulfill its required functions (load-bearing function and/or fire-separating function) for a specified load level, for a specified fire exposure and for a specified period of time</td>
<td>$R/E/I$</td>
<td>min</td>
</tr>
<tr>
<td>Fire scenario</td>
<td>Qualitative description of the course of a fire with time, identifying key events that characterize the fire and differentiate it from other possible fires. It typically defines the ignition and fire growth process, the fully developed stage, decay stage together with the building environment and systems that will impact on the course of the fire</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fire wall</td>
<td>A wall separating two spaces (generally two buildings) that is designed for fire resistance and structural stability, and may include resistance to horizontal loading such that, in the case of fire and failure of the structure on one side of the wall, fire spread beyond the wall is avoided</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Flash-over</td>
<td>Simultaneous ignition of all fire loads in a compartment</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Fully developed fire</td>
<td>State of full involvement of all combustible surfaces in a fire within a specified space</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Global structural analysis (for fire)</td>
<td>Analysis of the entire structure, when either the entire structure, or only a part of it, is exposed to fire. Indirect fire actions are considered throughout the structure</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Hydrocarbon fire curve</td>
<td>Nominal time–temperature curve representing the effects of a hydrocarbon fire</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Indirect fire actions</td>
<td>Internal forces and moments caused by thermal expansion</td>
<td>Various</td>
<td>Various</td>
</tr>
<tr>
<td>Insulation ($I$)</td>
<td>Ability of a separating element of building construction when exposed to fire on one side, to restrict the temperature rise of the unexposed face below specified levels</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Integrity ($E$)</td>
<td>Ability of a separating element of building construction, when exposed to fire on one side, to prevent the passage through it of flames and hot gases and to prevent the occurrence of flames on the unexposed side</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Load-bearing function ($R$)</td>
<td>Ability of a structure or member to sustain specified actions during the relevant fire, according to defined criteria</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Localized fire</td>
<td>Fire involving only a limited area of the fire load in the compartment</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Member</td>
<td>Basic part of a structure (such as beam, column, wall, truss) considered as isolated, with appropriate boundary and support conditions</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Member analysis (for fire)</td>
<td>Thermal and mechanical analysis of a structural member exposed to fire in which the member is assumed as isolated, with appropriate support and boundary conditions. Indirect fire actions are not considered except those arising from thermal gradients</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Net heat flux</td>
<td>Energy, per unit time and surface area, definitely absorbed by members</td>
<td>$H_{net}$</td>
<td>W/m²</td>
</tr>
<tr>
<td>Normal temperature design</td>
<td>Ultimate limit state design for ambient temperatures according to Parts 1.1 of EN 1992 to EN 1996 and EN 1999</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>One-zone model</td>
<td>Fire model where homogeneous temperatures of the gas are assumed in the compartment</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Parameter/term</td>
<td>Definition</td>
<td>Symbol</td>
<td>Units</td>
</tr>
<tr>
<td>---------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>--------</td>
<td>--------</td>
</tr>
<tr>
<td>Opening factor</td>
<td>Factor representing the amount of ventilation depending on the area of openings in the compartment walls, on the height of these openings and on the total area of the enclosure surfaces</td>
<td>$O$</td>
<td>$m^{1/2}$</td>
</tr>
<tr>
<td>Part of structure</td>
<td>Isolated part of an entire structure with appropriate support and boundary conditions</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Protective layers</td>
<td>Any material or combination of materials applied to a structural member for the purpose of increasing its fire resistance</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Protected members</td>
<td>Members for which measures are taken to reduce the temperature rise in the member due to fire</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Rate of heat release</td>
<td>Heat (energy) released by a combustible product as a function of time</td>
<td>$Q$</td>
<td>W</td>
</tr>
<tr>
<td>Reduced cross-section</td>
<td>Cross-section of the member in structural fire design used in the reduced cross-section method. It is obtained from the residual cross-section by removing parts with assumed zero strength and stiffness</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Section factor</td>
<td>For a steel member, the ratio between the exposed surface area and the volume of steel; for an enclosed member, the ratio between the internal surface area of the exposed encasement and the volume of steel</td>
<td>$A_m/V$</td>
<td>$m^{1/2}$</td>
</tr>
<tr>
<td>Separating element</td>
<td>Load-bearing or non-load-bearing element forming part of the enclosure of a fire compartment</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Separating function</td>
<td>Ability of a separating element to prevent fire spread or ignition beyond the exposed surface during the relevant fire</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Simple fire model</td>
<td>Design fire based on a limited application field of specific physical parameters</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Stainless steel</td>
<td>All steels referred to in EN 1993-1-4</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Standard fire resistance</td>
<td>Ability of a structure or part of a structure to fulfil required functions for the exposure to heating according to the standard time–temperature curve for a specified load combination and for a specified period of time</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Standard time–temperature curve</td>
<td>Nominal curve defined in EN 13501-1 for representing a model of a fully developed fire in a compartment</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Structural members</td>
<td>Load-bearing members of a structure including bracing</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Temperature analysis</td>
<td>Procedure of determining the temperature development in members on the basis of the thermal actions (net heat flux) and the thermal material properties of the members and of protective surfaces, where relevant</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Thermal actions</td>
<td>Actions on the structure described by the net heat flux to the members</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Two-zone model</td>
<td>Fire model where different zones are defined in a compartment: the upper layer, the lower layer, the fire and its plume, the external gas and walls. In the upper layer, uniform temperature of the gas is assumed</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>
CHAPTER 2

Design methods

2.1. Introduction

In the UK the traditional means of meeting the requirements of the Building Regulations in terms of fire resistance of structures has been to rely on tabulated data related to performance in standard fire tests. The Eurocodes present a range of options for the designer ranging from prescriptive rules based on standard fire resistance periods and the use of tabulated data, to calculation procedures based on a natural fire exposure and whole building behaviour. The extent to which each of these methods can be used with a particular form of construction is dependent on the state of knowledge regarding the material performance in fire and the availability of suitably validated design methods. The general design procedure for the fire limit state indicating the potential routes for compliance with the regulatory requirement is summarized in Fig. 2.1.

Table 2.1 summarizes the alternative methods available in the Eurocodes for the verification of fire resistance for concrete structures. Traditional UK practice generally only considers the most simple element in the matrix.

The hierarchy in terms of complexity is tabulated data followed by simple calculation methods followed by advanced calculation methods. For the designer the tabulated approach should be the first port of call and should be suitable for the vast majority of structures. Calculation methods can be used to demonstrate performance under specific conditions and may provide substantial savings in certain circumstances. Advanced calculation methods (typically non-linear finite-element models) may be used where the structure is very complex and where the provisions of the National regulations are not applicable. Examples of such structures would include sports stadia, exhibition halls or airport terminals.

The Eurocode approach to structural design will be unfamiliar to many UK engineers. However, there are a number of similarities between the approaches adopted. The standard design route will remain the use of tabulated data with reference to specified periods of fire resistance related to the standard fire test. The most significant difference in approach is one which is not restricted to the use of the fire parts of the material codes but is perhaps more pronounced in this area. The information required to carry out structural fire engineering design has traditionally been located within one material code. The structural Eurocodes are an integrated suite of design standards and are meant to be used as such. To carry out a design for concrete structures, for example, using tabulated values from the National standards, the designer needs only refer to the relevant National material codes. For a similar design to the Eurocode it is necessary to obtain partial factors from EN 1990, information on loads from EN 1991-1, information on the thermal and mechanical response from EN 1991-1-2 and finally obtain the required dimensions from EN 1992-1-2. Although the fire design methodology adopted in the Eurocodes is radically different from the procedures generally used in the UK, the end result, in terms of member sizes and cover to reinforcement is, in many cases, similar.
Fig. 2.1. Design procedure
In the fire parts of the various material codes (EN 1992-1-2, EN 1993-1-2, EN 1994-1-2), fire resistance may be determined either by:

- simple calculation models
- advanced calculation models, or
- tabulated data.

The current UK Standards are based on tabulated periods of fire resistance derived from standard fire tests and fire resistance derived from calculations in certain specific cases.

The regulatory requirement is generally specified in National regulations based on the type of occupancy (office, domestic, retail, etc.) and the height of the structure. The design procedure from the Eurocodes indicating the relationship between the various standards required for design is as follows:

- selection of relevant design fire scenario (EN 1991-1-2)
- determination of corresponding design fire (EN 1991-1-2)
- calculation of the temperature rise of the structural members (EN 1992-1-2, EN 1993-1-2, EN 1994-1-2)

The situation is illustrated schematically in Fig. 2.2.

The most significant difference in approach is that load effects both in relation to dead and imposed loading and the time–temperature regime to be used for assessment are not contained within the material code but in the relevant codes for actions on structures.

In the Eurocodes, simple calculation methods are based on an assessment of the deterioration in material properties at elevated temperature together with an assessment of the appropriate load for the fire limit state. The resistance is then calculated based on reduction factors appropriate to the design thermal exposure and compared to the load effects present at the time of the fire. Advanced calculation methods typically involve the use of complex finite-element models and would not in general be available to designers.

### Table 2.1. Alternative methods for verification of fire resistance to EN 1992-1-2

<table>
<thead>
<tr>
<th></th>
<th>Tabulated data: prescriptive methods</th>
<th>Simplified calculation methods</th>
<th>Advanced calculation methods</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Member analysis.</strong> The member is considered as isolated. Indirect fire actions are not considered, except those resulting from thermal gradients</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td>- Data given for standard fire only</td>
<td>- Standard fire and parametric fire</td>
<td>- Only the principles are given</td>
</tr>
<tr>
<td></td>
<td>- In principle, data could be developed for other fire curves</td>
<td>- Temperature profiles given for standard fire only</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Material models apply only to heating rates similar to standard fire</td>
<td></td>
</tr>
<tr>
<td><strong>Analysis of parts of the structure.</strong> Indirect fire actions within the sub-assembly are considered, but no time-dependent interaction with other parts of the structure</td>
<td>NO</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Standard fire and parametric fire</td>
<td>- Only the principles are given</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Temperature profiles given for standard fire only</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Material models apply only to heating rates similar to standard fire</td>
<td></td>
</tr>
<tr>
<td><strong>Global structural analysis.</strong> Analysis of the entire structure. Indirect fire actions are considered throughout the structure</td>
<td>NO</td>
<td>NO</td>
<td>YES</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Only the principles are given</td>
<td></td>
</tr>
</tbody>
</table>

In the fire parts of the various material codes (EN 1992-1-2, EN 1993-1-2, EN 1994-1-2), fire resistance may be determined either by:

- simple calculation models
- advanced calculation models, or
- tabulated data.

The current UK Standards are based on tabulated periods of fire resistance derived from standard fire tests and fire resistance derived from calculations in certain specific cases.

The regulatory requirement is generally specified in National regulations based on the type of occupancy (office, domestic, retail, etc.) and the height of the structure. The design procedure from the Eurocodes indicating the relationship between the various standards required for design is as follows:

- selection of relevant design fire scenario (EN 1991-1-2)
- determination of corresponding design fire (EN 1991-1-2)
- calculation of the temperature rise of the structural members (EN 1992-1-2, EN 1993-1-2, EN 1994-1-2)

The situation is illustrated schematically in Fig. 2.2.

The most significant difference in approach is that load effects both in relation to dead and imposed loading and the time–temperature regime to be used for assessment are not contained within the material code but in the relevant codes for actions on structures.

In the Eurocodes, simple calculation methods are based on an assessment of the deterioration in material properties at elevated temperature together with an assessment of the appropriate load for the fire limit state. The resistance is then calculated based on reduction factors appropriate to the design thermal exposure and compared to the load effects present at the time of the fire. Advanced calculation methods typically involve the use of complex finite-element models and would not in general be available to designers.
2.2. Design of concrete structures to EN 1992-1-2

The Eurocode design procedure for concrete structures subject to fire is classified according to the design method adopted which may be tabulated data, simple calculation models or advanced calculation models. The options available to the designer are summarized in Table 2.2.

<table>
<thead>
<tr>
<th>Thermal model</th>
<th>Structural model</th>
<th>Method of analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal (standard) fire curves</td>
<td>Single element</td>
<td>Tabulated data</td>
</tr>
<tr>
<td>Calculation based on standard curve (time equivalent)</td>
<td>Sub-assembly</td>
<td>Simple calculation models</td>
</tr>
<tr>
<td>Simple calculation based on compartment characteristics</td>
<td>Entire structure</td>
<td>Advanced calculation models (non-linear finite elements)</td>
</tr>
<tr>
<td>(parametric approach)</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Advanced calculation model (computational fluid dynamics)</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

Tabulated data are presented for reinforced concrete columns, walls (both load-bearing and non-load-bearing), beams (including simply supported and continuous, and reinforced and prestressed), and slabs (both solid and ribbed). Simplified calculation methods are presented together with appropriate strength reduction values. Two simple calculation methods, namely the 500°C isotherm method and the zone method, are given in Annex B of the code. General guidance is given on advanced methods for the determination of the temperature profile and the mechanical response of concrete structures.

2.3. Design of steel structures to EN 1993-1-2

The Eurocode design procedure for steel structures subject to fire is classified according to the type of analysis undertaken. The analysis may be undertaken for a single member, a part of the structure or a global analysis of the entire structure. The options available to the designer are summarized in Table 2.3. Simple calculation methods are presented for tension members, compression members, and members in bending. Verification may be carried out in relation to either resistance or temperature.
Table 2.3. Alternative methods for verification of fire resistance to EN 1993-1-2

<table>
<thead>
<tr>
<th>Tabulated data:</th>
<th>Simplified calculation methods</th>
<th>Advanced calculation methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member analysis. The member is considered as isolated. Indirect fire actions are not considered, except those resulting from thermal gradients</td>
<td>NO</td>
<td>YES</td>
</tr>
<tr>
<td><strong>Analysis of parts of the structure.</strong> Indirect fire actions within the sub-assembly are considered, but no time-dependent interaction with other parts of the structure</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td><strong>Global structural analysis.</strong> Analysis of the entire structure. Indirect fire actions are considered throughout the structure</td>
<td>NO</td>
<td>NO</td>
</tr>
</tbody>
</table>

2.4. Design of composite structures to EN 1994-1-2

The Eurocode design procedure for composite steel and concrete structures subject to fire is classified both according to the type of analysis undertaken and the design procedure adopted. The options available to the designer are summarized in Table 2.4. Tabulated
data are presented for composite beams with partial encasement and composite columns (including totally encased steel sections, partially encased steel sections and concrete-filled hollow sections). Simple calculation models are presented for composite slabs and composite beams (including bare steel beams and partially encased steel beams) and for composite columns. General guidance is given on the use of advanced calculation models for the calculation of thermal and mechanical response.

2.5. Design assisted by testing
For all materials considered in this handbook fire design may be based on the results of fire tests as an alternative to design by calculation. Design may be based on a combination of tests and calculations.
CHAPTER 3

Design fires

3.1. Introduction
The first stage in a structural fire engineering design is to define the appropriate fire design scenario. This will typically involve a consideration of a fire in various compartments within a building to establish the most suitable cases for design purposes. The choice of the design fire scenario will dictate the choice of design fire to be used. A determination of the thermal actions to be used in subsequent structural analysis can be achieved either through a prescriptive approach, which relies on data from standard test methods, or from a consideration of the physical parameters specific to a particular building. The former approach is consistent with the current prescriptive methods used by most designers in the UK. The latter method allows for calculation procedures and represents a radical departure from traditional fire design. There are effectively four levels or models, which can be used to determine the thermal exposure. These are described in Table 3.1 in order of increasing complexity, based on a matrix representation by Witteveen.27

To date, level 3 and 4 solutions have been restricted to research projects or complex and innovative structures. The implementation of the Eurocodes will allow a larger number of buildings to be designed according to the calculation procedures in the codes. Regulatory bodies have increasingly been moving away from a prescriptive approach towards a functional methodology whereby the designer is told what must be achieved but not how to meet the functional requirements. This freedom of choice allows the designer a range of options including the use of calculation procedures.

Table 3.1. Assessment methods for thermal exposure

<table>
<thead>
<tr>
<th>Assessment method</th>
<th>Model for thermal exposure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>H1</td>
<td>Standard fire exposure – test or tabulated data</td>
</tr>
<tr>
<td>Level 2</td>
<td>H2</td>
<td>Equivalent fire duration time – relates the severity of a compartment fire to an equivalent period in a standard furnace</td>
</tr>
<tr>
<td>Level 3</td>
<td>H3</td>
<td>Parametric exposure – uses physical characteristics of the fire compartment as input parameters</td>
</tr>
<tr>
<td>Level 4</td>
<td>H4</td>
<td>Advanced methods – zone or field models used to characterize the full compartment response for the required duration</td>
</tr>
</tbody>
</table>

3.2. General rules for calculating atmosphere temperatures
The thermal actions to be used in subsequent analysis may be either nominal, derived from simple calculation, or by advanced methods. The choice of a particular fire design scenario
should be based on a risk assessment taking into account the likely ignition sources and any fire detection/suppression methods available. The design fire should be applied to only one fire compartment at a time.

3.3. Nominal temperature–time curves

The nominal or standard fire curves provide a simple means of assessing building materials and components against a common set of performance criteria subject to a closely defined thermal and mechanical loading under prescribed loading and support conditions. Although notionally a representation of building fires, the standard fire curves do not take into account any of the physical parameters affecting fire growth and development. The nominal curves given in EN 1991-1-2 are described below.

3.3.1. Standard temperature–time curve

The standard fire curve has been used effectively for many years to determine the relative performance of construction materials. The temperature–time relationship is described below and set out in EN 1363.

\[ \Theta_g = 20 + 345 \log_{10}(8t + 1) \]  

where:

- \( \Theta_g \) is the gas temperature in the fire compartment (°C); and
- \( t \) is the time (min).

One crucial shortcoming of this (and the other nominal curves) is that there is no descending branch, i.e. no cooling phase. Large-scale experiments have shown that the cooling phase can be very important with regard to structural performance, particularly where large thermal restraint is present. This standard relationship is the basis for the tabulated data in the codes for steel, concrete and composite construction. Many of the design methods available through the Eurocodes are restricted to the choice of a design fire similar to the standard curve, as there is insufficient information on the thermal and structural performance of members and complete structures subject to natural fire exposures.

3.3.2. External fire curve

The external fire curve is used for structural members in a façade external to the main structure. The external fire curve is given by:

\[ \Theta_g = 660(1 - 0.687e^{-0.32t} - 0.313e^{-3.8t}) + 20 \]  

where \( \Theta_g \) and \( t \) are as defined above.

3.3.3. Hydrocarbon curve

In situations where petrochemicals or plastics form a significant part of the overall fire load, the temperature rise is very rapid due to the much higher calorific values of these materials. Therefore, for such situations, an alternative temperature–time curve has been developed of the form:

\[ \Theta_g = 1080(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) + 20 \]  

The three nominal curves defined in the Eurocodes are illustrated in Fig. 3.1 together with a typical natural fire exposure consisting of an ignition phase, a growth phase, a steady-state phase and a decay phase. A number of other nominal fire curves are used for specific circumstances such as assessing linings for tunnels. These curves are not included in the European standards. More information is available in the literature.
3.4. Equivalent time of fire exposure

EN 1991-1-2 includes a method for determining the appropriate fire resistance period for design based on a consideration of the physical characteristics of the fire compartment. This is effectively a ‘halfway house’ between the nominal curves so familiar to many and the behaviour of a realistic fire compartment. The method relates the severity of a real fire in a real compartment to an equivalent period of exposure in a standard test furnace. The relevant input parameters are the amount of fire load, the compartment size (floor area and height), the thermal properties of the compartment linings and the ventilation conditions. The formulation in the Eurocode (based on fire load density related to floor area) is:

\[ t_{e,d} = (q_{f,d}k_bw_t)k_c \]  

(3.4)

where:

- \( t_{e,d} \) is the equivalent time of fire exposure for design (min);
- \( q_{f,d} \) is the design fire load density (MJ/m\(^2\));
- \( k_b \) is a conversion factor dependent on thermal properties of linings;
- \( w_t \) is the ventilation factor; and
- \( k_c \) is a correction factor dependent on material. (Note: for protected steel and reinforced concrete \( k_c = 1.0 \), where no detailed assessment of the thermal properties is made the factor \( k_b = 0.09 \) (National Annex value).)

The ventilation factor is:

\[ w_t = (6/H)^{0.3}[0.62 + 90(0.4 - \alpha_v)^4] \]

in the absence of horizontal openings (roof lights) in the compartment;

where:

- \( H \) is the height of the fire compartment (m); and
- \( \alpha_v = A_v/A_f \) where \( A_v \) and \( A_f \) are the ventilation and floor area respectively (m\(^2\)).

The verification is then that the fire resistance of the member is greater than the time equivalent value. The concept of time equivalence is illustrated in Fig. 3.2 with respect to the maximum temperature of a structural member and the time taken for that member to achieve an identical temperature in a standard furnace test.

The concept is illustrated with reference to a worked example.

---

Fig. 3.1. Nominal fire curves – comparison with results from natural fire test
Example 3.1: Time equivalent calculation

Calculate the appropriate fire resistance period for a protected steel beam within a small office with boundaries of fire-resistant construction (compartment floors and walls).

Let us consider a small fire compartment within an office building – design parameters. Tables 3.2 to 3.5 below refer.

Table 3.2. Geometric data

<table>
<thead>
<tr>
<th>Description</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor area $A_f$ (m$^2$)</td>
<td>36 (6 m $\times$ 6 m)</td>
</tr>
<tr>
<td>Ventilation area $A_v$ (m$^2$)</td>
<td>7.2 (3.6 m wide $\times$ 2 m high)</td>
</tr>
<tr>
<td>Height of ventilation opening $h$ (m)</td>
<td>2</td>
</tr>
<tr>
<td>Height of compartment $H$ (m)</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 3.3. Material thermal data

<table>
<thead>
<tr>
<th>Element</th>
<th>Material</th>
<th>Thermal inertia ($b$ value – J/m$^2$ s$^{1/2}$/K)</th>
<th>Area (m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>Concrete</td>
<td>2280</td>
<td>36</td>
</tr>
<tr>
<td>Floor</td>
<td>Plasterboard</td>
<td>520</td>
<td>36</td>
</tr>
<tr>
<td>Walls</td>
<td>Plasterboard</td>
<td>520</td>
<td>76.8</td>
</tr>
</tbody>
</table>

Table 3.4. Factor $k_b$ to account for thermal properties of compartment linings

<table>
<thead>
<tr>
<th>Thermal properties</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b = (\rho c \lambda)^{1/2}$ (J/m$^2$ s$^{1/2}$/K)</td>
<td>$k_b$ (min m$^2$/MJ)</td>
</tr>
<tr>
<td>$b &gt; 2500$</td>
<td>0.04 (0.055)</td>
</tr>
<tr>
<td>$720 \leq b \leq 2500$</td>
<td>0.055 (0.07)</td>
</tr>
<tr>
<td>$b &lt; 720$</td>
<td>0.07 (0.09)</td>
</tr>
</tbody>
</table>

Note: UK National Annex values in brackets.
3.5. Parametric temperature–time curves

Along with the time equivalent approach, parametric fires are an example of the simple calculation methods for determining the compartment internal atmosphere time–temperature relationship. The current approach has been modified (in scope) in successive drafts of the fire part of Eurocode 1. The basic formulation has remained largely unchanged however and is based on work carried out by Wickström.\(^{30}\) The parametric approach provides a quick and easy approximation of compartment gas temperatures ideally suited for use on modern spreadsheets. The approach has been extensively validated over a number of years. It applies only to the post-flashover phase, which is of primary concern when considering structural issues and assumes a uniform temperature within the compartment. The basic formulation in Annex A of EN 1991-1-2 is as follows:

\[
\Theta_g = 20 + 1325 \left( 1 - 0.324 e^{-0.2t'} - 0.204 e^{-1.7t'} - 0.472 e^{-19t'} \right)
\]  

(3.5)

where:

- \(\Theta_g\) is the temperature in the fire compartment (°C);
- \(t' = t \Gamma\) (h);
- \(t\) is time (h);
- \(\Gamma = [O/b]^2/(0.04/1160)^2\);
- \(b = \sqrt{\rho c \lambda}\) (J/m\(^2\)s\(^{1/2}\)K);
- \(O = \) opening factor \((A_v \sqrt{h/A_t})\) (m\(^{1/2}\));
- \(A_v\) is the area of vertical openings (m\(^2\));
- \(h\) is the height of vertical openings (m);
- \(A_t\) is the total area of enclosure (m\(^2\));
- \(\rho\) is the density of boundary enclosure (kg/m\(^3\));
- \(c\) is the specific heat of boundary of enclosure (J/kgK); and
- \(\lambda\) is the thermal conductivity of boundary (W/mK).

The temperature within any given compartment is assumed to vary as a simple exponential function of modified (or parametric) time, depending on the variation in the ventilation area.
and the properties of the compartment linings. The values 0.04 and 1160 refer to the opening factor and thermal properties of the compartment used in the development of the approach. A parametric calculation with the same values corresponds to a time–temperature response very similar to the standard fire curve.

In the original draft for development of the Eurocode released for use in the UK with the UK National Application Document there were a number of restrictions on the use of this formula, which greatly limited the scope of application. Most of these have now been removed as validation for the approach has been developed and the method may now be used for most common building types.

The calculation procedure provides the engineer with a rate of temperature rise varying with time. In order to estimate the duration of the fire, the relationship between the fire load and the opening must be considered. The maximum temperature in the heating phase occurs at a time $t_{\text{max}}$ given by:

$$t_{\text{max}} = \text{maximum of } 0.2 \times 10^{-3} \times q_{d, d}/O \text{ or } t_{\lim}$$

where $q_{d,d}$ is the design value of the fire load density related to the total surface area of the enclosure (values of $q_{d,d}$ should be in the range 50–1000 MJ/m$^2$); and $t_{\lim}$ is a minimum value for the duration of the fire based on slow, medium or fast fire growth rates. For office accommodation a medium fire growth rate should be assumed corresponding to a value of $t_{\lim}$ equal to 20 min.

For most practical combinations of fire load, compartment geometry and opening factor $t_{\text{max}}$ will be in excess of the 20-minute limit. The temperature–time curves for the cooling phase are then given by:

$$\Theta_g = \Theta_{\text{max}} - 625(t^* - t_{\text{max}}^*) \quad \text{for } t_{\text{max}}^* \leq 0.5$$

$$\Theta_g = \Theta_{\text{max}} - 250(3 - t_{\text{max}}^*)(t^* - t_{\text{max}}^*) \quad \text{for } t_{\text{max}}^* < 2$$

$$\Theta_g = \Theta_{\text{max}} - 250(t^* - t_{\text{max}}^*) \quad \text{for } t_{\text{max}}^* \geq 2$$

Example 3.2 shows an example of a parametric calculation for a typical office compartment while Fig. 3.3 shows the predicted time–temperature response together with test results indicating the accuracy of the approach. The reader will note the similarities with the calculation procedure for time equivalence discussed above.

![Average compartment time-temperature response](image)

**Fig. 3.3.** Comparison between parametric prediction and test results
Example 3.2: Parametric calculation

A typical fire compartment within an office building has a floor area of 6 m × 6 m and a floor-to-ceiling height of 3.4 m. There is a single window opening in the front elevation 3.6 m wide × 2 m high. Table 3.6 summarizes the geometric parameters required for the calculation of the compartment time–temperature response.

<table>
<thead>
<tr>
<th>Description</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor area $A_f$ (m$^2$)</td>
<td>36 (6 m × 6 m)</td>
</tr>
<tr>
<td>Ventilation area $A_v$ (m$^2$)</td>
<td>7.2 (3.6 m wide × 2 m high)</td>
</tr>
<tr>
<td>Total area of compartment boundaries (including windows) (m$^2$)</td>
<td>153.6 [2 \times 6 \times 6 + (4 \times 3.4 \times 6)]</td>
</tr>
<tr>
<td>Height of ventilation opening $h$ (m)</td>
<td>2</td>
</tr>
<tr>
<td>Opening factor $O(m^{1/2}) = (A_v\sqrt{h})/A_t$</td>
<td>0.066</td>
</tr>
</tbody>
</table>

The walls and floor are lined with gypsum-based plasterboard and the ceiling is constructed from precast concrete planks. Table 3.7 summarizes the material properties required for the calculation of the compartment time–temperature response.

The $b$ value to be used for design is a weighted average where $b = \Sigma(b_j A_j/A_t)$. Here the relevant $b$ value to be used in the design is 945 J/m$^2$ s$^{1/2}$K.

<table>
<thead>
<tr>
<th>Construction</th>
<th>Material</th>
<th>Thermal inertia ($b$ value – J/m$^2$ s$^{1/2}$K)</th>
<th>Area (m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>Concrete</td>
<td>2280</td>
<td>36</td>
</tr>
<tr>
<td>Floor</td>
<td>Plasterboard</td>
<td>520</td>
<td>36</td>
</tr>
<tr>
<td>Walls</td>
<td>Plasterboard</td>
<td>520</td>
<td>76.8</td>
</tr>
</tbody>
</table>

No information on the thermal properties of commonly used construction materials is provided in the Eurocode. Some guidance is available in the literature and this is reproduced in Table 3.8. The main values for thermal inertia are taken from Competitive Steel Buildings Through Natural Fire Safety Concept,\textsuperscript{32} with the values in brackets taken from the CIB W14 Workshop Report, Design guide: structural fire safety.\textsuperscript{33} The discrepancies in $b$ value are indicative of the variation in supposedly similar materials. Clearly more information is required in this area. Wherever possible, designers should consult manufacturers to obtain accurate material property data for calculations.

<table>
<thead>
<tr>
<th>Material</th>
<th>Thermal inertia $b$ value (J/m$^2$ s$^{1/2}$K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal-weight concrete</td>
<td>2034.7 (2280)</td>
</tr>
<tr>
<td>Lightweight concrete</td>
<td>1122.5 (840)</td>
</tr>
<tr>
<td>Structural steel</td>
<td>13 422.3 (15 000)</td>
</tr>
<tr>
<td>Calcium silicate board</td>
<td>151.8</td>
</tr>
<tr>
<td>Timber</td>
<td>223.8 (600)</td>
</tr>
<tr>
<td>Brick</td>
<td>1521.5 (1200)</td>
</tr>
</tbody>
</table>

The characteristic fire load density is generally taken as the 80% fractile figure reproduced in the Eurocode. In this case a design fire load density of 570 MJ/m$^2$ has
been used. Fire load densities are published in both EN 1991-1-2 and UK Standards and the CIB Design Guide. The relevant information is given in Table 3.9 with the main figures coming from the Eurocode and the figures in brackets from the UK Standard. The source data for fire load density may be found in the CIB Design Guide and, it should be noted, display a wide variation depending on the country of origin and the precise nature of the occupancy. The codified values are not always in agreement with one another and, in certain cases, such as the characteristic values quoted in the European and UK codes for residential accommodation, give very different answers. Again more work is required to rationalize these critical design parameters.

<table>
<thead>
<tr>
<th>Occupancy</th>
<th>Characteristic fire load density (MJ/m²) – 80% fractile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dwelling</td>
<td>948 (400)</td>
</tr>
<tr>
<td>Hospital</td>
<td>280 (350)</td>
</tr>
<tr>
<td>Hotel</td>
<td>377 (400)</td>
</tr>
<tr>
<td>Office</td>
<td>511 (570)</td>
</tr>
<tr>
<td>School classroom</td>
<td>347 (360)</td>
</tr>
</tbody>
</table>

With these input values, equations (3.5) and (3.6) are used to calculate the compartment time–temperature response and the anticipated duration. In this case the parametric equation predicts a maximum temperature of 995 °C and a time to maximum temperature of 24 min. The predicted response is illustrated in Fig. 3.3 and compared to measured values from two real fire tests.

### 3.6. External atmosphere temperature

External structural members may be exposed to fire by flames and radiated heat emanating from openings in the building. Annex B of EN 1991-1-2 provides a calculation approach for determining thermal actions for external members based on work carried out by Law.

The method allows for the calculation of the maximum compartment temperature, the size and temperature of the flame plume emerging from the openings, and the heat transfer parameters for radiation and convection.

### 3.7. Advanced fire models

In certain circumstances it may be necessary to go beyond a reliance on nominal fire exposures or simple calculation methods. Advanced methods, including zone models based on a solution of the equations for conservation of mass and energy or more complex computational fluid dynamics (CFD) models, may be used to provide information based on a solution of the thermodynamic and aerodynamic variables at various points within the control zone. Such models have been used effectively for many years to model the movement of smoke and toxic gases and are now being extended to model the thermal environment for particular post-flashover fire scenarios. Such complex models are not generally available to structural engineers responsible for the fire engineering design of buildings and would generally be used by research institutions or specialist fire engineering consultants.
CHAPTER 4

Member temperatures

4.1. Introduction
The calculation of the atmosphere temperatures within the fire compartment presented in the previous chapter is the first step in a rational fire engineering design process. The next step is to determine, either through calculation or reference to published data, the temperature distribution within the structural elements.

4.2. Section factors for steel and composite construction
The section factor $A/V$ is a convenient parameter to measure the thermal response of a steel member. Basically, the rate at which a steel beam or column will increase in temperature is proportional to the surface area ($A$) of steel exposed to the fire and inversely proportional to the mass or volume ($V$) of the section. In a fire, a member with low section factor will heat up at a slower rate than one with high section factor.

Calculation of the section factor for different types of unprotected section is shown in Fig. 4.1.

4.3. Unprotected steelwork
EN 1993-1-2 provides a simple design approach for calculating the thermal response of unprotected steel members. This approach can be extended to other metals including wrought iron, cast iron, aluminium alloys and stainless steels.

Assuming an equivalent uniform temperature distribution in a cross-section, the increase of temperature $\Delta \theta_{a,t}$ [K] in an unprotected steel member during a time interval $\Delta t$ is given by:

$$\Delta \theta_{a,t} = k_{sh} \frac{A_m/V}{c_a \rho_a} h_{net,d} \Delta t \quad \text{for } \Delta t \leq 5 \text{s}$$

(4.1)

where:
- $\rho_a$ is the unit mass of steel (kg/m$^3$);
- $A_m$ is the surface area of the member per unit length (m$^2$);
- $A_m/V$ is the section factor for unprotected steel members (m$^{-1}$);
- $c_a$ is the specific heat of steel (J/kgK);
- $h_{net,d}$ is the net heat flux per unit area (W/m$^2$);
- $k_{sh}$ is the correction factor for the shadow effect ($k_{sh} = 1.0$ if the shallow effect is ignored);
- $\Delta t$ is the time interval (s); and
- $V$ is the volume of the member per unit length (m$^3$).
For cross-sections with a convex shape, such as rectangular or circular hollow sections, fully embedded in fire, the shadow effect does not play a role and it can be taken as \( k_{sh} = 1.0 \). Otherwise, the correction factor for the shadow effect \( k_{sh} \) is given by:

\[
k_{sh} = \frac{0.9 [A_m/V]_b}{A_m/V} \quad \text{for I-sections under nominal fire actions}
\]

\[
A_m/V = \frac{1}{t} \quad \text{for } t < b
\]

where \( A_m/V \geq 10 \text{ m}^{-1} \); and \([A_m/V]_b\) is the box value of the section factor.

### 4.4. Steelwork insulated by fire protection

EN 1993-1-2 provides a simple design approach for insulated steel members with non-reactive fire protection materials. The insulating materials can be in the form of profiled or boxed systems, but do not include intumescent coatings. Assuming uniform temperature distribution, the temperature increase \( \Delta \theta_{a,t} \) of an insulated steel member during a time interval \( \Delta t (\leq 30 \text{ s}) \) is given by:

\[
\Delta \theta_{a,t} = \frac{\lambda_p A_p/V}{d_p c_a \rho_a} (\theta_{g,t} - \theta_{a,t}) \Delta t (e^{\phi/10} - 1) \Delta \theta_{g,t} \quad \text{but } \Delta \theta_{a,t} \geq 0 \text{ if } \Delta \theta_{g,t} > 0 \quad (4.2)
\]

with

\[
\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p/V
\]
where:

- \( \lambda_p \) is the thermal conductivity of fire protection material (W/mK);
- \( \theta_{s,t} \) is the steel temperature at time \( t \) (°C);
- \( \theta_{g,t} \) is the ambient gas temperature at time \( t \) (°C);
- \( \Delta \theta_{g,t} \) is the increase of ambient gas temperature during time interval \( \Delta t \) (K);
- \( \rho_s \) is the unit mass of steel (kg/m³);
- \( \rho_p \) is the unit mass of fire protection material (kg/m³);
- \( A_p/V \) is the section factor for steel members insulated by fire protection material (m⁻¹);
- \( A_p \) is the appropriate area of fire protection material per unit length (m²);
- \( c_a \) is the temperature-dependent specific heat of steel (J/kgK);
- \( c_p \) is the temperature-independent specific heat of fire protection material (J/kgK);
- \( d_p \) is the thickness of fire protection material (m);
- \( \Delta t \) is the time interval (s); and
- \( V \) is the volume of the steel member per unit length (m³).

Figure 4.2 illustrates some design values of the section factors \( A_p/V \) for insulated steel members. It is worth noting that the area \( A_p \) of the fire protection material is generally taken as the area of its inner surface. For hollow encasement with a clearance around the steel members, the value of \( A_p \) is taken as that for hollow encasement without a clearance.

### Table 4.2

<table>
<thead>
<tr>
<th>Sketch</th>
<th>Description</th>
<th>Section factor ( A_p/V )</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Contour encasement of uniform thickness" /></td>
<td>Contour encasement of uniform thickness</td>
<td>( A_p = \frac{\text{steel perimeter}}{V} ) steel cross-sectional area</td>
</tr>
<tr>
<td><img src="image" alt="Contour encasement of uniform thickness, exposed to fire on 3 sides" /></td>
<td>Contour encasement of uniform thickness, exposed to fire on 3 sides</td>
<td>( A_p = \frac{\text{steel perimeter} - b}{V} ) steel cross-sectional area</td>
</tr>
<tr>
<td><img src="image" alt="Hollow encasement of uniform thickness. (The clearance ( c_1 ) and ( c_2 &lt; h/4 )" /></td>
<td>Hollow encasement of uniform thickness. (The clearance ( c_1 ) and ( c_2 &lt; h/4 ))</td>
<td>( A_p = \frac{2(b + h)}{V} ) steel cross-sectional area</td>
</tr>
<tr>
<td><img src="image" alt="Hollow encasement of uniform thickness, exposed to fire on 3 sides" /></td>
<td>Hollow encasement of uniform thickness, exposed to fire on 3 sides (The clearance ( c_1 ) and ( c_2 &lt; h/4 ))</td>
<td>( A_p = \frac{2h + b}{V} ) steel cross-sectional area</td>
</tr>
</tbody>
</table>

**Fig. 4.2. Section factor for insulated steel members**

**Fig. 4.3. Evaluation of moisture plateau for protection materials (DD ENV 13381-4: 2002)**
For moist fire protection materials, the steel temperature increase $\Delta \theta_a$ may be modified to allow for a time delay $t_d$ in the rise of the steel temperature when it reaches 100°C, due to the latent heat of vaporization of the moisture, as shown in Fig. 4.3. The calculation method for $t_d$ is given in DD ENV 13381-4 (2002).35

4.5. Unprotected composite slabs

EN 1994-1-2 provides the simple calculation models for determining the sagging and hogging moment resistances of unprotected composite slabs with profiled steel decking exposed to the standard fire. The evaluation of the temperature profiles within the slab is given in its Annex D (informative).

The approach allows the temperatures of the steel sheet, reinforcement bars in the ribs and the concrete slab to be calculated separately. The temperatures of the lower flange, web and upper flange of the steel decking, and the reinforcement bars in the ribs can be obtained by using the empirical formulae. However, the calculation of temperature profiles for the concrete part of the slabs is rather complicated as the temperature distribution across a concrete cross-section exposed to fire conditions will not be uniform. It will be too complicated to establish the isotherms within the concrete by using empirical formulae.

Currently, EN 1994-1-2 only provides a simple model for establishing the isotherm for a certain limiting temperature within the concrete, with temperatures beyond the limiting temperature being neglected and the remaining cross-section being taken as ambient temperature. It must be emphasized that the limiting temperature is derived from equilibrium over the cross-section and has no relation with temperature penetration.36 Such simplification may be adequate for the calculation of hogging moment resistance, but not for the thermal response analysis of the slabs. Alternatively, EN 1994-1-2 provides a conservative approximation by treating the composite slabs as solid slabs with the temperature distribution given in a table.

One assumption of the method is that the steel deck remains bonded to the concrete. Evidence from fire tests suggests that this is not a valid assumption.

4.5.1. Steel decking

The temperatures $\theta_a$ of the lower flange, web and upper flange of the steel decking are given by:

$$\theta_a = b_0 + b_1 \frac{A}{L_r} + b_2 \Phi + b_3 \Phi^2$$

with

$$\Phi = \frac{1}{l_3} \left[ \sqrt{h_2^2 + \left( \frac{l_1 + l_2}{2} \right)^2} - \sqrt{h_2^2 + \left( \frac{l_1 - l_2}{2} \right)^2} \right]$$

where:

- $\Phi$ is the view factor of the upper flange;
- $b_0 \ldots b_4$ are the coefficients for determining the temperatures of various parts of the steel decking as given in Table 4.1;
- $A/L_r$ is the rib geometry factor (mm);
- $A$ is the concrete volume of the rib per metre rib length ($mm^3/m$);
- $h_2$ is the depth of the rib (mm);
- $L_r$ is the exposed area of the rib per metre rib length ($mm^2/m$);
- $l_1, l_2$ are the distances as shown in Fig. 4.4 (mm); and
- $l_3$ is the width of the upper flange (mm).

The definition of the geometric dimensions and factors of a typical composite slab is given in Fig. 4.4.
4.5.2 Reinforcement bars

The temperature \( \theta_i \) of the reinforcement bars in the rib (see Fig. 4.5) is given by:

\[
\theta_i = c_0 + c_1 \frac{u_3}{h_2} + c_2 z + c_3 \frac{A}{L_r} + c_4 \alpha + c_5 \frac{1}{l_3}
\]

where:
- \( \alpha \) is the angle of the web (degrees);
- \( c_0 \ldots c_5 \) are the coefficients for determining the temperature of rebars in the rib as given in Table 4.2;
- \( A/L_r \) is the rib geometry factor (mm);
- \( A \) is the concrete volume of the rib per metre rib length (mm\(^3\)/m);

\[
\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}}
\]

The rib geometry factor \( A/L_r \) is given by:

\[
A = \frac{h_2}{L_r} \left( \frac{h_2 + b}{2} \right)
\]

\[
L_r = \frac{b + 2}{h_2 + \left( \frac{l_1 - b}{2} \right)^2}
\]

**Fig. 4.4. Definition of geometric dimensions of composite slabs**

**Table 4.1. Coefficients for determining temperatures of various parts of steel decking**

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Standard fire resistance</th>
<th>Part of steel sheet</th>
<th>( b_0 ) (°C)</th>
<th>( b_1 ) (°C mm)</th>
<th>( b_2 ) (°C mm)</th>
<th>( b_3 ) (°C)</th>
<th>( b_4 ) (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal-weight</td>
<td>R60</td>
<td>Lower flange</td>
<td>951</td>
<td>-1197</td>
<td>-2.32</td>
<td>86.4</td>
<td>-150.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>661</td>
<td>-833</td>
<td>-2.96</td>
<td>537.7</td>
<td>-351.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>340</td>
<td>-3269</td>
<td>-2.62</td>
<td>1148.4</td>
<td>-679.8</td>
</tr>
<tr>
<td></td>
<td>R90</td>
<td>Lower flange</td>
<td>1018</td>
<td>-839</td>
<td>-1.55</td>
<td>65.1</td>
<td>-108.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>816</td>
<td>-959</td>
<td>-2.21</td>
<td>464.9</td>
<td>-340.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>618</td>
<td>-2786</td>
<td>-1.79</td>
<td>767.9</td>
<td>-472.0</td>
</tr>
<tr>
<td></td>
<td>R120</td>
<td>Lower flange</td>
<td>1063</td>
<td>-679</td>
<td>-1.13</td>
<td>46.7</td>
<td>-82.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>925</td>
<td>-949</td>
<td>-1.82</td>
<td>344.2</td>
<td>-267.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>770</td>
<td>-2460</td>
<td>-1.67</td>
<td>592.6</td>
<td>-379.0</td>
</tr>
<tr>
<td>Lightweight</td>
<td>R30</td>
<td>Lower flange</td>
<td>800</td>
<td>-1326</td>
<td>-2.65</td>
<td>114.5</td>
<td>-181.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>483</td>
<td>-286</td>
<td>-2.26</td>
<td>439.6</td>
<td>-244.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>331</td>
<td>-2284</td>
<td>-1.54</td>
<td>488.8</td>
<td>-131.7</td>
</tr>
<tr>
<td></td>
<td>R60</td>
<td>Lower flange</td>
<td>955</td>
<td>-622</td>
<td>-1.32</td>
<td>47.7</td>
<td>-81.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>761</td>
<td>-558</td>
<td>-1.67</td>
<td>426.5</td>
<td>-303.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>607</td>
<td>-2261</td>
<td>-1.02</td>
<td>664.5</td>
<td>-410.0</td>
</tr>
<tr>
<td></td>
<td>R90</td>
<td>Lower flange</td>
<td>1019</td>
<td>-478</td>
<td>-0.91</td>
<td>32.7</td>
<td>-60.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>906</td>
<td>-654</td>
<td>-1.36</td>
<td>287.8</td>
<td>-230.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>789</td>
<td>-1847</td>
<td>-0.99</td>
<td>469.5</td>
<td>-313.0</td>
</tr>
<tr>
<td></td>
<td>R120</td>
<td>Lower flange</td>
<td>1062</td>
<td>-399</td>
<td>-0.65</td>
<td>19.8</td>
<td>-43.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web</td>
<td>989</td>
<td>-629</td>
<td>-1.07</td>
<td>186.1</td>
<td>-152.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Upper flange</td>
<td>903</td>
<td>-1561</td>
<td>-0.92</td>
<td>305.2</td>
<td>-197.2</td>
</tr>
</tbody>
</table>

Note: For intermediate values, linear interpolation is allowed.
Table 4.2. Coefficients for determining temperatures of rebars in the rib

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Standard fire resistance</th>
<th>$c_0$ (°C)</th>
<th>$c_1$ (°C)</th>
<th>$c_2$ (°C mm$^{0.5}$)</th>
<th>$c_3$ (°C mm$^{1/5}$)</th>
<th>$c_4$ (°C)</th>
<th>$c_5$ (°C mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal-weight</td>
<td>R60</td>
<td>1191</td>
<td>-250</td>
<td>-240</td>
<td>-5.01</td>
<td>1.04</td>
<td>-925</td>
</tr>
<tr>
<td>concrete</td>
<td>R90</td>
<td>1342</td>
<td>-256</td>
<td>-235</td>
<td>-5.30</td>
<td>1.39</td>
<td>-1267</td>
</tr>
<tr>
<td>Lightweight</td>
<td>R120</td>
<td>1387</td>
<td>-238</td>
<td>-227</td>
<td>-4.79</td>
<td>1.68</td>
<td>-1326</td>
</tr>
<tr>
<td>concrete</td>
<td>R30</td>
<td>809</td>
<td>-135</td>
<td>-243</td>
<td>-0.70</td>
<td>0.48</td>
<td>-315</td>
</tr>
<tr>
<td></td>
<td>R60</td>
<td>1336</td>
<td>-242</td>
<td>-292</td>
<td>-6.11</td>
<td>1.63</td>
<td>-900</td>
</tr>
<tr>
<td></td>
<td>R90</td>
<td>1381</td>
<td>-240</td>
<td>-269</td>
<td>-5.46</td>
<td>2.24</td>
<td>-918</td>
</tr>
<tr>
<td></td>
<td>R120</td>
<td>1397</td>
<td>-230</td>
<td>-253</td>
<td>-4.44</td>
<td>2.47</td>
<td>-906</td>
</tr>
</tbody>
</table>

Note: For intermediate values, linear interpolation is allowed.

Figure 4.5 illustrates how to measure the distances $u_1$, $u_2$ and $u_3$ for the reinforcement bars in the ribs of a composite slab.

4.5.3. Concrete slab over steel decking

For the concrete slab over trapezoidal or dovetail steel decking, EN 1994-1-2 does not provide a simple model for calculating the temperature distribution. It does, however, provide a temperature distribution based on an equivalent solid slab.

In the calculation, the composite slab is replaced by a solid slab with an effective thickness $h_{\text{eff}}$ which is given by:

$$h_{\text{eff}} = \begin{cases} 
  h_1 + 0.5h_2 \left( \frac{l_1 + l_2}{l_1 + l_3} \right) & \text{for } h_2/h_1 \leq 1.5 \text{ and } h_1 > 40 \text{ mm} \\
  h_1 \left[ 1 + 0.75 \left( \frac{l_1 + l_2}{l_1 + l_3} \right) \right] & \text{for } h_2/h_1 > 1.5 \text{ and } h_1 > 40 \text{ mm}
\end{cases} \quad (4.5)$$
where the cross-section dimensions $h_1, h_2, l_1, l_2$ and $l_3$ are given in Fig. 4.6. The temperature at a depth $x$ from $h_{\text{eff}}$ can then be obtained from Table 4.3.

### 4.6. Temperature profile for concrete members

Annex A (informative) of EN 1992-1-2\(^23\) provides a series of calculated temperature profiles for slabs or walls, beams and columns. The available design charts are summarized in Table 4.4.

#### Table 4.4. Summary of design charts for concrete members

<table>
<thead>
<tr>
<th>Member</th>
<th>Cross-sectional dimensions (mm)</th>
<th>Standard fire resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slabs or walls exposed to one side</td>
<td>Thickness = 200</td>
<td>R30–R240</td>
</tr>
<tr>
<td>Beams</td>
<td>Height $\times$ width = $300 \times 160$</td>
<td>R30–R90 and $500, ^\circ \text{C}$ isotherms</td>
</tr>
<tr>
<td></td>
<td>Height $\times$ width = $600 \times 300$</td>
<td>R60–R120</td>
</tr>
<tr>
<td></td>
<td>Height $\times$ width = $800 \times 500$</td>
<td>R60–R240</td>
</tr>
<tr>
<td>Square columns</td>
<td>Height $\times$ width = $300 \times 300$</td>
<td>R30–R120 and $500, ^\circ \text{C}$ isotherms</td>
</tr>
<tr>
<td>Circular columns</td>
<td>Diameter = 300</td>
<td>R30–R120 and $500, ^\circ \text{C}$ isotherms</td>
</tr>
</tbody>
</table>

The design charts are based on the following assumptions:

- The specific heat of concrete corresponds to 1.5% of moisture content.
- The lower limit of thermal conductivity is used.
- The emissivity of concrete surface is 0.7.
- The convection factor is 25.
- The spalling of concrete does not occur during fire exposure.

Figure 4.7 shows the temperature profiles for slabs with thickness = 200 mm and 600 mm for R30 to R240. Figure 4.8 shows the temperature profiles for beams with width $\times$ height = $(160 \times 230 \text{ mm})$ and $(300 \times 600 \text{ mm})$ for R30 to R120.
**Fig. 4.7.** Temperature profiles for slabs with height = 200 mm

**Fig. 4.8.** Temperature profiles for beams with different width-to-height ratios
CHAPTER 5

Static loads

5.1. Introduction
An accurate assessment of the performance of a structural member during a fire requires a knowledge of both the reduction in material properties at increasing temperature and an accurate assessment of the loads acting on the structure at the time of the fire. Load effects have a significant impact during a fire and this is reflected in the requirement for realistic load levels to be in place during standard fire tests. The importance of applied load on fire resistance has long been recognized and was specifically incorporated into the calculation models in the fire part of the British Standard for steel structures, BS 5950 Part 8. The Eurocodes include load effects not only in relation to steel and composite structures but also for concrete members. This is an important development as there is no explicit allowance for the influence of applied load on the performance of concrete structures in the National standard.

5.2. Partial safety factors for loads
The calculation of the load effects at the fire limit state is different to the procedure adopted in current National standards. The designer must be familiar with both EN 1990 (basis of design) which provides the required load combinations (as for ambient temperature design) and with EN 1991-1-2 (the fire part of the Actions code) which in addition to specifying the available options for thermal actions for temperature analysis (see Chapter 3) also specifies the mechanical actions for structural analysis. In particular EN 1991-1-2 specifies the partial factor for imposed (assuming leading variable action) loading for the fire limit state. Fire loading is an ultimate limit state accidental design situation (see EN 1990) of the form:

$$E_d = E(G_k, P; A_d; \Psi_1, \Psi_2; Q_{k,1})$$

for $$j \geq 1; i \geq 1$$

where:

- $E$ is the effect of actions ($E_d$ is the design value of the effect of actions);
- $G$ is the permanent action (dead load);
- $P$ is the relevant representative value of a prestressing action (where present);
- $A_d$ is the design value of an accidental action;
- $\Psi_1$ is the factor for frequent value of a variable action;
- $\Psi_2$ is the factor for quasi-permanent value of a variable action; and
- $Q_{k,1}$ is the characteristic value of a single variable action ($Q_{k,1}$ is the characteristic value of the leading variable action – often the imposed load).

In the fire situation $A_d$ is the effect of the fire itself on the structure, i.e. the effects of restrained thermal expansion, thermal gradients, etc. However, EN 1991-1-2 states that:

*Indirect actions from adjacent members need not be considered when fire safety requirements refer to members under standard fire conditions.*
And also that:

*Imposed and constrained expansions and deformations caused by temperature changes due to fire exposure results in effects of actions, e.g. forces and moments which shall be considered with the exception of those where they:

- May be recognised a priori to be negligible or favourable
- Are accounted for by conservatively chosen support models and boundary conditions and/or implicitly considered by conservatively specified fire safety requirements.

EN 1990 allows the use of either $\Psi_1$ or $\Psi_2$ with the main variable action (generally the imposed load). EN 1991-1-2 recommends the use of $\Psi_2$; however, the UK National Annex will specify the use of $\Psi_1$ as detailed in Table 5.1.

The benefits of this approach for concrete construction where the ratio of dead to imposed loads is relatively high (compared to typical steel-framed structures) are not particularly significant. The detailed calculations may be found in the worked examples section. The relationship between the reduction factor and the ratio of the dead and imposed loads is illustrated in Fig. 5.1.

### Table 5.1. $\psi_b$ values for UK

<table>
<thead>
<tr>
<th>Action</th>
<th>$\psi_1$</th>
<th>$\psi_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Imposed loads in buildings:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Category A: domestic, residential</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category B: office areas</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category C: congregation areas</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category D: shopping areas</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category E: storage areas</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td>Category F: traffic area ≤ 30 kN</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Category G: traffic area 30–160 kN</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Category H: roofs</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Snow load: $H \leq 1000$ m a.s.l.</td>
<td>—</td>
<td>0</td>
</tr>
<tr>
<td>Wind loads on buildings</td>
<td>0.2</td>
<td>0</td>
</tr>
</tbody>
</table>

And also that:

![Fig. 5.1. Relationship between reduction factor $\eta_b$ and ratio of dead and imposed loads for values of $\psi_b$](chart.png)

### 5.3. Design values of loads

The concept of reduced partial factors for the fire limit state will be explained using a design example. The calculations are for the design of a column in a six-storey braced frame to resist the loading applied from a 6 m × 9 m floor area. The design axial force in the column at the fire limit state is calculated using the appropriate partial factors.
5.3.1. Loading

**Permanent actions (G)**
Uniformly distributed load (UDL) over floor area $G_k = 2.00 \text{kN/m}^2$

**Variable actions (Q)**
Uniformly distributed load (UDL) over floor area $Q_k = 3.50 \text{kN/m}^2$

5.3.2. Ambient temperature design loads

**Load factors**
- Partial loading factor for permanent actions $\gamma_G = 1.35$ (EN 1990 Table A1.2B and NA)
- Partial loading factor for variable actions $\gamma_Q = 1.50$ (EN 1990 Table A1.2B and NA)

**Design values of actions – ultimate limit state**
- Area UDL per floor $W = (\gamma_G G_k) + (\gamma_Q Q_k) = 7.95 \text{kN/m}^2$
- Design axial force $N_{Ed} = 6 \text{m} \times 9 \text{m} \times W \times 5 = 2146.50 \text{kN}$

5.3.3. Fire limit state design loads

For the fire limit state, partial loading factors ($\gamma_i$) are not applied to either permanent actions or variable actions.

- Combination coefficient for variable action $\Psi_1 = 0.50$. This is the value for offices using $\psi_1 = 0.5$ from Table 5.1.
- Note: EN 1990 allows use of either $\Psi_1$ or $\Psi_2$ with the main variable action. The National Annex will specify which coefficient to use. EN 1991-1-2 recommends the use of $\Psi_2$; however, the UK National Annex will specify $\Psi_1$.

5.3.4. Design values of actions – ultimate limit state fire design situation

- Area UDL per floor $W = G_k + (\Psi_1 Q_k) = 3.75 \text{kN/m}^2$
- Design axial force $N_{Ed} = 6 \text{m} \times 9 \text{m} \times W \times 5 = 1012.50 \text{kN}$

5.4. Definition of load level, load intensity and degree of utilization

Despite a slight difference in terminology between the different European standards, the basic concept of load ratio, load level, load intensity or degree of utilization is the same. The resistance of the member at the fire limit state is assessed according to the amount of load applied during a fire compared to the ambient temperature load-bearing capacity. The concept of load ratio is very useful with regard to tabulated data as it allows for generic solutions that cover a wide range of potential applications.

Throughout all the fire parts of the relevant Eurocodes the concept of a reduction factor for the fire limit state is used, where:

**Reduction factor** $\eta_h = (G_k + \Psi_1 Q_{k,1})/((\gamma_G G_k + \gamma_Q Q_{k,1})$ for load combination (6.10) in EN 1990

In EN 1992-1-2 two factors related to the applied load are employed, namely load level and degree of utilization. These are defined as follows.

5.4.1. Load level ($n$)

This term is used to determine the fire resistance of reinforced concrete columns and relates the load imposed at the time of the fire to the ambient temperature load capacity.

$$n = N_{0Ed,h}/[0.7(A_c f_{ed} + A_y f_{yd})]$$

(5.2)
where:

\[ N_{0Ed,fi} \] is the axial load under fire conditions (kN);
\[ A_c \] is the area of concrete (mm\(^2\));
\[ f_{cd} \] is the concrete design compressive strength (N/mm\(^2\));
\[ A_s \] is the area of steel (mm\(^2\)); and
\[ f_{yd} \] is the steel design tensile strength (N/mm\(^2\)).

The load imposed at the time of the fire is dependent on the choice of the partial factor for loading at the fire limit state. The choice of the partial factor is a Nationally Determined Parameter (NDP) and is chosen at the discretion of the National body. The permissible values are set out in EN 1990 and the choice is made in the National Annex to the fire part of EN 1991. For most common cases (domestic and office) the value of the partial factor for imposed load will be 0.5, i.e. 0.5 \( \times \) the ambient temperature value. As a conservative assumption when calculating the load level, \( N_{0Ed,fi} \) may be taken as 0.7\( N_{0Ed} \) (\( \eta_{fi} = 0.7 \)) unless calculated explicitly.

5.4.2. Degree of utilization (\( \mu_{fi} \))

This is the ratio of the load applied at the fire limit state to the load applied under ambient conditions and is dictated by the choice of partial factor for the fire limit state as discussed above. It is used in the design of both columns and load-bearing walls.

\[ \mu_{fi} = \frac{N_{Ed,fi}}{N_{Rd}} \]  \hspace{1cm} (5.3)

where \( N_{Ed,fi} \) is the design axial load in the fire situation (kN) and \( N_{Rd} \) is the design resistance of the column at normal temperature conditions (kN).

The reduction factor \( \eta_{fi} \) may be used instead of \( \mu_{fi} \) for the design load level as a conservative assumption, as \( \eta_{fi} \) assumes the member is fully loaded under ambient temperature conditions. In EN 1994-1-2 the tabulated values are generally dependent on the load level for fire design \( \eta_{fi} \) which can be explicitly calculated according to the formula above or taken conservatively as 0.65.
CHAPTER 6

Thermal and mechanical properties of materials

6.1. Introduction

6.2. Steel
Hot-finished carbon steel begins to lose strength at temperatures above 300°C and reduces in strength at a steady rate up to 800°C. The small residual strength then reduces more gradually until it reaches the melting temperature at around 1500°C. This behaviour is similar for hot-rolled reinforcing steels. For cold-worked steels, including reinforcement, there is a more rapid decrease of strength after 300°C. In addition to the reduction of material strength and stiffness, steel displays a significant creep phenomenon at temperatures over 450°C.

High-temperature creep is dependent on the stress level and heating rate. The occurrence of creep indicates that the stress and the temperature history have to be taken into account in estimating the strength and deformation behaviour of steel structures in fire. Including creep explicitly within analytical models is complex. For the simple design methods presented in the Eurocodes it is widely accepted that the effect of creep is implicitly considered in the stress–strain–temperature relationship.

Thermal and mechanical properties of different types of steel at elevated temperatures are discussed. These include:
- hot-rolled carbon steel
- stainless steel
- light-gauged steel.

6.2.1. Hot-rolled carbon steel
EN 1993-1-2 provides the material properties for hot-rolled steel of grades S235, S275 and S355 in accordance with EN 10025.

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

The coefficient of thermal elongation of steel $\frac{\Delta l}{l}$ can be determined by:

$$\frac{\Delta l}{l} = 1.2 \times 10^{-5} \theta_a + 0.4 \times 10^{-8} \theta_a^2 - 2.416 \times 10^{-4} \quad \text{for } 20°C \leq \theta_a < 750°C$$  \hspace{1cm} (6.1a)
where:

\( l \) is the length at 20°C;

\( \Delta l \) is the temperature-induced elongation; and

\( \theta_a \) is the steel temperature (°C).

The variation of the thermal elongation with temperature is shown in Fig. 6.1.

The specific heat of steel \( c_a \) (in J/kg K) can be determined by:

\[
c_a = 425 + 7.73 \times 10^{-1} \theta_a - 1.69 \times 10^{-3} \theta_a^2 + 2.22 \times 10^{-6} \theta_a^3 \quad \text{for} \ 20°C \leq \theta_a < 600°C
\]

\[
c_a = 666 + 13002/(738 - \theta_a) \quad \text{for} \ 600°C \leq \theta_a < 735°C
\]

\[
c_a = 545 + 17820/(\theta_a - 731) \quad \text{for} \ 735°C \leq \theta_a < 900°C
\]

\[
c_a = 650 \quad \text{for} \ 900°C \leq \theta_a < 1200°C
\]

The variation of the specific heat with temperature is shown in Fig. 6.2.

The thermal conductivity of steel \( \lambda_a \) (in W/m K) can be determined by:

\[
\lambda_a = 54 - 3.33 \times 10^{-2} \theta_a \quad \text{for} \ 20°C \leq \theta_a < 800°C
\]

\[
\lambda_a = 27.3 \quad \text{for} \ 800°C \leq \theta_a < 1200°C
\]

The variation of the thermal conductivity with temperature is shown in Fig. 6.3.

6.2.1.2. Mechanical properties

The stress–strain behaviour of carbon steel at high temperatures shows no clear yield plateau, with strain hardening occurring throughout the plastic range. British Steel (now Corus) carried out an extensive small-scale tensile transient test programme in the 1980s on BS 4360: Grade 43A and Grade 50B steels to provide elevated temperature data for
structural fire engineering design applications. To represent the behaviour of beams and columns in standard fire tests, the heating rates were set at a range 5–20 °C/min.

The test results show that carbon steel begins to lose strength at temperatures above 300 °C and reduces in strength at a steady rate up to 800 °C. The well-defined yield plateau at 20 °C is replaced by a gradual increase of strength with increasing strain (or strain-hardening) at high temperatures.

Based on the British Steel data, EN 1993-1.2 derives the reduction factors for effective yield strength, proportional limit and slope of linear elastic range as given in Table 6.1. The effective yield strength is related to 2% strain limit. Figure 6.4 illustrates the variation of the reduction factors with temperature.

The definitions of effective yield strength, proportional limit and slope of linear elastic range are established on the basic characteristic of the stress–strain model for steel at high temperatures given in EN 1993-1.2. Figure 6.5 shows that the first part of the curve is a linear line progressing up to the proportional limit \( f_{p,\theta} \) and the elastic modulus \( E_{a,\theta} \) is equal to the slope of this straight-line segment. The second part of the curve depicts the transition from the elastic to the plastic range. This region is formulated by an elliptical progression up to the effective yield strength \( f_{y,\theta} \). The third part of the curve is a flat yield plateau up to a limiting strain. The last part of the curve is characterized by a linear line decreasing to zero stress at the ultimate strain.

Comparing their reduction factors at elevated temperatures (Table 6.1), it can be seen that the stiffness of steel reduces more rapidly than the strength. This indicates that the failure mode of steel members may change at elevated temperatures. For instance, a steel beam comprising a slender I-section, which is designed for plastic-hinge failure under ultimate

<table>
<thead>
<tr>
<th>Steel temperature ( \theta_a ) (°C)</th>
<th>Reduction factor for effective yield strength ( k_y = f_{y,\theta}/f_y )</th>
<th>Reduction factor for proportional limit ( k_p = f_{p,\theta}/f_y )</th>
<th>Reduction factor for the slope of the linear elastic range ( k_{E_a} = E_{a,\theta}/E_a )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>100</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>200</td>
<td>1.000</td>
<td>0.807</td>
<td>0.900</td>
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<td>1.000</td>
<td>0.613</td>
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<tr>
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<td>0.110</td>
<td>0.050</td>
<td>0.090</td>
</tr>
<tr>
<td>900</td>
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<td>0.0675</td>
</tr>
<tr>
<td>1000</td>
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<td>0.0250</td>
<td>0.0450</td>
</tr>
<tr>
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<td>0.0225</td>
</tr>
<tr>
<td>1200</td>
<td>0.000</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>
load at ambient temperature, may experience the premature failure of web buckling at elevated temperatures.

EN 1993-1-2 provides detailed mathematical formulations for the stress–strain relationship of steel at elevated temperatures, as shown in Fig. 6.6.

The effect of creep is implicitly considered, with the material models being applicable for heating between 2 and 50 K/min.

![Fig. 6.4. Reduction factors for stress–strain relationship of carbon steel at elevated temperatures](image)

**Fig. 6.4.** Reduction factors for stress–strain relationship of carbon steel at elevated temperatures

![Fig. 6.5. Stress–strain relationship for carbon steel at elevated temperatures](image)

**Fig. 6.5.** Stress–strain relationship for carbon steel at elevated temperatures

load at ambient temperature, may experience the premature failure of web buckling at elevated temperatures.

EN 1993-1-2 provides detailed mathematical formulations for the stress–strain relationship of steel at elevated temperatures, as shown in Fig. 6.6.

The effect of creep is implicitly considered, with the material models being applicable for heating between 2 and 50 K/min.

![Fig. 6.6. Mathematical formulations of stress–strain relationship for carbon steel at elevated temperatures](image)

**Fig. 6.6.** Mathematical formulations of stress–strain relationship for carbon steel at elevated temperatures

<table>
<thead>
<tr>
<th>Strain range</th>
<th>Stress $\sigma_a (\theta_a)$</th>
<th>Tangent modulus $E_{a\theta}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon \leq \varepsilon_{p,\theta}$</td>
<td>$\varepsilon E_{a\theta}$</td>
<td>$E_{a\theta}$</td>
</tr>
<tr>
<td>$\varepsilon_{p,\theta} &lt; \varepsilon &lt; \varepsilon_{y,\theta}$</td>
<td>$f_{p,\theta} - c + \frac{b}{a} \sqrt{a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2}$</td>
<td>$\frac{b(\varepsilon_{y,\theta} - \varepsilon)}{a \sqrt{a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2}}$</td>
</tr>
<tr>
<td>$\varepsilon_{y,\theta} \leq \varepsilon &lt; \varepsilon_{u,\theta}$</td>
<td>$f_{p,\theta}$</td>
<td>0</td>
</tr>
<tr>
<td>$\varepsilon_{u,\theta} \leq \varepsilon &lt; \varepsilon_{u,\theta}$</td>
<td>$f_{p,\theta}[1 - (\varepsilon - \varepsilon_{u,\theta})(\varepsilon - \varepsilon_{u,\theta})]$</td>
<td>0</td>
</tr>
<tr>
<td>$\varepsilon = \varepsilon_{u,\theta}$</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Parameters**

- $f_{p,\theta}$: effective yield strength at elevated temperatures;
- $f_{p,\theta}$: proportional limit at elevated temperatures;
- $E_{a\theta}$: slope of the linear elastic range at elevated temperatures;
- $E$: slope of the linear elastic range at room temperature;
- $\varepsilon_{p,\theta}$: strain at the proportional limit at elevated temperatures;
- $\varepsilon_{y,\theta}$: yield strain at elevated temperatures;
- $\varepsilon_{t,\theta}$: limiting strain for yield strength at elevated temperatures;
- $\varepsilon_{u,\theta}$: ultimate strain at elevated temperatures.

**Functions**

- $a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + \sigma E_{a\theta})$
- $b^2 = c(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a\theta} + c^2$
- $c = (\varepsilon_{p,\theta} - \varepsilon_{0})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta}) - 2(\varepsilon_{p,\theta} - \varepsilon_{0})$
EN 1993-1-2 further extends the stress–strain relationship to include strain-hardening for steel temperatures below 400°C, provided local or overall buckling does not lead to premature collapse (Fig. 6.7). In this case, the mathematical formulations in Fig. 6.6 need to be modified as follows:

For $0 < \varepsilon < 0.04$

$$\sigma_a = 50(f_{u,\theta} - f_{y,\theta})\varepsilon + 2f_{y,\theta} - f_{u,\theta}$$

For $0.04 \leq \varepsilon \leq 0.15$

$$\sigma_a = f_{u,\theta}$$

For $0.15 < \varepsilon < 0.20$

$$\sigma_a = f_{u,\theta}[1 - 20(\varepsilon - 0.15)]$$

For $\varepsilon \geq 0.20$

$$\sigma_a = 0$$

where $f_{u,\theta}$ is the ultimate strength at elevated temperatures, allowing for strain-hardening. The ultimate strength at elevated temperatures $f_{u,\theta}$, allowing for strain-hardening, should be determined as follows:

For $\theta_a < 300°C$

$$f_{u,\theta} = 1.25f_{y,\theta}$$

For $300°C \leq \theta_a < 400°C$

$$f_{u,\theta} = f_{y,\theta}(2 - 0.0025\theta_a)$$

For $\theta_a \geq 400°C$

$$f_{u,\theta} = f_{y,\theta}$$

Figure 6.8 shows the stress–strain relationships for S275 steel at elevated temperatures, allowing for strain hardening.

### 6.2.2. Stainless steel

Stainless steel covers a wide range of corrosion and heat-resistant iron-based materials, which contain at least 10% chromium, a maximum 1.2% carbon and other alloying elements. There are five basic groups of stainless steel, classified according to their metallurgical structure, namely austenitic, ferritic, martensitic, duplex and precipitation-hardening groups. Austenitic and duplex stainless steels are the most widely used in architectural and structural engineering applications, mainly due to their good weldability.

Annex C of EN 1993-1-2 provides guidance on the material properties, at elevated temperatures, for stainless steel Grades 1.4301, 1.4401, 1.4571, 1.4003 and 1.4462. For other grades of stainless steel, the Code suggests that their mechanical properties may be
6.2.2.1. Thermal properties

The thermal properties of stainless steel are quite different from those of carbon steel. The main differences are as follows.

- The rate of thermal expansion of stainless steel remains relatively constant up to 1200°C compared to carbon steel since stainless steel does not experience a phase transformation.
- The magnitude of thermal expansion of stainless steel is greater than the thermal expansion of carbon steel.
- The specific heat of stainless steel increases slightly at elevated temperatures compared to carbon steel, which has a huge increase in specific heat at 730°C due to a chemical transformation from ferrite-pearlite to austenite.
- At ambient temperature, stainless steel has a much lower thermal conductivity compared to carbon steel. However, the thermal conductivity of stainless steel increases at elevated temperatures and exceeds the value for carbon steel above 1000°C.

The thermal elongation of austenitic stainless steel $\Delta l/l$ may be determined by:

$$\frac{\Delta l}{l} = (16 + 4.76 \times 10^{-3} \theta_a - 1.243 \times 10^{-6} \theta_a^2) \times (\theta_a - 20) \times 10^{-6}$$

(6.4)

where:

- $l$ is the length at room temperature of stainless steel member;
- $\Delta l$ is the temperature-induced elongation of stainless steel member; and
- $\theta_a$ is the steel temperature (°C).

The variation of the thermal elongation with temperature is shown in Fig. 6.9.
The specific heat of stainless steel \( c_a \) (in J/kg K) may be determined by:
\[
c_a = 450 + 0.28 \times \theta_a - 2.91 \times 10^{-4} \theta_a^2 + 1.34 \times 10^{-7} \theta_a^3
\] (6.5)

The variation of the specific heat with temperature is shown in Fig. 6.10.

The thermal conductivity of stainless steel \( \lambda_a \) (in W/m K) may be determined by:
\[
\lambda_a = 14.6 + 1.27 \times 10^{-2} \theta_a
\] (6.6)

The variation of the thermal conductivity with temperature is shown in Fig. 6.11.

6.2.2.2. Mechanical properties

The stress–strain relationship for stainless steel at elevated temperatures, given in EN 1993-1-2, is applicable for heating rates between 2 and 50 K/min. The detailed mathematical formulations are shown in Fig. 6.12.

Annex C of EN 1993-1-2 provides reduction factors, relative to the appropriate value at 20°C, for the stress–strain relationship of several grades of stainless steel at elevated temperatures as follows:

- Slope of linear elastic range, relative to slope at 20°C: \( k_{E,\theta} = E_{a,\theta}/E_a \)
- Proof strength, relative to yield strength at 20°C: \( k_{0.2p,\theta} = f_{0.2p,\theta}/f_y \)
- Tensile strength, relative to tensile strength at 20°C: \( k_u = f_{u,\theta}/f_u \)

In addition, the Code gives a correction factor for the yield strength \( k_{2\%_\theta} \) for the use of simple calculation methods. It is assumed that the ‘effective’ yield strength to be used in simple calculation methods should be between the values of proof strength \( f_{0.2p,\theta} \) and tensile strength \( f_u \) as given by:
\[
f_y = f_{0.2p,\theta} + k_{2\%_\theta}(f_{u,\theta} - f_{0.2p,\theta})
\] (6.7)

where the values of \( k_{2\%_\theta} \) for various grades of stainless steel, ranging from 0.19 to 0.47, are given in Annex C of EN 1993-1-2.

Table 6.2 and Fig. 6.13 illustrate the variation of the above-mentioned reduction factors for Grade 1.4301 stainless steel. Annex C of EN 1993-1-2 provides values for other grades of stainless steel.

---

**Fig. 6.10.** Specific heat of stainless steel as a function of temperature

**Fig. 6.11.** Thermal conductivity of stainless steel as a function of temperature
\[ \sigma = \tan \alpha \]

where:
- \( f_{u, \theta} \) is the tensile strength;
- \( f_{0.2p, \theta} \) is the proof strength at 0.2% plastic strain;
- \( E_{a, \theta} \) is the slope of the linear elastic range;
- \( E_{ct, \theta} \) is the slope of proof strength;
- \( \varepsilon_{c, \theta} \) is the total strain at proof strength;
- \( \varepsilon_{u, \theta} \) is the ultimate strain.

Strain range \( \varepsilon \leq \varepsilon_{c, \theta} \):

\[ \frac{E_{a, \theta}}{1 + a \varepsilon^b} \]

Strain range \( \varepsilon_{c, \theta} < \varepsilon < \varepsilon_{u, \theta} \):

\[ f_{0.2p, \theta} - \varepsilon + \frac{d}{c} \sqrt{\varepsilon^2 - \left(\varepsilon_{u, \theta} - \varepsilon\right)^2} \]

Parameters:
- \( \varepsilon_{c, \theta} = f_{0.2p, \theta} / E_{a, \theta} + 0.02 \)

Functions:
- \( a = \frac{E_{a, \theta} \varepsilon_{c, \theta} - f_{0.2p, \theta}}{E_{a, \theta} \varepsilon_{c, \theta} - f_{0.2p, \theta}} \)
- \( b = \frac{1 - \varepsilon_{c, \theta} E_{a, \theta} f_{0.2p, \theta}}{E_{a, \theta} \varepsilon_{c, \theta} f_{0.2p, \theta} - 1} \)
- \( c^2 = (\varepsilon - \varepsilon_{c, \theta}) \left( \varepsilon - \varepsilon_{c, \theta} + \frac{e}{E_{a, \theta}} \right) \)
- \( d^2 = \sigma(\varepsilon - \varepsilon_{c, \theta}) E_{a, \theta} + \sigma^2 \)
- \( e = \frac{f_{0.2p, \theta} - f_{0.2p, \theta}}{E_{a, \theta} \varepsilon_{c, \theta} \varepsilon_{u, \theta} - 2f_{0.2p, \theta} \varepsilon_{c, \theta}} \)

where:
- \( f_{u, \theta} \) is the tensile strength;
- \( f_{0.2p, \theta} \) is the proof strength at 0.2% plastic strain;
- \( E_{a, \theta} \) is the slope of the linear elastic range;
- \( E_{ct, \theta} \) is the slope of proof strength;
- \( \varepsilon_{c, \theta} \) is the total strain at proof strength;
- \( \varepsilon_{u, \theta} \) is the ultimate strain.

Fig. 6.12. Stress–strain relationship for stainless steel at elevated temperatures

<table>
<thead>
<tr>
<th>( \theta_a ) (°C)</th>
<th>( E_{a, \theta} / E_a )</th>
<th>( f_{0.2p, \theta} / f_y )</th>
<th>( f_{u, \theta} / f_y )</th>
<th>( k_{2%\theta} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.26</td>
</tr>
<tr>
<td>100</td>
<td>0.96</td>
<td>0.82</td>
<td>0.87</td>
<td>0.24</td>
</tr>
<tr>
<td>200</td>
<td>0.92</td>
<td>0.68</td>
<td>0.77</td>
<td>0.19</td>
</tr>
<tr>
<td>300</td>
<td>0.88</td>
<td>0.64</td>
<td>0.73</td>
<td>0.19</td>
</tr>
<tr>
<td>400</td>
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<td>0.60</td>
<td>0.72</td>
<td>0.19</td>
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<tr>
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<td>0.27</td>
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<td>0.00</td>
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<td>0.40</td>
</tr>
</tbody>
</table>
6.2.3. Light-gauge steel

Light-gauge steel sections can be produced in a large variety of sections and profiled sheeting. Traditionally, in building construction, the commonly used sections are cold-formed ‘C’ or ‘Z’ shapes used as roof purlins and side rails to support the cladding in industrial buildings. More recently, light-gauge sections have been widely used as steel frames, trusses, wall partitions, lintels, floor joists and storage racking.

The main advantage of light-gauge sections is the high strength-to-weight ratio at ambient temperature. For cold-formed sections, the strain-hardening caused by the cold-working process increases the yield strength and ultimate strength of the materials. However, these characteristics make them more vulnerable to fire exposure. Light-gauge sections have little fire resistance because they heat up quickly if directly exposed to fire due to their high section factors. The increase of mechanical strength due to strain-hardening will also be removed quickly during heating.

Although widely used in the UK, the performance of light-gauge steel in fire is only briefly described in the Eurocodes.

6.2.3.1. Thermal properties

The thermal properties of light-gauged steel should be assumed to be similar to the thermal properties of hot-rolled steel.

6.2.3.2. Mechanical properties

Annex E of EN 1993-1-2 provides reduction factors for the design strength and elastic modulus of Class 4 sections made of carbon steel at elevated temperatures as shown in Table 6.3 and Fig. 6.14, which can be used for light-gauge sections.

Table 6.3. Reduction factors for carbon steel for Class 4 sections at elevated temperatures

<table>
<thead>
<tr>
<th>Steel temperature $\theta_1$ (°C)</th>
<th>Reduction factor for the design strength $k_{p0,2,\theta}$</th>
<th>Reduction factor for the slope of the linear elastic range $k_{E,\theta} = E_{\theta}/E_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
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</tr>
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<td>100</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>200</td>
<td>0.89</td>
<td>0.90</td>
</tr>
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<td>300</td>
<td>0.78</td>
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</tr>
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</tr>
<tr>
<td>600</td>
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</tr>
<tr>
<td>700</td>
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<tr>
<td>1100</td>
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</tr>
<tr>
<td>1200</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
For simplicity, EN 1993-1-2 conservatively adopts 0.2% proof stress as the effective yield strength for the design of Class 4 steel sections at elevated temperatures. For hot-rolled and welded thin-walled sections, the reduction factor for the design strength $k_{p0.2}$ is taken relative to the yield strength at $20^\circ C f_y$ as follows:

$$k_{p0.2} = \frac{f_{p0.2}}{f_y}$$

where $f_{p0.2}$ is the 0.2% proof strength at steel temperature $\theta_a$ taken as the effective yield value.

For cold-formed light-gauge sections, the reduction factor for the design strength $k_{p0.2}$ is taken relative to the basic yield strength at $20^\circ C f_{yb}$ as follows:

$$k_{p0.2} = \frac{f_{p0.2}}{f_{yb}}$$

where $f_{yb}$ is the basic yield strength as defined in EN 1993-1-3.

The reduction factor for elastic modulus is assumed to be identical to that of carbon steel.

### 6.3. Concrete

Concrete is a non-homogeneous material whose fire performance is controlled by that of the aggregate and the cement paste. Concretes are conventionally grouped as normal-weight concrete and lightweight concrete, depending on the density of the aggregates used. EN 1994 does not cover the design of composite structures with concrete strength classes lower than C20/25 and LC20/25 and higher than C60/75 and LC60/75. For strengths higher than C60/75 guidance is provided in EN 1992-1-2.

Concrete has a low thermal conductivity (50 times lower than steel) and therefore heats up very slowly in a fire. It is the low thermal conductivity that provides good inherent fire resistance of concrete structures.

EN 1994-1-2 provides material properties for normal-weight siliceous concrete and lightweight concrete. For calcareous concrete material properties either the siliceous concrete properties can, conservatively, be used or reference made to EN 1992-1-2.

#### 6.3.1. Normal-weight concrete

This section provides the thermal and mechanical properties of normal-weight concrete with siliceous or calcareous aggregates in accordance with EN 1992-1-2. The concrete strength classes range from C12/15 to C50/60. The strength classification of C12/15 refers to a concrete grade with characteristic cylinder and cube strength of 12 N/mm$^2$ and 15 N/mm$^2$ respectively.

#### 6.3.1.1. Thermal properties

EN 1992-1-2 and EN 1994-1-2 provide the following models incorporating temperature effects for the thermal properties of normal-weight concrete.
The thermal strain (expansion) of concrete $c$ can be determined by:

For siliceous aggregates:

$$
\varepsilon_{c,\theta} = \begin{cases} 
-1.8 \times 10^{-4} + 9 \times 10^{-6}\theta + 2.3 \times 10^{-11}\theta^3 & \text{for } 20^\circ C \leq \theta_c \leq 700^\circ C \\
14 \times 10^{-3} & \text{for } 700^\circ C < \theta_c \leq 1200^\circ C 
\end{cases} (6.8a)
$$

For calcareous aggregates:

$$
\varepsilon_{c,\theta} = \begin{cases} 
-1.2 \times 10^{-4} + 6 \times 10^{-6}\theta + 1.4 \times 10^{-11}\theta^3 & \text{for } 20^\circ C \leq \theta_c \leq 805^\circ C \\
12 \times 10^{-3} & \text{for } 805^\circ C < \theta_c \leq 1200^\circ C 
\end{cases} (6.8b)
$$

where $\theta_c$ is the concrete temperature ($^\circ C$).

The thermal strain (expansion) of concrete $\varepsilon_{c,\theta}$ can be determined by:

For siliceous aggregates:

$$
\varepsilon_{c,\theta} = \begin{cases} 
900 & \text{for } 20^\circ C \leq \theta_c \leq 100^\circ C \\
900 + (\theta - 100) & \text{for } 100^\circ C < \theta_c \leq 200^\circ C \\
1000 + (\theta - 200)/2 & \text{for } 200^\circ C < \theta_c \leq 400^\circ C \\
1100 & \text{for } 400^\circ C < \theta_c \leq 1200^\circ C 
\end{cases} (6.9)
$$

The variation of the thermal elongation with temperature is shown in Fig. 6.15.

The specific heat of dry concrete $c_{c,\theta}$ (in J/kg K) (i.e. moisture content by weight $u = 0\%$) can be determined by:

For siliceous and calcareous aggregates:

$$
c_{c,\theta} = \begin{cases} 
1470 & \text{for } u = 1.5\% \\
2020 & \text{for } u = 3.0\% \\
5600 & \text{for } u = 10.0\% 
\end{cases} (6.10)
$$

The variation of the specific heat with temperature is shown in Fig. 6.16.

Where the moisture content $u$ is not considered explicitly in analysis, the specific heat of concrete may be modelled by peak value at $115^\circ C$ as given below:

$$
\varepsilon^* = \begin{cases} 
1470 & \text{for } u = 1.5\% \\
2020 & \text{for } u = 3.0\% \\
5600 & \text{for } u = 10.0\% 
\end{cases}
$$

The variation of the specific heat with temperature is shown in Fig. 6.16.
The value of $u = 10.0\%$ may occur for hollow sections filled with concrete. For other moisture contents, linear interpolation between the above given values is acceptable. The peak values of specific heat are shown in Fig. 6.16.

The variation of density of concrete $\rho_c$ with temperature is influenced by free water loss and is defined as follows:

$$\rho_c = \begin{cases} 
\rho_{c,20} & \text{for } 20^\circ\text{C} \leq \theta_c \leq 115^\circ\text{C} \\
\rho_{c,20}[1 - 0.02(\theta - 115)/85] & \text{for } 115^\circ\text{C} < \theta_c \leq 200^\circ\text{C} \\
\rho_{c,20}[0.98 - 0.03(\theta - 200)/200] & \text{for } 200^\circ\text{C} < \theta_c \leq 400^\circ\text{C} \\
\rho_{c,20}[0.95 - 0.07(\theta - 400)/800] & \text{for } 400^\circ\text{C} < \theta_c \leq 1200^\circ\text{C} 
\end{cases}$$

(6.11)

where $\rho_{c,20}$ is concrete density at ambient temperature.

The variation of the ratio of $\rho_c$ to $\rho_{c,20}$ with respect to temperature is shown in Fig. 6.17.

The thermal conductivity of concrete $\lambda_c$ (in W/m K), for $20^\circ\text{C} \leq \theta_c \leq 1200^\circ\text{C}$, can be determined between the lower and upper limit values as follows:

$$\lambda_c = \begin{cases} 
2 - 0.2451(\theta_c/100) + 0.0107(\theta_c/100)^2 & \text{for upper limit} \\
1.36 - 0.136(\theta_c/100) + 0.0057(\theta_c/100)^2 & \text{for lower limit} 
\end{cases}$$

(6.12)

The variation of the upper limit and lower limit of thermal conductivity with temperature is shown in Fig. 6.18.

6.3.1.2. Mechanical properties

Figure 6.19 illustrates the stress–strain relationship model for concrete under uniaxial compression at elevated temperatures in accordance with EN 1992-1-2\textsuperscript{23}, with the values for each of these parameters as a function of concrete temperatures given in Table 6.4.

Figure 6.20 shows the corresponding reduction in strength for normal weight and light-weight concrete. Figure 6.21 shows the typical stress–strain curves for normal-weight

**Fig. 6.17.** Density of concrete at elevated temperatures

**Fig. 6.18.** Thermal conductivity of concretes at elevated temperatures
concrete at elevated temperatures in accordance with the mathematical model given in Fig. 6.19. It can be seen that the peak compressive strength $f_{c,0}$ reduces and the corresponding strain increases with increasing temperature.

Conservatively, the tensile strength of concrete can be ignored. However, EN 1992-1-2 allows the tensile strength to be taken into account. The reduction of the characteristic tensile strength of concrete is governed by the coefficient $k_{c,t} = k_{c,1}(\theta)$ as given by:

$$f_{ck,t}(\theta) = k_{c,t}(\theta) f_{ck,1}$$  \hspace{1cm} (6.13)

Table 6.4. Parameters for stress–strain relationship of normal-weight and lightweight concrete at elevated temperatures

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Siliceous NWC</th>
<th></th>
<th></th>
<th></th>
<th>Carbonate NWC</th>
<th></th>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th>Concrete</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>f_{c,0}/f_{ck}</td>
<td>e_{c,1}/\epsilon</td>
<td>\epsilon_{c,1}</td>
<td>\epsilon_{c,1}</td>
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<td>\epsilon_{c,1}</td>
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<td>e_{c,1}</td>
<td>\epsilon_{c,1}</td>
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<td>0.60</td>
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</tr>
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<td>0.0400</td>
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<td>0.0250</td>
<td>0.0400</td>
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<td>0.0475</td>
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<td></td>
</tr>
</tbody>
</table>

Fig. 6.19. Stress–strain relationship of concrete under compression at elevated temperatures.

Fig. 6.20. Reduction in strength for normal- and lightweight concretes at elevated temperatures.
6.3.2. Lightweight concrete

This section provides the thermal and mechanical properties of lightweight concrete (LWC) in accordance with EN 1994-1-2. The density of unreinforced lightweight concrete considered in this code shall be in the range 1600–2000 kg/m³.

6.3.2.1. Thermal properties

Lightweight concrete has very good thermal properties with half the thermal expansion and thermal conductivity of normal-weight concrete. EN 1994-1-2 provides the following models for the thermal properties of lightweight concrete.

The related thermal elongation $l_{el}$ of lightweight concrete may be determined from:

$$ l_{el} = 8 \times 10^{-6} (\theta_c - 20) $$

where:
- $l$ is the length at room temperature of lightweight concrete member;
- $\Delta l$ is the temperature-induced elongation of lightweight concrete member; and
- $\theta_c$ is the concrete temperature (°C).

The variation of the thermal elongation with temperature is shown in Fig. 6.15. The specific heat $c_c$ (in J/kg K) of lightweight concrete may be considered to be independent of the concrete temperature:

$$ c_c = 840 $$

The value of specific heat of lightweight concrete is shown in Fig. 6.16. The thermal conductivity $\lambda_c$ (in W/m K) of lightweight concrete may be determined from the following:

$$ \lambda_c = \begin{cases} 1.0 - (\theta_c / 1600) & \text{for } 20^\circ C \leq \theta_c \leq 800^\circ C \\ 0.5 & \text{for } \theta_c > 800^\circ C \end{cases} $$

The variation of the thermal conductivity with temperature of lightweight concrete is shown in Fig. 6.18.

6.3.2.2. Mechanical properties

EN 1994-1-2 adopts the same stress–strain relationship model for lightweight concrete as normal-weight concrete. The model is given in Fig. 6.19 and Table 6.4. However, the code only provides the values for the reduction of strength $k_{c,\theta} = f_{c,\theta}/f_{ck}$ as shown in Table 6.4. The values of the strain corresponding to $f_{c,\theta}$ should be obtained from tests. The strength
reduction factor of lightweight concrete is compared to that of normal-weight concrete in Fig. 6.20. Lightweight concrete has better strength retention than normal-weight concrete.

6.3.3. High-strength concrete

The strength of normal-weight concrete (NWC) is typically limited by the strength of the cement matrix. It is commonly known that concrete compressive strength is inversely related to the water–cement ratio. The advance of material technology and production has led to higher grades of concrete, ranging from 50 to 130 MPa. High-strength concrete (HSC) is mainly achieved by either adding water-reducing admixtures to obtain a low water–cement ratio, or adding silica fume. Consequently, HSC has lower permeability and water content compared with NWC.

HSC provides superior structural properties including higher strength, higher stiffness and better durability. Hence, HSC is usually considered as a high-performance construction material, and HSC structural elements have been widely used in construction projects around the world, including bridges, high-rise buildings and special structures.

Since the 1980s, many fire tests have been conducted to investigate the material properties of HSC at elevated temperatures. The test results have generally identified the two main differences in the behaviour of HSC at high temperatures from that of normal-strength concrete:

1. The strength loss of HSC at elevated temperatures is more pronounced.
2. The susceptibility of HSC to explosive spalling at temperatures below 400°C.

With the exception of these two points, it is commonly suggested that HSC can be treated as conventional, normal-strength concrete in the fire engineering design. This hypothesis has been adopted in the Eurocodes.

This section provides the thermal and mechanical properties of HSC in accordance with EN 1992-1-2. The code divides high-strength concrete into three classes, namely:

1. Class 1 for concrete C55/67 and C60/75
2. Class 2 for concrete C70/85 and C80/95
3. Class 3 for concrete C90/105.

The strength notation of C55/67 refers to a concrete grade with characteristic cylinder and cube strength of 55 N/mm² and 67 N/mm², respectively.

<table>
<thead>
<tr>
<th>Concrete temperature</th>
<th>$f_{c0} / f_{ck}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class 1</td>
</tr>
<tr>
<td>$\theta$ (°C)</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1.00</td>
</tr>
<tr>
<td>50</td>
<td>1.00</td>
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<tr>
<td>100</td>
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<tr>
<td>200</td>
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<tr>
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<td>0.30</td>
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<td>0.15</td>
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<td>900</td>
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<tr>
<td>1000</td>
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</tr>
<tr>
<td>1200</td>
<td>0.00</td>
</tr>
</tbody>
</table>
6.3.3.1. Thermal properties

EN 1992-1-2 assumes that HSC has the same thermal properties as NWC. However, it is noteworthy that the actual thermal conductivity of HSC may be higher than that for NWC.

6.3.3.2. Mechanical properties

Based on a limited number of test results, EN 1992-1-2 provides the reduction of strength $f_{c0}/f_{ck}$ of HSC at elevated temperatures as shown in Table 6.5 and Fig. 6.22. Compared to normal-weight concrete with siliceous aggregate, HSC generally suffers greater strength reduction at high temperatures, in particular at low temperatures below 300°C.

6.4. Reinforcing steel

EN 1994-1-2 covers hot-rolled and cold-worked reinforcing steel. It is assumed that pre-stressing steel will not be used in composite structures.

The thermal and mechanical properties of hot-rolled reinforcing steel are assumed to be the same as hot-rolled steel, covered in Section 6.2.1. For cold-worked reinforcing steel the thermal properties are assumed to be the same as hot-rolled steel but the mechanical properties vary. The three main parameters for cold-worked reinforcing steel representing the stiffness, extent of the proportional limit, and the yield strength, for a given temperature, are shown in Table 6.6. The values are represented as reduction factors based on values at ambient temperature.

Table 6.6. Reduction factors for cold-worked reinforcing steel

<table>
<thead>
<tr>
<th>Steel temperature $\theta_i$ (°C)</th>
<th>Reduction factor for effective yield strength $f_{yp}/f_y$</th>
<th>Reduction factor for proportional limit $f_{sp}/f_{y}$</th>
<th>Reduction factor for effective yield strength $f_{yp}/f_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>100</td>
<td>1.00</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>200</td>
<td>0.87</td>
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</tr>
<tr>
<td>300</td>
<td>0.72</td>
<td>0.81</td>
<td>1.00</td>
</tr>
<tr>
<td>400</td>
<td>0.56</td>
<td>0.63</td>
<td>0.94</td>
</tr>
<tr>
<td>500</td>
<td>0.40</td>
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</tr>
<tr>
<td>1100</td>
<td>0.02</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>1200</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
6.5. Bolts and welds

Annex D of EN 1993-1-2 provides limited information on the fire performance of bolts and welds, comprising mechanical properties with varying temperature relative to the adjoining beam. For bolted connections, EN 1993-1-2 states that net-section failure at fastener holes need not be considered, provided that there is a fastener in each hole, since it is assumed that the steel temperature is normally lower at connections due to the presence of additional material.

Based on a limited number of tests, the code assigns the same strength reduction factor $k_{b,\theta}$ for bolts in shear and tension, regardless of bolt types. For friction grip bolts, it is assumed that the bolts slip in fire and the fire resistance of a single bolt may be designed for shear and bearing.

The fire design resistance for bolts should be determined from the following:

For shear resistance:

$$F_{v,1,Rd} = F_{v,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,f}}$$

For bearing resistance:

$$F_{b,1,Rd} = F_{b,Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,f}}$$

For tension resistance:

$$F_{\text{ten},1,Rd} = F_{\text{ten},Rd} k_{b,\theta} \frac{\gamma_{M2}}{\gamma_{M,f}}$$

where:

$F_{b,Rd}$ is the design bearing resistance according to EN 1993-1.8;

$F_{t,Rd}$ is the design tension resistance according to EN 1993-1.8;

$F_{v,Rd}$ is the design shear resistance of the bolt according to EN 1993-1.8;

$k_{b,\theta}$ is the reduction factor for appropriate bolt temperature from Table 6.7;

$\gamma_{M2}$ is the partial safety factor at normal temperature; and

$\gamma_{M,f}$ is the partial safety factor for fire conditions.

The variation of the strength reduction factor is illustrated in Fig. 6.23.

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

<table>
<thead>
<tr>
<th>Temperature $\theta$ ($^\circ$C)</th>
<th>Bolts in shear and tension $k_{b,\theta}$</th>
<th>Fillet welds $k_{w,\theta}$</th>
<th>Butt welds $k_{w,\theta}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>100</td>
<td>0.968</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>200</td>
<td>0.935</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>300</td>
<td>0.903</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>400</td>
<td>0.775</td>
<td>0.876</td>
<td>1.000</td>
</tr>
<tr>
<td>500</td>
<td>0.550</td>
<td>0.627</td>
<td>0.780</td>
</tr>
<tr>
<td>600</td>
<td>0.220</td>
<td>0.379</td>
<td>0.470</td>
</tr>
<tr>
<td>700</td>
<td>0.100</td>
<td>0.130</td>
<td>0.230</td>
</tr>
<tr>
<td>800</td>
<td>0.067</td>
<td>0.074</td>
<td>0.074</td>
</tr>
<tr>
<td>900</td>
<td>0.033</td>
<td>0.018</td>
<td>0.018</td>
</tr>
<tr>
<td>1000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>
The design resistance per unit length of a fillet weld in fire should be determined from:

\[ F_{w,Rd} = F_{w,Rd}k_{w,\theta} \frac{\gamma_{M2}}{\gamma_{M,fi}} \]

where:

- \( F_{w,Rd} \) is the design weld resistance per unit length according to EN 1993-1-8; and
- \( k_{w,\theta} \) is the reduction factor for appropriate weld temperature from Table 6.7.

The variation of strength reduction factor is illustrated in Fig. 6.23. Fillet welds are considered to have better fire performance than bolts, but have a lower strength retention compared to butt welds or the parent metal.
CHAPTER 7

Design of tension members

7.1. Introduction
This chapter gives guidance on the design of steel tension members. Tension members can be used in many applications, some of which are detailed below.

- Single angles, tees, channels and structural hollow sections are used in light trusses and lattice girders.
- Single sections, compound sections consisting of double angles or channels and bars and flats are used as bracing members in buildings.
- Ropes and cables are used as main cables and deck suspension cables in cable-stayed structures and suspension bridges.
- Heavy rolled sections and heavy compound sections of built-up H and box sections are used as the hangers in suspended structures.

Typical section types and examples of where tension members are used in buildings and bridges are given in Figs 7.1 and 7.2 respectively.

EN 1993: Part 1.2 permits two methods of assessing the fire resistance of steel members in tension. The first, the ‘design resistance’ method, consists of calculating the design resistance of a member based on the distribution of temperature through its cross-section, the area of the cross-section and its reduced material properties at elevated temperature. The second, the ‘critical temperature’ method, consists of calculating the temperature at which the member will fail, assuming a uniform temperature distribution and a given degree of utilization. Both methods are explained in the next sections.

7.2. Design resistance method
EN 1993: Part 1.2 gives two approaches for calculating the design resistance of a tension member at elevated temperature. The first approach can be used for a tension member with
a non-uniform temperature distribution across its cross-section, while the second is for a
tension member with a uniform temperature distribution across its cross-section. Alternatively,
the design resistance of a tension member with a non-uniform temperature distribution across
its cross-section can be conservatively taken as equal to the design resistance of a member with
a uniform temperature distribution provided the uniform temperature taken is equal to the
maximum temperature in the section with a non-uniform distribution.

7.2.1. Non-uniform temperature distribution
EN 1993: Part 1.2 gives the design resistance, \( N_{\text{fi,Rd}} \), at time, \( t \), of a tension member with a
non-uniform temperature distribution across its cross-section as:

\[
N_{\text{fi,Rd}} = \sum_{i=1}^{n} A_i k_{y,\theta_i} f_y / \gamma_{\text{M,fi}}
\]  

(7.1)

where:

- \( A_i \) is an elemental area of the cross-section with a temperature \( \theta_i \);
- \( k_{y,\theta_i} \) is the reduction factor for the yield strength of steel at temperature \( \theta_i \);
- \( \theta_i \) is the temperature in the elemental area \( A_i \);
- \( f_y \) is the yield strength at 20°C; and
- \( \gamma_{\text{M,fi}} \) is the partial factor for the relevant material property for the fire situation.
In this method the cross-section is divided into a number of discrete elements and the area, temperature, reduction factor and yield strength for each element determined. The above equation is then used to sum together the resistances of each of the individual elements to obtain the resistance of the complete cross-section. Any convenient subdivision may be used but for accurate results the section should be divided into elements that have an approximately uniform temperature distribution. Where the temperature distribution varies over the area of the element, the highest temperature should be used.

A worked example follows.

**Example 7.1: Tension resistance of equal angle**

It is required to calculate the tension resistance of the tension member shown in Fig. 7.3(a). The temperature distribution is non-uniform over the cross-section, varying linearly from 600°C at the bottom of the angle to 200°C at the top of the angle.

### Section size and material properties

- **Section size:** 100 × 100 × 15 mm equal angle
- **Steel grade:** S275

### Calculation procedure

The angle can be conveniently subdivided into its vertical and horizontal legs, as shown in Fig. 7.3(b). The maximum temperature in each of these elements is 600°C and 540°C respectively.

The tension resistance of element 1 can be calculated as follows:

\[
N_{fi, Rd, 1} = A_1 k_y, f_y, f_{y1} / \gamma_{M, f, 1}
\]

![Fig. 7.3. Tension member subject to non-uniform temperature distribution: (a) non-uniform temperature distribution; (b) maximum temperature in each leg; and (c) subdividing vertical leg. (All dimensions in mm)](image)
7.2.2. Uniform temperature distribution

EN 1993-1-2 gives the design resistance, $N_{fi,t,Rd}$, of a tension member with a uniform temperature across its cross-section as:

$$N_{fi,t,Rd} = k_{y,\theta} N_{Rd} \left( \gamma_{M,1} / \gamma_{M,R} \right)$$  \hspace{1cm} (7.2)

where:

- $k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature, $\theta$; and
- $N_{Rd}$ is the design resistance of the gross cross-section $N_{pl,Rd}$ for normal temperature design according to EN 1993-1-1.

This method is based on the design method for tension members at the cold condition given EN 1993: Part 1.1 but introduces a reduction factor $k_{y,\theta}$ to account for the reduction in the material properties at elevated temperature. However, there is one key difference between this approach and that used for cold designs and it concerns the performance of the member at net sections. In the fire condition it is assumed that at a bolt hole, provided the hole is filled with a bolt, the section around the hole will not be heated to the same extent as the rest of the member. This is because locally the thermal mass is increased due to the presence of the bolt. The result of this effect is to increase the resistance of the member at its net section to such an extent that gross section properties can be used in place of the net section properties.

7.3. Critical temperature method

The critical temperature, $\theta_{a,cr}$, for a steel tension member at time, $t$, with a uniform temperature distribution is the temperature at which the capacity of the member is reduced to the applied load. In EN 1993 the critical temperature may be determined for any degree of utilization from the following expression:

$$\theta_{a,cr} = 39.19 \ln \left[ \frac{1}{0.9674 \mu_o^{1.333}} - 1 \right] + 482$$  \hspace{1cm} (7.3)

where:

- $\mu_o$ is the degree of utilization in the member and is given by the following expression:

$$\mu_o = E_{n,d} / R_{n,d,\theta}$$  \hspace{1cm} (7.4)
where:
\( R_{fi,d,0} \) is the value of \( R_{fi,d,1} \) for time \( t = 0 \) (i.e. in the cold condition);
\( R_{fi,d,1} \) is as given in Chapter 5 of this handbook; and
\( E_{fi,d} \) is as given in Chapter 5 of this handbook.

The degree of utilization can be defined as the ratio of applied load at the fire Limit State (\( E_{fi,d} \)) to the capacity of the member under normal conditions (\( R_{fi,d,0} \)). For tension members the capacity at fire time \( t = 0 \) can be taken as its ambient temperature capacity. The capacity of a tension member at ambient temperature can be determined from the appropriate expression given in EN 1993: Part 1.1.

Alternatively, for tension members \( \mu_o \) may be conservatively obtained from the following expression:

\[ \mu_o = \eta_{f1}(\gamma_{M,R}/\gamma_{M,1}) \]  \hspace{1cm} (7.5)

where:
\( \eta_{f1} \) is the reduction factor as given in Chapter 5 of this handbook.

This method assumes that the member has been designed to its full strength at the cold condition and therefore the utilization factor may be calculated from a ratio of the applied load at the fire Limit State to the ultimate design load at the ambient condition. If the structure is subject to a combination of dead load (\( G \)) and imposed load (\( Q \)), the utilization factor can therefore be expressed in the following form:

\[ \mu_o = \frac{G + \psi Q}{1.35G + 1.5Q}(\gamma_{M,R}/\gamma_{M,1}) \]

where \( \psi \) is the combination factor for the imposed load under fire conditions and the factors 1.35 and 1.5 are the partial safety factors for dead load and imposed load.

Table 7.1 compares the temperatures predicted by the critical temperature method of EN 1993-1-2 with those from BS 5950: Part 8 for tension members. From this table it can be seen that the critical temperature method gives very similar results to the limiting temperature, load-ratio method given in BS 5950: Part 8.

A worked example follows.

**Example 7.2: Calculation of critical temperature**

In this example calculate the critical temperature for the single angle in the previous worked example, assuming the member has an applied load of 500 kN.

The tension resistance at time \( t = 0 \) is \( R_{fi,d,0} = 2800 \times 275/1000 = 770 \) kN. Therefore the utilization factor is \( \mu_o = 500/770 = 0.649 \). From equation (7.3) the critical temperature of the member is 540°C.
CHAPTER 8

Design of compression members

8.1. Introduction
This chapter gives guidance on the design of members subjected mainly to compression (hereafter to be referred to as columns). It will consider steel, composite and reinforced concrete columns.

In the various Eurocodes, the aim of the design calculations is to ensure that a structural member has sufficient load-carrying capacity at elevated temperatures to resist the applied load in the structural member at the fire limit state, this design philosophy being identical to that at ambient temperature. Therefore, temperatures in the structural member should be obtained first. For steel columns, it is relatively simple to calculate the steel temperature using the equations in Chapter 4, provided one has appropriate values of thermal properties of the fire protection materials. For reinforced concrete columns, EN 1992-1-2 contains a number of design graphs to give temperature distributions in a range of rectangular cross-sections at different standard fire exposure times (e.g. Fig. 4.8 of this book). For composite members and more general cases of reinforced concrete members, because the temperature distribution in the cross-section of a member is not uniform, no simple calculation procedure is available and it is necessary to employ numerical procedures, such as the finite-element, or the finite-difference method to evaluate the non-uniform temperature distribution. EN 1994-1-2 has provided some general heat transfer equations; however, detailed numerical implementation of these equations is beyond the scope of this book. It is assumed that the reader has access to such a tool. To summarize, this handbook will assume that the temperature distribution in a structural member is available as input.

The various Eurocodes contain a number of methods to evaluate the load-carrying capacity of different types of compressive members. For steel members, EN 1993-1-2 includes the simplified calculation method and the critical temperature method. For composite and reinforced concrete members, EN 1994-1-2 and EN 1992-1-2 include the tabulated method, the simplified calculation method and the advanced calculation method.

The tabulated method contains a number of design tables to directly relate various column design parameters to the available column standard fire resistance rating. These tables are generally based on standard fire resistance tests or results of numerical calculations and should be used wherever possible because of their simplicity. This handbook will not repeat these tables.

The advanced method is based on general structural engineering principles and will inevitably involve using numerical analysis procedures. This subject is beyond the scope of this handbook. Therefore, this handbook will mainly discuss implementations of the various simple calculation methods.
8.2. Effective length of columns in fire

The effective length of a column is a key parameter when calculating its compression resistance. The column effective length for fire limit stage design may be different from that at ambient temperature.

This comes about when a column is exposed to fire attack but the column is contained within a fire-resistant compartment, the column rotational stiffness is reducing at elevated temperatures but the rotational stiffness of the cold adjacent structure is unchanged. In relative terms, the column is provided with increasing rotational restraint. The various Eurocodes recognize this fact and recommend using reduced column effective lengths in fire compared to that at ambient temperature. Furthermore, Eurocodes assume that the relative stiffness of the adjacent cold structure to the heated column approaches infinite so that the heated column may be considered to be rotationally fixed at the ends. Figure 8.1 illustrates the Eurocode recommendations. To apply the Eurocode design recommendations as illustrated in Fig. 8.1, it should be pointed out that only the adjacent cold columns continuous out of the fire-resistant compartment floors from the heated column would provide the heated column with reliable enhanced rotational restraint. This comes about because the adjacent horizontal members would be exposed to the same fire attack as the heated column and may not provide the heated column with reliable rotational restraint.

![Figure 8.1](a) (b) (c)

**Fig. 8.1.** Effective length of column in braced frame in fire: (a) section through building; (b) deformation mode at room temperature; and (c) deformation mode at elevated temperature

8.3. Axially loaded steel columns

Because it is necessary to consider stability in column design, the critical temperature method in clause 4.2.4 of EN 1993-1-2 should not be applied. The simplified calculation method for steel columns should follow clauses 4.2.3.2 and 4.2.3.6 of EN 1993-1-2. In this method, the column temperature is the starting-point of the design calculations. At the design column temperature, the reduced column strength is calculated and compared to the applied load in the column in fire. The column is considered to be safe if the residual column strength exceeds the applied column load in fire. In the simplified calculation method, the applied load in the column is assumed to remain constant during fire exposure. If the design objective is to find the required column fire protection thickness, an iterative procedure will be necessary. Figure 8.2 illustrates this iterative process.

The design method in EN 1993-1-2 is similar to that in EN 1993-1-1 for steel columns at ambient temperature. Nevertheless, there are a number of differences. Apart from the difference in column effective length as described in section 8.2 of this chapter, other differences are concerned with column cross-section classification for local buckling and the column global buckling curve. For column cross-section classification for local buckling,
this should be performed for fire design using a reduced value for $\varepsilon$ as given by:

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

(8.1)

The coefficient 0.85 is an approximate value for $\sqrt{k_{E,\theta}k_{Y,\theta}}$ where $k_{E,\theta}$ and $k_{Y,\theta}$ are the reduction factors for modulus of elasticity and effective yield strength of steel at temperature $\theta$ (Table 6.1).

Using the plate width to thickness ratio limits in EN 1993-1-1 for the flanges and the web of rolled sections under compression, it is easy to verify that all Corus-produced UC (universal column) cross-sections are class 3 or better if the steel grade is S275. If the steel grade is S355, only sections $356^{\times}368^{\times}129$UC and $152^{\times}152^{\times}23$UC are class 4 cross-sections.

8.3.1. Uniformly heated column with class 1, 2 or 3 cross-section

For columns with class 1, 2 or 3 cross-section, there is no need to consider local buckling in the design calculations and the column design compressive resistance $N_{b,f,1,Rd}$ is determined from:

$$
N_{b,f,1,Rd} = \chi_{fi}Af_{y}/\gamma_{M,fi}
$$

(8.2)

where $A$ is the column gross cross-section area and $\gamma_{M,fi}$ is the material partial safety factor for steel at the fire limit state. $\chi_{fi}$ is the column strength reduction factor which is a function of the column slenderness $\lambda_{fi}$ and is determined from:

$$
\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\frac{\varphi_\theta^2}{\lambda_{fi}^2}}} \quad \text{with} \quad \varphi_\theta = \frac{1}{2}[1 + \alpha\lambda_{fi} + \lambda_{fi}^2] \quad \text{and} \quad \alpha = 0.65\sqrt{235/f_y}
$$

(8.3)

When calculating the column slenderness $\lambda_{fi}$, in addition to taking into consideration the reduced column effective length as described in section 8.2, it is also necessary to include the effect of different changes in the effective yield strength and modulus of elasticity of steel at
elevated temperatures. EN 1993-1-2 gives the column slenderness $\bar{\lambda}_0$ as:

$$\bar{\lambda}_0 = \bar{\lambda} \sqrt{\frac{k_y \beta}{k_E \beta}}$$

(8.4)

where $\bar{\lambda}$ is the column slenderness at ambient temperature, but calculated using the reduced column effective length in fire as described in section 8.2.

If the aim of design calculations is to find the limiting temperature of the column, the column temperature is unknown and the second term on the right-hand side of equation (8.4) cannot be calculated. However, within the practical range of steel temperatures (300–800°C), the second term on the right-hand side of equation (8.4) has a value of about 1.2. Therefore, the column slenderness in fire may approximately be calculated using:

$$\bar{\lambda}_0 = 1.2 \bar{\lambda}$$

(8.5)

A worked example follows.

Example 8.1: Calculation of the limiting temperature for a steel column

It is required to calculate the limiting temperature of a steel column.

**Input parameters**

- Column section size: $305 \times 305 \times 118$ UC
- Height: 4.2 m between two fire-resistant floors with the column continuous at both ends
- Steel grade: S275

**Applied compression loads**

- Permanent load: 1000 kN
- Variable load: 1200 kN

**Calculation procedures**

**Step 1: Column slenderness for the fire limit state**

The column effective length at fire limit is $L_{e,fi} = 0.5 \times 4.2 = 2.1$ m, giving the Euler buckling load as:

$$N_{cr} = \frac{\pi^2 EI}{L_{e,fi}^2} = \frac{\pi^2 \times 205 \times 90 \times 590}{2.1^2} \times 1000 = 41562 \text{ kN}$$

The column plastic resistance at ambient temperature is:

$$N_u = f_y A = 0.275 \times 15000 = 4125 \text{ kN}$$

The column slenderness at ambient temperature is:

$$\bar{\lambda} = \sqrt{\frac{N_u}{N_{cr}}} = 0.315$$

The approximate column slenderness for fire design is:

$$\bar{\lambda}_0 = 1.2 \times \bar{\lambda} = 0.378$$

**Step 2: Column limiting temperature**

Assuming a partial safety factor of 0.5 for the variable load (Table 5.1), the applied load in column at the fire limit state is $1000 + 1200 \times 0.5 = 1600$ kN.
8.3.2. Uniformly heated column with class 4 cross-section
For columns with class 4 cross-section where local buckling is important, EN 1993-1-2 presents two alternative methods of assessing the column fire resistance. In the simpler approach, EN 1993-1-2 recommends a conservative column limiting temperature of 350°C. The alternative approach is presented in Annex E of EN 1993-1-2. In this approach, the effective width is used to consider the effects of local buckling and EN 1993-1-2 recommends using the identical effective widths as at ambient temperature. Afterwards, the design calculations should proceed in the same way as for columns with class 1, 2 or 3 cross-section, except that the gross cross-sectional properties of a class 1, 2 or 3 cross-section should be replaced by the effective cross-sectional properties of a class 4 cross-section.

8.3.3. Uniformly heated column with combined axial load and bending moment
For uniformly heated columns under the combined action of axial load and bending moment, EN 1993-1-2 provides two sets of equations, one for members with class 1, 2 and 3 cross-sections and one for members with class 4 cross-sections. These equations are identical to those at ambient temperature in EN 1993-1-1, but the member resistance under individual axial load or bending at elevated temperatures should be used.

8.3.4. Non-uniformly heated steel columns
There are a number of practical situations where a steel column is non-uniformly heated, for example a column forming part of a wall where fire exposure is from one side. At present, the EN 1993-1-2 recommendation on this issue is rather simplistic, stating that ‘the design resistance of a compression member with a non-uniform temperature distribution may be taken as equal to the design resistance of a compression member with a uniform temperature θ equal to the maximum steel temperature’. For columns with a class 1, 2 or 3 cross-section, the EN 1993-1-2 recommendation is reasonable if the column slenderness is either very low or very high. For columns with medium slenderness (40 < λ < 100), the EN 1993-1-2 recommendation may not be safe. It may be necessary to employ more advanced calculation methods. For columns with a class 4 cross-section, Annex E of EN 1993-1-2 may be used, but the effect of thermal bowing induced bending moment should be included.

8.4. Axially loaded composite column
EN 1994-1-2 provides two alternative simplified calculation methods for composite columns. The main text (clause 4.3.5) of the code gives the general design method for all types of composite columns, with specific limits of application for steel sections with partial concrete encasement (clause 4.3.5.2) and concrete-filled hollow sections (clause 4.3.5.3). Two annexes give alternative specific calculation methods for partially encased steel sections (Annex G) and concrete-filled hollow steel sections (Annex H). This guide will describe the design
procedure using the general design method and provide some design aids for implementation of the alternative calculation methods of the two annexes.

### 8.4.1. General design method

The general design method involves the following steps.

1. **Temperatures in the composite column**
   
   Since the thermal conductivity of concrete is low, the temperature distribution in a composite cross-section is highly non-uniform. No simplified calculation method is available for heat transfer analysis of this type of column and numerical heat transfer analysis software should be used to perform this task.

2. **Cross-sectional properties at elevated temperatures**

   Two values of the composite cross-section should be calculated: the cross-section plastic resistance to axial compression and the cross-section effective flexural stiffness. When performing these calculations, the composite cross-section is divided into a large number of blocks each having approximately the same temperature. The property of the composite cross-section is obtained by summing up contributions of all the blocks. Use the subscripts \( \text{a, s and c} \) to represent the steel profile, reinforcing bars and concrete respectively.

   The plastic resistance to axial compression of the composite cross-section is:
   \[
   N_{\text{fi,pl,Rd}} = \sum_j (A_a f_{ay,\theta})/\gamma_{M,\text{fi,a}} + \sum_k (A_s f_{sy,\theta})/\gamma_{M,\text{fi,s}} + \sum_m (A_c f_{c,\theta})/\gamma_{M,\text{fi,c}} \tag{8.6}
   \]
   where the composite cross-section is divided into \( j \)-blocks of steel, \( k \) reinforcing bars and \( m \)-blocks of concrete. For each block of the composite cross-section, \( A \) is its area and \( f \) the design strength of its material at the appropriate temperature. The \( \gamma \)-factors are material partial safety factors for fire design.

   Similarly, the effective flexural stiffness of the composite cross-section is:
   \[
   (EI)_{\text{fi,eff}} = \sum_j \left( \varphi_{a,\theta} E_a \varepsilon_{a,\theta,\text{sec}} I_{a,\theta} \right) + \sum_k \left( \varphi_{s,\theta} E_s \varepsilon_{s,\theta,\text{sec}} I_{s,\theta} \right) + \sum_m \left( \varphi_{c,\theta} E_c \varepsilon_{c,\theta,\text{sec}} I_{c,\theta} \right) \tag{8.7}
   \]
   where \( E \) is the initial modulus of elasticity of steel or reinforcement at temperature \( \theta \). For concrete, the secant modulus should be used, this being calculated as the design compression strength of concrete \( f_{c,\theta} \) divided by the corresponding strain at peak stress \( \varepsilon_{c1,\theta} \), whose values are given in Table 6.4. For each block of the composite cross-section, \( I \) is its second moment of area around the relevant axis of the entire cross-section.

   Equation (8.7) contains a set of reduction coefficients \( \varphi \), which have been introduced to account for the effect of thermal stresses caused by non-uniform temperature distribution in the composite cross-section and unequal thermal expansions in different materials. In principle, this non-uniform thermal strain distribution gives rise to non-uniform mechanical strain distribution in the composite cross-section. Since the stress–strain relationships of steel and concrete are non-linear, this non-uniform distribution of mechanical strains would result in a different distribution of material stiffness around the composite cross-section from that based on uniform strain distribution. The general calculation method in clause 4.3.5.1(5) of EN 1994-1-2 does not give values to these reduction coefficients. In Annex G for partially encased steel sections, different values are given for different standard fire resistance ratings. These values may be used in the general calculation method for design under the standard fire exposure. Since the general calculation method is not limited to the standard fire exposure and may be applied to other types of fire exposure, e.g. parametric fire curves in EN 1991-1-2 (Chapter 3 of this guide), it is recommended that the designer should use the lower values in Annex G so as to be on the safe side. For concrete-filled steel hollow sections, Annex H does not include these reduction coefficients, implying that they have a value of unity. This should be adopted when using the general calculation method, irrespective of the type of fire exposure.
3. Column resistance to axial compression

Using the effective flexural stiffness of the composite cross-section, the Euler buckling load of the composite column is calculated by:

\[ N_{fi,cr} = \pi^2 (EI)_{fi,eff} L_0^2 / C_{25}^2 \]  

(8.8)

where the effective length of the composite column in fire \( L_{e,f} \) should be evaluated following Section 8.2.

The non-dimensional slenderness ratio of the composite column in fire is:

\[ \bar{\lambda}_0 = \sqrt{\frac{N_{fi,pl,R}}{N_{fi,cr}}} \]  

(8.9)

in which \( N_{fi,pl,R} \) is the value of \( N_{fi,pl,Rd} \) according to equation (8.6), but with the material partial safety factors set to unity.

The resistance of the composite column to axial compression is obtained from:

\[ N_{fi,Rd} = \chi N_{fi,pl,Rd} \]  

(8.10)

in which the buckling curve 'c' of EN 1993-1-1 should be used to obtain the reduction coefficient \( \chi \) irrespective of the type of composite cross-section.

When using the general calculation method, the limits of applicability in clause 4.3.5.2(2) for partially encased steel sections and in clause 4.3.5.3(2) for concrete-filled hollow sections should be observed.

The general design method is only applicable to axially loaded composite columns. For composite columns with eccentricity, the alternative design methods in Annex G and Annex H should be followed.

Example 8.2: General design method

**Input information**

The cross-section for this example is a concrete-filled circular hollow section (CHS) 355.6 × 12.5 mm. The material properties are as follows:

- Structural steel: \( f_y = 275 \text{ N/mm}^2, E_a = 210 \text{ kN/mm}^2, \gamma_{ma} = 1.0 \)
- Concrete: \( f_{ck} = 25 \text{ N/mm}^2, E_{cm} = 30.5 \text{ kN/mm}^2, \gamma_{ma} = 1.1 \)

The column is 4.5 m high and is bounded by fire-resistant floors and continuous at both ends.

This example will determine the compressive strength of the column at standard fire-resistance rating R60, with no external fire protection and no internal reinforcement.

**Results of calculation**

**Step 1: Temperature distribution in the composite cross-section**

For simplicity, the approximate method of Lawson and Newman\(^{40}\) is used. In this example, the steel shell is treated as one layer of uniform temperature and the concrete core is divided into ten layers. To account for the steep temperature gradient at the outer layers of the concrete core, each of the five outer layers of the concrete core will have a thickness of 10 mm. The remaining inner concrete core is divided into five layers, each having a thickness of 23.06 mm.

Assuming an ambient temperature of 20°C, the approximate method of Lawson and Newman\(^ {40}\) gives temperatures for each layer of the composite cross-section in Table 8.1.

**Step 2: Composite cross-section properties**

At the elevated temperatures in Table 8.1, the reduced effective yield strength and modulus of elasticity of steel, the reduced design strength and secant modulus of concrete can be obtained from Tables 6.1 and 6.4 respectively. These values are also listed in Table 8.1.
### Table 8.1. Results of calculation for worked Example 8.2

<table>
<thead>
<tr>
<th>Layer</th>
<th>Inner–outer radius (mm)</th>
<th>( \theta ) (°C)</th>
<th>( k_{f,\theta} )</th>
<th>( \varepsilon_{cu} \times 1000 )</th>
<th>( f_0 ) (N/mm²)</th>
<th>( E_0 ) (N/mm²)</th>
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<td>165.3–177.8</td>
<td>827.1</td>
<td>0.066</td>
<td>N/A</td>
<td>18.15</td>
<td>8589</td>
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<td>L1</td>
<td>155.3–165.3</td>
<td>518</td>
<td>0.573</td>
<td>10.04</td>
<td>14.33</td>
<td>1427</td>
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<td>6.795</td>
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<td>0.8595</td>
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<tr>
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<tr>
<td>L9</td>
<td>23.06–46.12</td>
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<tr>
<td>L10</td>
<td>0–23.06</td>
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<td>0.9375</td>
<td>3.75</td>
<td>23.44</td>
<td>6251</td>
</tr>
</tbody>
</table>

For each layer, its area \( A \) and the second moment of area \( I \) can be calculated using the following equations:

\[
A = \frac{\pi}{4} (D_o^2 - D_i^2) \quad (8.11a)
\]

\[
I = \frac{\pi}{64} (D_o^4 - D_i^4) \quad (8.11b)
\]

where \( D_o \) and \( D_i \) are the outer and inner diameter of the layer respectively.

Using equations (8.6) and (8.7) and results in Table 8.1, the plastic resistance and effective flexural stiffness of the composite column are:

\[
N_{f_i,pl,Rd} = 1605 \text{ kN}
\]

\[
N_{f_i,pl,R} = 1741 \text{ kN}
\]

\[
(\text{EI})_{f_i,\text{eff}} = 3682 \text{ kN.m}^2
\]

**Step 3: Column compression resistance**

According to Section 8.2, the column effective length at the fire limit state is:

\[
L_{c,f_i} = 0.5 \times 4.5 = 2.25 \text{ m}
\]

The Euler buckling load is:

\[
N_{f_i,cr} = \frac{\pi^2 \times 3682}{2.25^2} = 7178 \text{ kN}
\]

giving the column slenderness as:

\[
\lambda_0 = \sqrt{\frac{1741}{7178}} = 0.492
\]

Using buckling curve ‘c’ of EN 1993-1-1, the column strength reduction factor is \( \chi = 0.8473 \), giving the compression resistance of the column as:

\[
N_{f_i,Rd} = 0.8473 \times 1605 = 1360 \text{ kN}
\]

For comparison, according to EN 1994-1-1, the compression resistance of the column at ambient temperature may be calculated to be 4336.7 kN. Therefore, at the standard fire resistance rating of R60, the unprotected and unreinforced composite column has a residual strength of 31.4% of its resistance at ambient temperature.
8.4.2. Alternative design method for composite column with partially encased steel section

The alternative design method follows the general calculation method, presented in the previous section. It should be pointed out that the alternative design method is only applicable to bending about the weak axis of the cross-section. Also the limits of application of the alternative design method (clause G.6(5)) are different from those of the general calculation method (clause 4.3.5.2(2)). This alternative method is applicable only to the standard fire exposure.

The alternative design method differs from the general calculation method in calculations of the properties of the composite cross-section. In the general calculation method, the composite cross-section is divided into numerous small blocks. In the alternative method, the composite cross-section is divided into only four large components: the flanges of the steel profile, the web of the steel profile, the concrete contained by the steel profile and the reinforcing bars.

The flanges of the steel profile are assumed to have the same temperature and Annex G.2 gives information on how to calculate this temperature value for different standard fire resistance ratings.

The web of the steel profile is assumed to have a reduced height with the same modulus of elasticity as at ambient temperature, but a reduced strength.

For concrete, it is also assumed to be at uniform temperature but with reduced dimensions.

For the reinforcing bars, the reduction factors for their effective yield strength and modulus of elasticity are directly given depending on the design standard fire resistance rating and the concrete cover.

The following paragraphs will present design aids to assist in implementing the alternative design method in Annex G.

8.4.2.1. Design aids for the web of the steel profile (Annex G.3)

Figure 8.3 shows the various symbols used in the calculations. The part of the web with height \( h_{w,fi} \), measured from the inner edge of the flange, should be neglected. The calculations in Annex G.3 involve using the factor \( \sqrt{1 - 0.16(H_t/h)} \), where \( H_t \) is an empirical number depending on the design standard fire resistance rating and \( h \) is the depth of the steel section as shown in Fig. 8.3. Denoting this factor by \( \beta \), Fig. 8.4 plots the \( \beta \)-value as a function of the steel section depth for different standard fire resistance ratings.

Using this value, the effective depth of the web is:

\[
d_{w,fi} = h - 2e_t - 2h_{w,fi} = (h - 2e_t) \left( 1 - \frac{h_{w,fi}}{h - 2e_t} \right)
= (h - 2e_t) \sqrt{1 - (0.16H_t/h)} = \beta(h - 2e_t) \tag{8.12}
\]

The maximum stress level is obtained from:

\[
f_{ay,w,1} = f_{ay,w} \sqrt{1 - (0.16H_t/h)} = \beta f_{ay,w} \tag{8.13}
\]

Fig. 8.3. Reduced cross-section for partially encased steel sections
Rearranging the equations in Annex G.3, the following equations may be used to calculate the contributions of the web of the steel profile to the plastic resistance and effective flexural stiffness of the composite cross-section:

\[
N_{fi,pl,Rd,w} = e_w d_w f_{ay,pl,w}/\gamma_{M,fi,a} \quad (8.14)
\]

\[
(EI)_{fi,w,z} = E_{a,w} e_w^3 / 12 \quad (8.15)
\]

**Example 8.3: Alternative design method for composite column with the partially encased steel section**

Calculate the plastic resistance and flexural rigidity of a partially encased steel section with a 305 × 305 × 137UC profile for a standard fire resistance rating R60. The steel is grade S355 and the design concrete strength at ambient temperature is 40 N/mm². Assume siliceous aggregates. The material partial safety factors are 1.0. When calculating the effective flexural rigidity \( (EI)_{fi,eff,z} \), use values of \( \phi_{f,\beta} = 0.9 \), \( \phi_{w,\beta} = 1.0 \) and \( \phi_{s,\beta} = 0.8 \) for the flanges, the web and concrete respectively according to Table G.7 of EN 1994-1-2.

**Results of calculation**

The dimensions of the cross-section are: overall width \( b = 309.2 \) mm, overall depth \( h = 320.5 \) mm, flange width \( b = 309.2 \) mm, flange thickness \( e_f = 21.7 \) mm, web depth \( = 277.1 \) mm, web thickness \( e_w = 13.8 \) mm.

Section factor \( A_m/V = 2 \times (0.3092 + 0.3205)/(0.3092 \times 0.3205) = 12.71 \text{ m}^{-1} \)

**Flanges**

From clause G.2(1) and Table G.1, the steel flange temperature is:

\( \theta_{f,1} = 680 + 9.55 \times 12.71 = 801.4 \text{°C} \)

From Table 6.1, the maximum stress level is \( 0.1093 \times 355 = 38.80 \text{ N/mm}^2 \). The elastic modulus is \( 0.0897 \times 205000 = 18388.5 \text{ N/mm}^2 \).

The plastic resistance of the flanges is:

\( N_{fi,pl,Rd,f} = 2 \times 309.2 \times 21.7 \times 38.8/1000 = 520.7 \text{ kN} \)

The effective flexural rigidity of the flanges is:

\( (EI)_{fi,f,z} = 18388.5 \times 21.7 \times 309.2^2 / 6 \times 10^9 = 1966 \text{ kN.m}^2 \)
Web
Table G.2 gives $H_t = 770$ mm.

$$\beta = \sqrt{1 - (0.16H_t/h)} = 0.7846$$

From equation (8.12), the effective depth of the web is:

$$d_{w,fi} = 0.7846 \times 277.1 = 217.4\text{mm}$$

From equation (8.13), $f_{sy,w,t} = 0.7846 \times 355 = 278.53$ N/mm$^2$.
From equations (8.14) and (8.15):

$$N_{fi,pl,Rd,w} = 13.8 \times 217.4 \times 278.53/1000 = 835.6\text{kN}$$

$$(EI)_{fi,w,z} = 205\,000 \times 217.4 \times 13.8^3/12/10^9 = 9.8\text{kN.m}^2$$

Concrete
According to Table G.3, $b_{c,fi} = 15.0\text{mm}$. From Table G.4, $\theta_{c,\varepsilon} = 330.9^\circ$. From Table 6.4, $k_{c,\theta} = 0.819$, $\varepsilon_{cu,\theta} = 0.007927$.

$$E_{c,sec,\theta} = 40 \times 0.819/0.007927 = 4133.12\text{N/mm}^2$$

$$f_{c,\theta} = 0.819 \times 40 = 32.76\text{N/mm}^2$$

From Clause G.4(4):

$$N_{fi,pl,Rd,c} = 0.86 \times (320.5 - 2 \times 21.7 - 2 \times 15) \times (309.2 - 13.8 - 2 \times 15) \times 32.76/1000$$

$$= 1847.85\text{kN}$$

$$(EI)_{fi,c,z} = 4133.2 \times (320.5 - 2 \times 21.7 - 2 \times 15) \times [(309.2 - 2 \times 15)^3 - 13.8^3]/12/10^9$$

$$= 1852.1\text{kN.m}^2$$

The total plastic resistance is:

$$N_{fi,pl,Rd} = 520.7 + 835.6 + 1847.85 = 3204.2\text{kN}$$

The total effective flexural rigidity is:

$$(EI)_{fi,eff,z} = 0.9 \times 1966 + 1.0 \times 9.8 + 0.8 \times 1852.1 = 3260.88\text{kN.m}^2$$

### 8.4.2.2. Some design tables

Although the detailed design calculation equations in Annex G are clearly presented and should be easy to follow, as demonstrated by the worked Example 8.3, they still require a large amount of calculation effort. To help designers make use of this type of composite column, calculations have already been performed to obtain the contributions of the different components of a partially encased composite cross-section to its plastic resistance and effective flexural stiffness for the Corus-manufactured UC steel sections.

Tables 8.2 to 8.5 present these calculation results, giving the plastic resistance and effective flexural stiffness of the flanges of the steel profile, the web of the steel profile and concrete. The tables have been generated using the following material properties:

**Steel:** $f_y = 275\text{N/mm}^2$, $E = 205\,000\text{N/mm}^2$, $\gamma_{M,fi,\alpha} = 1.0$

**Concrete:** $f_c = 25\text{N/mm}^2$, $E_{c,sec} = 25/0.0025 = 10\,000\text{N/mm}^2$, $\gamma_{M,c,\varepsilon} = 1.0$

For completeness, Tables 8.2–8.5 give values for all Corus UC sections for fire resistance rating up to R120. However, it is clear that it would not be appropriate to use some of the smaller sections for high fire resistance rating.

These tables do not include reinforcing bars because they do not have fixed sizes or positions, and calculating their contributions is relatively simple and straightforward.

If the material properties or the partial safety factors are different from above, the values in Tables 8.2–8.5 should be modified by the ratios of the appropriate new design property to the assumed value.

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Table 8.2. Plastic resistance and effective flexural stiffness of different parts of partially encased steel section, standard fire resistance R30

<table>
<thead>
<tr>
<th>UC profile</th>
<th>Plastic resistance (kN)</th>
<th>Effective flexural stiffness (kN.m²)</th>
<th>Plastic resistance (kN)</th>
<th>Effective flexural stiffness (kN.m²)</th>
<th>Plastic resistance (kN)</th>
<th>Effective flexural stiffness (kN.m²)</th>
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</thead>
<tbody>
<tr>
<td>UC profile</td>
<td>Flanges</td>
<td>Web</td>
<td>Concrete</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>356 × 406 × 634</td>
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Table 8.3. Plastic resistance and effective flexural stiffness of different parts of partially encased steel section, standard fire resistance R60

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Table 8.5. Plastic resistance and effective flexural stiffness of different parts of partially encased steel section, standard fire resistance R120

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An example is given below to illustrate how to use these tables.

**Example 8.4: Use of design tables**

Using the information in Table 8.3, recalculate the plastic resistance and effective flexural stiffness of the section in worked Example 8.3.

**Results of calculation**

From Table 8.3, the contributions of the flanges of the steel profile, the web of the steel profile and concrete to the plastic resistance and effective flexural stiffness of the composite cross-section are:

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</tr>
<tr>
<td>Concrete</td>
<td>1154.9</td>
<td>1157.6</td>
</tr>
</tbody>
</table>

The plastic resistance of the composite cross-section is:

$$N_{\text{fl.pl,Rd}} = (403.4 + 647.4) \times 355/275 + 1154.9 \times 40/25 = 3204.3 \text{ kN}$$

The effective flexural rigidity of the composite cross-section is:

$$(EI)_{\text{eff},z} = 0.9 \times 1955.9 + 9.8 + 0.8 \times 40/25 \times 1157.6 = 3260.93 \text{ kN.m}^2$$

**Eccentricity of loading**

In buildings, columns may be subjected to some eccentricity of loading. Calculating the compressive resistance of composite columns under eccentricity is already a rather complicated and long process at ambient temperature. Adding non-uniform temperature distribution to these calculations makes it extremely difficult to derive a simple hand-calculation method. However, for composite columns with nominal eccentricity, since the effect of eccentricity is relatively small, Annex G of EN 1994-1-2 allows the following simple scaling equation to be used to calculate the reduced column compressive resistance under eccentricity of loading in fire:

$$N_{\text{fl,Rd,δ}} = N_{\text{fl,Rd}} \frac{N_{\text{Rd,δ}}}{N_{\text{Rd}}} \quad (8.16)$$

where $N_{\text{Rd}}$ and $N_{\text{Rd,δ}}$ are the compressive resistance of the column at ambient temperature with and without eccentricity of loading $\delta$ respectively. $N_{\text{fl,Rd}}$ is the compressive resistance of the column without eccentricity of loading in fire.

**8.4.3. Alternative design method for composite columns with concrete-filled hollow sections**

The alternative design method in Annex H of EN 1994-1-2 is completely different from the general design method presented in Section 8.4.1.

When using the alternative design method in Annex H, a step-by-step approach should be taken, with the strain of the column increasing until the design compressive resistance to axial loading is obtained. At each step, the plastic resistance and Euler buckling load of the composite column are evaluated. The plastic resistance of the column is calculated differently from that in the general calculation method. It is calculated as the sum of the area of each component of the column cross-section times its stress. Here the stress is taken from the stress–strain relationship of the material at the column strain of the current step.
as the strain of the material increases, the plastic resistance of the composite column also increases. This is illustrated as curve 1 in Fig. 8.5. In the meantime, the Euler buckling load of the composite column is calculated using the tangent modulus of the stress–strain relationship of the material at the material temperature and strain of the current step. As the tangent modulus of the material decreases at increasing strain, the Euler buckling load of the composite column also decreases with increasing strain. This is shown as curve 2 in Fig. 8.5. When these two curves meet, the plastic resistance of the column becomes equal to the Euler load of the column and this value is the design compressive strength of the composite column.

Annex H has not specified any limit of application of the alternative design method. To be on the safe side, the limits of application of the general design method in clause 4.3.5.3(2) of EN 1994-1-2 should be observed.

The alternative design method in Annex H of EN 1994-1-2 will be extremely difficult to perform by hand. Not only does it involve dealing with non-uniform temperature distributions, but it also requires using detailed stress–strain relationships of the concrete and steel at elevated temperatures. Designers are encouraged to use the general design method.

8.4.3.1. Eccentricity of loading

Similar to the alternative design method for composite columns with partially encased steel sections, the eccentricity of loading is also treated in a simplistic way. For concrete-filled tubular columns, the design strength of the composite column with eccentricity of loading is obtained by multiplying the design strength of the axially loaded composite column by two factors, one depending on the percentage of reinforcement \(\phi_s\) and one on the column slenderness and eccentricity of loading \(\phi_e\). The values of these modification factors are presented in graphic form as Figs H1 and H2 of EN 1994-1-2 respectively.

8.5. Reinforced concrete columns

EN 1992-1-2 provides two simplified calculation methods for reinforced concrete columns, the ‘500°C isotherm method’ in Annex B.1 and the ‘zone method’ in Annex B.2. For reinforced concrete columns exposed to the standard fire condition, either method may be used. For reinforced concrete columns under parametric fire curves of EN 1991-1-2
only the 500°C isotherm method can be used. If the parametric fire curves are adopted, the opening factor of the fire compartment (Section 3.5) should not be less than 0.14 m²/².

8.5.1. 500°C isotherm method
This method assumes that concrete heated above 500°C has no strength or stiffness while concrete below 500°C retains the full strength and stiffness at ambient temperature. The concrete column is then designed as at ambient temperature, but using the reduced cross-section size. All reinforcing bars should be included in the calculations even if some of them may fall outside the reduced concrete cross-section. The strength of any reinforcing bar should be that at the appropriate elevated temperature.

8.5.2. Zone method
The zone method is only suitable for rectangular cross-sections. In this method, the original concrete cross-section is reduced to allow for fire damage. The damaged zone is a function of the temperature distribution of the entire concrete cross-section. Only the remaining concrete cross-section (the original cross-section minus the damaged zone) is then used in column strength calculations. Unlike the 500°C isotherm method, the reduced concrete cross-section has reduced strength and modulus of elasticity.

It is best using an example to illustrate the application of this method.

Example 8.5: Application of the zone method
Figure 8.6 shows a rectangular concrete cross-section which is heated from all four surfaces. In the zone method, it is assumed that the damaged zone has the same thickness on both the short and long sides of the cross-section, being that determined for the short side of the cross-section. Therefore, for the cross-section shown in Fig. 8.6(a), the width (short side = 2w₁) of the concrete cross-section is reduced by 2a₁ and the length (long side) of the concrete cross-section is also reduced by 2a₁.

![Fig. 8.6. Determination of reduced concrete cross-section for reinforced concrete columns: (a) column cross-section; and (b) equivalent wall](image)

To determine the damaged zone of the short side, the effect of the long side is ignored, so that the column cross-section becomes an equivalent wall (Fig. 8.6(b)). Assume that the wall thickness is 2w₁ and that the centre of the wall is represented by point M₁ anywhere along the centre line of the wall.

The half-thickness (w₁) of the wall is divided into n-parallel zones of equal thickness (n ≥ 3). Each zone is assumed to have the same temperature as that at the centre of the
Comparing the 500°C isotherm method and the zone method, it is clear that the zone method is much more laborious. Therefore, it is recommended that the 500°C isotherm method be used wherever possible. The zone method should only be used when the column would not have sufficient resistance when designed using the 500°C isotherm method and it is not possible to increase the cross-section size or the reinforcement.

Both the 500°C isotherm and the zone methods may be used to design columns with combined axial load and bending moments.

For a reinforced concrete column under fire condition, because of the reduction in its effective cross-section size and reduction in the modulus of elasticity at elevated temperatures, the second-order effect will become more pronounced than at ambient temperature. If the concrete column becomes slender according to the definition of EN 1992-1-1, Annex B.3 of EN 1992-1-2 should be followed to deal with the increased second-order effect. This design method follows the ambient temperature method in EN 1992-1-1 but is rather complex and numerical implementation of this method would be necessary. Alternatively, the tabulated results in Annex C of EN 1992-1-2 may be used, which have been generated by numerically implementing the calculation procedure in Annex B.3.

Advanced calculation methods should be used for reinforced concrete columns of non-rectangular cross-section.

\[ k_{cm} = \frac{1 - 0.2/n}{n} \sum_{i=1}^{n} k_c(\theta_i) \]  

(8.17)

where the subscript \( i \) indicates the zone number, \( \theta_i \) its temperature and \( k_c(\theta_i) \) the strength reduction factor of the zone. The factor \( (1 - 0.2/n) \) is incorporated to allow for variation in temperature within each zone.

The thickness of the fire-damaged zone is given by:

\[ a_{z1} = w_1 \left[ 1 - \left( \frac{k_{cm}}{k_c(\theta_m)} \right)^{\alpha} \right] \]  

(8.18)

where \( \theta_m \) is the temperature and \( k_c(\theta_m) \) the strength reduction factor at point \( M_1 \) along the centre line of the wall. \( \alpha = 1.3 \) for compression members (columns and walls) and \( \alpha = 1 \) for bending members (beams and slabs).

If the column is not heated on four sides, the damaged zone should be taken out only from the sides of the cross-section that are exposed to fire. If the column is heated from one side only, it forms half of the equivalent wall in Fig. 8.6(b). By deduction, if the rectangular cross-section of a column is heated from three sides with the unheated side being the shorter dimension of the cross-section, the half-thickness of the equivalent wall in Fig. 8.6(b) should be the lesser of the shorter dimension or half of the longer dimension.

Having determined the reduced concrete cross-section, the designcompressive resistance of the reinforced concrete column is calculated using the ambient temperature design method in EN 1992-1-1, but the design strength and modulus of elasticity of concrete should be reduced according to the temperature at point \( M_1 \) along the centre line of the cross-section. As in the 500°C isotherm method, the reinforcing bars should be calculated individually, including any reinforcing bar that may fall outside the reduced concrete cross-section.
CHAPTER 9

Design of bending members

9.1. Introduction
This chapter will provide guidance on the design of bending members (beams), including steel beams, composite beams and reinforced concrete beams. For composite beams, EN 1994-1-2 gives a number of approximate calculation methods for different types of composite beam. They will be dealt with separately.

It is assumed that temperatures in the cross-section of the beam are given as input data. Alternatively, readers should refer to the guidance in Chapter 4.

9.2. Steel beams
Design calculations for steel beams at the fire limit state are similar to those in EN 1993-1-1 at ambient temperature. In EN 1993-1-2, a steel beam should be checked for:

- sufficient bending moment capacity
- sufficient shear resistance
- sufficient lateral torsional buckling resistance
- deformation limit.

9.2.1. Bending moment capacity
Steel beams are normally exposed to fire attack from three sides with the top of the beams being insulated by the floors, therefore temperature distribution in the cross-section of a beam tends to be non-uniform. The non-uniform temperature distribution means that the steel strength and stiffness will be different at different locations of the cross-section, which should be accounted for in design calculations.

In EN 1993-1-2, the bending moment capacity of a class 1 or 2 cross-section is equal to the plastic bending moment capacity of the cross-section, which may be calculated using two methods. In the first method which is called the plastic bending moment capacity method (clause 4.2.3.3(2)), the steel cross-section is divided into a number of blocks. Each block is assumed to reach its yield strength. After finding the plastic neutral axis of the cross-section, which divides the cross-section into a compression part and a tension part with the same axial capacity, the plastic bending moment capacity of the cross-section is obtained by summing up contributions of all the blocks. This method requires information about temperatures of the entire cross-section. In the alternative method (clause 4.2.3.3(3)), the plastic bending moment capacity of the cross-section with non-uniform temperature distribution is related to that with uniform temperature distribution as follows:

\[
M_{f1,Rd} = \frac{M_{f0,Rd}}{\kappa_1}
\]  

(9.1)
where $M_{f1,Rd}$ is the plastic bending moment capacity of the cross-section with non-uniform temperature distribution, $\kappa_1 (= 0.7)$ is a modification factor and $M_{f0,Rd}$ is the plastic bending moment capacity of the cross-section at a uniform temperature $\theta$. Leaving out the material partial safety factors, $M_{f0,Rd}$ is obtained by multiplying the plastic bending moment capacity of the cross-section at ambient temperature by the yield strength reduction factor of steel ($k_y$ in Table 6.1). In EN 1993-1-2, the uniform temperature $\theta$ is referred to as ‘the uniform temperature $\theta$, at time $t$ in a cross-section which is not thermally influenced by the supports’. This is a rather confusing definition. Since the lower flange of the cross-section is the most critical element, the uniform temperature $\theta$ should be taken as that of the lower flange away from the supports.

For class 3 cross-sections, equation (9.1) should be used. However, $M_{f1,Rd}$ should be taken as the elastic bending moment capacity of the cross-section at a uniform temperature $\theta$.

An example is provided below to illustrate the two calculation methods.

**Example 9.1: Calculation of bending moment capacity**

Figure 9.1 shows the temperature distribution in the cross-section of a steel beam made of $457 \times 152 \times 67$UB. It is required to evaluate the plastic bending moment of the cross-section. The steel grade is S275 and the material safety factor is 1.0.

Fig. 9.1. Assumed temperature distribution for Example 9.1

**Calculation results**

**Method 1: plastic bending moment capacity method**

The cross-section may be checked to be class 1. The cross-section is divided into five layers consisting of: the top flange (153.8 mm by 15.0 mm), the top-most $\frac{1}{4}$ of the web, $\frac{1}{4}$ of the web just above the centre line (upper $\frac{1}{4}$ web), $\frac{1}{2}$ of the web below the centre line (lower $\frac{1}{2}$ web) and the lower flange. The root radius area of the entire cross-section is assumed to be equally distributed along the web so that a web thickness of 9.22 mm is used in the calculations. Results are summarized in Table 9.1 where it is assumed that the lower part of the cross-section is in tension and the upper part in compression. Since the top flange has more compressive capacity than the combined tensile capacity of the other parts, the plastic neutral axis is in the top flange which is further divided into two layers, one in compression ($14.876$ mm thick) and one in tension ($0.124$ mm thick) so that the tensile and the compressive capacities of the cross-section are equal.

**Table 9.1. Results of calculation for Example 9.1**

<table>
<thead>
<tr>
<th>Layer</th>
<th>Layer temperature ($^\circ$C)</th>
<th>Design strength (N/mm$^2$)</th>
<th>Capacity (kN)</th>
<th>Lever arm to PNA (mm)</th>
<th>Moment resistance (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top flange</td>
<td>550</td>
<td>$0.625 \times 275 = 171.9$</td>
<td>393.3 (C)</td>
<td>7.438 (C)</td>
<td>2.925</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.28 (T)</td>
<td>0.062 (T)</td>
<td>0.002</td>
</tr>
<tr>
<td>Top $\frac{1}{4}$ web</td>
<td>600</td>
<td>$0.47 \times 275 = 129.25$</td>
<td>127.5 (T)</td>
<td>53.624 (T)</td>
<td>6.837</td>
</tr>
<tr>
<td>Upper $\frac{1}{4}$ web</td>
<td>700</td>
<td>$0.23 \times 275 = 63.25$</td>
<td>62.40 (T)</td>
<td>160.624 (T)</td>
<td>10.023</td>
</tr>
<tr>
<td>Lower $\frac{1}{4}$ web</td>
<td>750</td>
<td>$0.17 \times 275 = 46.75$</td>
<td>92.24 (T)</td>
<td>321.124 (T)</td>
<td>29.620</td>
</tr>
<tr>
<td>Lower flange</td>
<td>750</td>
<td>46.75</td>
<td>107.85 (T)</td>
<td>435.624 (T)</td>
<td>46.982</td>
</tr>
<tr>
<td>Total</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>C = T = 393.3</td>
<td>96.389</td>
</tr>
</tbody>
</table>

PNA = plastic neutral axis
The plastic bending moment capacity of the cross-section is \( M_{pl,RD} = 96.389 \text{kN.m} \)

**Method 2: Equation (9.1)**

The plastic modulus of the cross-section is 1453 cm\(^3\). The plastic bending moment capacity of the cross-section is:

\[
M_{pl,RD} = (0.17 \times 0.275 \times 1453)/0.7 = 97.04 \text{kN.m}
\]

For this cross-section and the specific temperature distribution, the two methods give almost identical answers. In general, the alternative method (equation (9.1)) has been found to be reasonably accurate. The usefulness of the more complex plastic bending moment capacity method is when dealing with cross-sections with very steep non-uniform temperature distributions – for example, shelf angle beams or slim floor beams.

### Example 9.2: Critical temperature method

Calculate the critical temperature of the cross-section used in Example 9.1. Assume that the applied maximum bending moment in the beam at the fire limit state is equal to the plastic bending moment capacity calculated using method 2 in Example 9.1 above.

**Calculation results**

The degree of utilization is:

\[
\mu_0 = 0.7 \times 97.04/(0.275 \times 1453) = 0.17
\]

Equation (9.2) gives \( \theta_{a,cr} = 749.4^\circ\text{C} \). This critical temperature is identical to the maximum temperature of 750°C used in Example 9.1. This is not surprising as the critical temperature–degree of utilization relationship (equation (9.2)) is the result of curve-fitting the steel effective yield strength–temperature relationship given in Table 6.1.

### 9.2.2. Shear resistance

Calculating the shear resistance of a steel cross-section in fire follows the same procedure as at ambient temperature, i.e. the shear resistance of the cross-section is that of the web. In
clause 4.2.3.3(6), the reference temperature of the web is defined as the average temperature of the web. This is an approximation because, strictly speaking, the averaging process should be applied on the steel design strength of the web, instead of the temperature of the web. However, averaging the steel design strength of the web would be more laborious. The following worked example can be used to show the difference of the two averaging processes.

### Example 9.3: Calculation of shear resistance
Calculate the shear resistance of the cross-section in Example 9.1.

**Calculation results**

1. Using average temperature of the web
   - The average temperature of the web is:
     \[
     \frac{1}{2} \times [(550 + 750)/2 + 750] = 700\, ^\circ\text{C}
     \]
   - giving the reduction factor for the yield strength of steel \(k_{y,web} = 0.23\) according to Table 6.1.

2. Using average steel design strength of the web
   - Assume the web is divided into three parts as in Example 9.1. The average reduction factor for the yield strength of steel of the web is:
     \[
     k_{y,web} = \frac{1}{2} \times [(0.47 + 0.23)/2 + 0.17] = 0.26
     \]
   - Calculating the average web temperature can be troublesome because it will be necessary to know the temperature distribution in the entire web. Since shear resistance usually does not govern design, it is safe to assume that the web temperature is the same as that of the heated flange.

### 9.2.3. Lateral torsional buckling
Lateral torsional buckling should rarely be a problem for steel beams at the fire limit state. The calculation methods in EN 1993-1-2 are briefly described for completeness.

#### 9.2.3.1. Steel beam with uniform temperature distribution in the cross-section
For a beam with uniform temperature distribution in the cross-section, calculating the lateral torsional buckling resistance of the beam at fire limit state follows the same procedure as at ambient temperature in EN 1993-1-1. But there are two modifications: (1) the slenderness of the beam should be modified to take account of the difference in reduction factors in the yield strength and modulus of elasticity of steel at elevated temperatures; (2) the strength reduction factor for lateral torsional buckling at fire limit state follows equation (8.3) instead of any of the buckling curves in EN 1993-1-1. Both modifications are identical to steel columns which have already been described in Section 8.3.

#### 9.2.3.2. Steel beam with non-uniform temperature distribution in the cross-section
When temperature distribution in the cross-section of a steel beam is non-uniform, the calculation method for checking lateral torsional buckling resistance is the same as for a beam with uniform temperature distribution, but the maximum temperature of the compression flange \(\theta_{a,\text{com}}\) should be used as the reference uniform temperature of the beam. This will be conservative when the compression flange of the cross-section has the highest temperature. But this method may not be safe if the compression flange of the cross-section has the lowest temperature.

### 9.2.4. Control of deformation
The fundamental objective of fire safety design is to prevent fire spread. For a load-bearing member, the main design criterion is to avoid structural collapse, therefore the design strength of the member is the main consideration of the Eurocodes. However, for some
beams, deformation may also have to be controlled. For example, deflection control becomes important for the following two cases:

(1) Large beam deflections cause the fire protection material to undergo large strains, which may become brittle and detach from the steel beam.
(2) Large beam deformation may lead to openings in the separating members underneath the beam, leading to fire integrity failure of the fire-resistant compartment.

It is likely that the beam deformation limits for the above two cases will be different. If design is concerned with performance of the fire protection material, the deformation limit should correspond to the maximum steel strain that would make the fire protection material ineffective. In general, commonly used fire protection materials can accommodate large steel strains so that the full steel strength can be sustained. However, if a fire protection material cannot accommodate large strains, the steel design strength should be reduced to that corresponding to the appropriate maximum strain level of the fire protection material.

For the second case, how much a steel beam can deform without breaching fire integrity of the fire resistant compartment has never been properly considered. But in light of recent advances in structural fire engineering in which structural deformations can approach very large values without causing a structural collapse, fire safety design based on structural strength alone may not be adequate and it may be necessary to explicitly check structural deformations. To do so, it will be necessary to employ advanced calculation methods because the simplified calculation methods in the various Eurocodes cannot perform this task.

9.3. Steel beam exposed to fire on three sides with concrete slab on the fourth side

A steel beam exposed to fire on three sides with concrete slab on the fourth side may be designed either as a steel beam or as a composite beam. Section 9.2 has described the design for steel beams. This section will provide additional guidance if the beam is designed as a composite beam.

For this type of composite beam, there is no need to check for lateral torsional buckling (clause 4.3.4.1.1) because under sagging bending moment, the compression flange of the beam is restrained by the concrete slab. Under hogging bending moment in a continuous beam near a support, the unrestrained length of the beam will likely be short and the bending moment gradient is steep.

The vertical shear capacity of a composite beam (clause 4.3.4.1.3) is taken as the shear resistance of the steel section, which has already been described in Section 9.2.2 above.

A composite beam should have adequate longitudinal shear capacity at the interface between the concrete slab and the steel profile. The calculation method (clause 4.3.4.1.5 in EN 1994-1-2) is the same as in EN 1994-1-1 for a composite beam at ambient temperature, but the compressive capacity of the concrete slab and the tensile capacity of the steel profile should account for non-uniform temperature distributions in the concrete slab and the steel profile.

The sagging and hogging bending moment capacity of a composite cross-section may be calculated by using the plastic theory for any class of cross-section except for class 4 cross-section under hogging bending moment (clause 4.3.4.1.2). The calculation procedure is the same as at ambient temperature in EN 1994-1-1, but should use the relevant material properties at elevated temperatures, considering non-uniform temperatures in the concrete slab, the steel profile and reinforcement.

In order to facilitate the above calculations for longitudinal shear capacity and bending moment capacity, it is necessary to have data on shear resistance of the stud shear connectors in fire. EN 1994-1-2 provides simple recommendations to relate temperatures of the stud connectors and concrete to that of the upper flange of the steel profile. These temperatures are then used to obtain the reduced material strengths of the stud shear connectors and
concrete, which are then substituted into relevant equations in EN 1994-1-1 to give the reduced shear resistance of the stud shear connectors.

Alternatively, the critical temperature method may be used to estimate the critical temperature of the lower flange of the composite beam under a given sagging bending moment. This method is very simple and can be used to quickly estimate whether the composite beam is adequate in fire without going through the detailed calculations described in this section. However, for a composite beam designed for partial shear connection at ambient temperature, using the critical temperature method is likely to underestimate the resistance of the composite beam. This comes about because the critical temperature method does not take into account the better performance of the stud shear connectors than the steel profile. If this is the case, and should clause 4.3.4.2.3 of EN 1994-1-2 indicate that the composite beam with partial shear connection is slightly inadequate, the current version of BS 5950 Part 8 may be used to give a higher critical temperature. However, to ensure consistency in the use of design codes, the designer should still perform detailed calculations using EN 1994-1-2 to prove that the composite beam does have sufficient resistance in fire.

9.4. Composite beams comprising steel beams with partial concrete encasement

Two methods may be used to calculate the bending resistance of a composite beam comprising a steel section with partial concrete encasement. In the first method, the temperature distribution in the composite cross-section should be known, either from fire tests or from numerical modelling. The plastic analysis method, as described in Section 9.2.1 and illustrated in Example 9.1, may be used, taking into consideration reduced strengths of the steel profile, the concrete and the reinforcing bars at the respectively elevated temperatures.

The alternative method, which is given in Annex F, is only applicable to standard fire exposure beneath the concrete slab. In the alternative calculation method, there is no need to calculate the temperature distribution in the composite cross-section. Instead, the composite cross-section is divided into a few blocks, each having the same mechanical properties. For calculating the sagging moment resistance (refer to Fig. 9.2), the composite cross-section is divided into the following parts:

- concrete section: with a reduced thickness and the same design strength as at 20°C
- upper flange of the steel profile: with a reduced effective width and the same design strength as at 20°C

![Fig. 9.2. Elements of a partially encased composite cross-section for calculation of the sagging moment resistance: (a) example of stress distribution in concrete; (b) example of stress distribution in steel](image)
For calculating the hogging moment resistance (refer to Fig. 9.3), the composite cross-section is divided into the following parts:

- top part of the web: with the same design strength of steel as at 20°C
- bottom part of the web: with a reduced design strength of steel
- lower flange of the steel profile: with a reduced design strength of steel
- reinforcing bars in the concrete between the flanges of the steel profile: with a reduced design strength.

For calculating the hogging moment resistance (refer to Fig. 9.3), the composite cross-section is divided into the following parts:

- reinforcing bars in the slab: with reduced design strengths depending on fire resistance rating and positions of the reinforcing bars
- upper flange of the steel profile: as for calculating the sagging moment resistance
- concrete between the flanges of the steel profile: with a reduced cross-section but the same design strength of concrete as at ambient temperature
- reinforcing bars in the concrete between the flanges of the steel profile: as for calculating the sagging moment resistance.

The concrete slab, the web and lower flange of the steel profile should be ignored. To check the vertical shear resistance of the steel web, the steel design strength distribution as for calculating the sagging moment resistance should be used. The shear connectors may be assumed to retain their full strength of ambient temperature provided they are fixed directly to the effective width of the upper flange of the steel profile.

A worked example is provided below (Example 9.4) to illustrate the calculation method in Annex F.

**Example 9.4: Use of Annex F**

Figure 9.4 shows the dimensions of a partially encased composite beam cross-section. The required standard fire resistance rating is R60. The steel grade is S275 and the cylinder strength of concrete is 25 N/mm². The design strength of the reinforcing bars is 460 N/mm². The partial safety factor for materials is 1.0. Calculate the sagging bending moment resistance and shear resistance of the cross-section.

**Calculation results**

(1) Concrete slab

From Table F.1, the reduction in slab thickness is \( h_{c,fi} = 20 \) mm. The effective thickness of the slab is \( 200 - 20 = 180 \) mm.
Fig. 9.4. Dimensions used in Example 9.4 (units in mm)

(2) Upper flange of the steel profile
From Table F.2, the reduction in flange width is:
\[ b_{\text{fl}} = 14.5/2 + 10 = 17.25 \text{ mm} \]
The effective width of the upper flange is:
\[ 190.4 - (2 \times 17.25) = 155.9 \text{ mm} \]
The tensile capacity of the upper flange is:
\[ R_{a,\text{uf}} = 155.9 \times 14.5 \times 275/1000 = 621.6 \text{ kN} \]

(3) Top part of the web
\[ h = 457/190.4 = 2.4 \]
Table F.3 gives \( a_1 = 9500 \text{ mm}^2 \) and \( a_2 = 0 \). Therefore
\[ h_1 = 9500/190.4 = 49.9 \text{ mm} \]
\[ h_2 = 457 - (2 \times 14.5) - 49.9 = 378.1 \text{ mm} \]
The tensile capacity of the top part of the web is:
\[ R_{a,\text{uw}} = 378.1 \times 9.0 \times 275/1000 = 935.8 \text{ mm} \]

(4) Bottom part of the web
\[ a_0 = 0.018 \times 14.5 + 0.7 = 3.31 \]
Table F.4 gives:
\[ k_a = [0.21 - 26/190.4 + 457/(24 \times 190.4)] \times 3.31 = 0.57 > k_{a,\text{max}} = 0.4 \]
Therefore \( k_a = 0.4 \).
The tensile capacity of the bottom part of the web is:
\[ R_{a,\text{lw}} = 49.9 \times 9.0 \times (1 + 0.4)/2 \times 275/1000 = 86.5 \text{ kN} \]
The centre of this tensile capacity is 21.4 mm from the bottom of the web.

(5) Lower flange of the steel profile
The tensile capacity of the lower flange is:
\[ R_{a,\text{lf}} = 190.4 \times 14.5 \times 0.4 \times 275/1000 = 303.7 \text{ kN} \]

(6) Additional reinforcing bars
\[ A_m/V = (2 \times 457 + 190.4)/(457 \times 190.4) = 0.01269 \text{ mm}^{-1} \]
From equation (F.2)
\[ u = 1/[1/80 + 1/40 + 1/(190.4 - 9.0 - 40)] = 22.4 \text{ mm} \]
9.5. Reinforced concrete beam

EN 1992-1-2 provides two calculation methods for reinforced concrete beams. The first calculation method can be used on its own while the second calculation method should be used in conjunction with the tabulated data. The first calculation method is based on the reduced concrete cross-section that is obtained either from the 500°C isotherm method or the zone method, which have already been described in Section 8.5. Once the reduced concrete cross-section is determined, the bending moment capacity of the cross-section may be calculated in the same way as in EN 1992-1-1 for reinforced concrete beams at ambient temperature.

When checking the shear and torsion capacity of a reinforced concrete beam at fire limit state, the reference link temperature should be taken as that at the intersection of the links with the limit line of the effective tension area of the reduced cross-section that is calculated according to EN 1992-1-1.

The second calculation method is given in Annex E of EN 1992-1-2. This method is used to extend scope of application of the tabulated data in Section 5 of EN 1992-1-2. The tabulated data give the minimum dimensions of reinforced concrete beams and the minimum concrete cover to reinforcement. When using the method in Annex E of EN 1992-1-2, the minimum dimensions of the reinforced concrete beams cannot be changed. Therefore, Annex E should only be used when it is necessary to justify reducing the concrete cover to reinforcement (axis distance $a$) in the tabulated data. Furthermore, this calculation method should only be applied where the applied load is predominantly uniformly distributed.

From Table F.5:

\[
k_r = \frac{22.4 \times 0.034 - 0.04}{0.101/\sqrt{0.01269}} = 0.647
\]

The total area of 2T24 bars is 904.8 mm$^2$, therefore the tensile capacity of the reinforcing bars is:

\[
R_s = 904.8 \times 0.647 \times 460/1000 = 269.3 \text{kN}
\]

**Sagging bending moment resistance**

The total tensile capacity of the steel profile and reinforcing bars is:

\[
R_s = 621.6 + 935.8 + 86.5 + 303.7 + 269.3 = 2261.9 \text{kN}
\]

The compression capacity of the effective concrete slab depth is:

\[
R_c = 2000 \times 180 \times 0.85 \times 25/1000 = 7650 \text{kN} > 2261.9 \text{kN}
\]

The depth of concrete in compression is:

\[
d_c = 2261.9/7650 \times 180 = 53.22 \text{mm}
\]

Taking moments about the top of the concrete slab, the sagging bending moment resistance of the partially encased composite cross-section is:

\[
M_{f1,Rd} = -2261.9 \times 53.22/2/1000 + 621.6 \times (200 + 14.5/2)/1000 + 935.8 \times (200 + 14.5 + 378.1/2)/1000 + 86.5 \times (200 + 457 - 14.5 - 21.4)/1000 + 303.7 \times (200 + 457 - 14.5)/1000 + 269.3 \times (200 + 457 - 14.5 - 80)/1000
\]

\[
= 848.7 \text{kN.m}
\]

**Shear resistance**

The shear resistance of the steel web is:

\[
\frac{935.8 + 86.5}{\sqrt{3}} = 590.2 \text{kN}
\]
9.6. Comments on EN 1992-1-2 tabulated data

Tabulated data for reinforced concrete beams are given in Tables 5.5–5.7 of EN 1992-1-2. Table 5.5 is for simply supported beams made with reinforced and prestressed concrete. Table 5.6 for continuous beams made with reinforced and prestressed concrete. For reinforced and prestressed concrete continuous I-beams with high shear (clause 5.6.3(6)), the minimum beam width and web thickness are given in Table 5.7.

The tabulated data are based on critical steel temperatures of 500°C for conventional reinforcing bars (clause 5.2.(4)), 400°C for prestressing tendons and 350°C for strands and wires (clause 5.2.(5)), corresponding to a reduction factor for the design load level for the fire situation (clause 2.4.2) of $\eta_f = 0.7$. If the design load level is different, EN 1992-1-2 allows the axis distance of the reinforcing bars to be changed (clause 5.2(7) and clause 5.2.(8)). Furthermore, if the critical steel temperature is lower than 400°C, the minimum cross-sectional dimensions should be increased (clause 5.2(10)).
CHAPTER 10

Design of slabs

10.1. Introduction
This chapter provides guidance on the design of fire-resistant composite slabs according to EN 1994-1-2 and reinforced concrete slabs according to EN 1992-1-2.

A fire-resistant slab may be divided into two generic types: non-load-bearing and load-bearing. A non-load-bearing fire-resistant slab should meet the fire-resistant requirement of sufficient thermal insulation and integrity. A load-bearing fire-resistant slab should also meet the fire-resistant requirement of sufficient load-carrying capacity, in addition to thermal insulation and integrity. At present, it is difficult to evaluate fire integrity by calculation. It is assumed that if the fire safety design follows the tabulated data or the simplified calculation methods presented in EN 1994-1-2 or EN 1992-1-2, the design would meet the fire-resistant requirement of integrity and no further check is necessary. If advanced calculation methods are used, the designer should ensure that the integrity of the fire-resistant compartment is maintained by appropriate detailing, so that the large structural deformations may be accommodated by the fire-resistant compartment. As with dealing with other types of structural member, this chapter will only provide guidance on using simplified calculation methods.

10.2. Composite slabs
To check for both fire-resistant requirements of thermal insulation and load-bearing capacity of a composite slab, it is essential that temperatures in the slab are available. The main focus of the design clauses in EN 1994-1-2 is on providing equations to calculate temperatures in different parts of a composite slab. These equations (Annex D of EN 1994-1-2) were derived by Both36 who curve-fitted the results of extensive numerical simulations using the finite-element software DIANA. Detailed equations are given in section 4.5 of this guide.

The calculation method in Annex D is applicable to slabs with both re-entrant and trapezoidal steel decking profiles. The range of applicability in clause 4.3.2(8) of EN 1994-1-2 is based on an inventory of commonly used steel decking types. Concrete can be either normal weight or lightweight. Although not mentioned in EN 1994-1-2, the average moisture content was 4% for normal-weight concrete and 5% for lightweight concrete, by dry weight. Since higher moisture content will reduce the concrete temperatures, Annex D may be safely applied to concrete with higher moisture contents. However, if the concrete moisture content is significantly below the assumed levels, additional thermal analysis may be necessary. An important assumption (not clearly mentioned in Annex D of EN 1994-1-2) by Both36 is that the steel decking remains fully bonded to the concrete. Steel decking has been observed to debond in some fire tests. Therefore, if it is considered that the steel decking would debond in fire, its contribution to the bending moment resistance of the slab should not be included.
The following worked examples illustrate the step-by-step implementation of the design recommendations in Annex D of EN 1994-1-2.

**Example 10.1: Load carrying capacity of composite slab**

Figure 10.1 shows the dimensions of the unit width cross-section of a continuous composite slab using trapezoidal steel decking. It is required to calculate the load-carrying capacity of the composite slab under uniformly distributed load in fire.

**Fig. 10.1.** Dimensions of unit width of slab (in mm)

<table>
<thead>
<tr>
<th>Input data</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab span:</td>
<td>$L = 4.5,\text{m}$</td>
</tr>
<tr>
<td>Fire-resistance rating:</td>
<td>60 min</td>
</tr>
<tr>
<td>Steel decking thickness:</td>
<td>1 mm</td>
</tr>
<tr>
<td>Steel decking grade:</td>
<td>S355 cold-formed</td>
</tr>
<tr>
<td>Concrete grade:</td>
<td>C30, normal weight, $\gamma_{M,\text{f,e}} = 1.1$</td>
</tr>
<tr>
<td>Top reinforcement:</td>
<td>A193 mm$^2$/m mesh, axis distance = 20 mm</td>
</tr>
<tr>
<td>Additional reinforcement:</td>
<td>T12 in each rib, axis distance = 30 mm</td>
</tr>
<tr>
<td>Reinforcement grade:</td>
<td>460 N/mm$^2$</td>
</tr>
</tbody>
</table>

**Calculation results**

From the dimensions in Fig. 10.1, the geometric dimensions of Figs 4.4 and 4.5 can be calculated as:

\[
\begin{align*}
 l_1 &= 180\,\text{mm}, \quad l_2 = l_3 = 120\,\text{mm}, \quad h_1 = 70\,\text{mm}, \quad h_2 = 55\,\text{mm}, \quad \alpha = 61.4\,\text{degrees}
\end{align*}
\]

(Note: these dimensions fall outside the range of applicability of the calculation method in Annex D. However, the following calculations are still performed using this calculation method for the purpose of illustrating the application of the design method.)

\[
\begin{align*}
 A &= 55 \times \left( \frac{180 + 120}{2} \right) = 8250\,\text{mm}^2/\text{m}
\end{align*}
\]

\[
\begin{align*}
 L_r &= 120 + 2 \sqrt{55^2 + \left( \frac{180 - 120}{2} \right)^2} = 245.3\,\text{mm}^2/\text{m}
\end{align*}
\]

\[
\begin{align*}
 \frac{A}{L_r} &= 33.63\,\text{mm}, \quad \Phi = 0.809
\end{align*}
\]

\[
\begin{align*}
 u_1 = u_2 = \sqrt{30^2 + 60^2} = 67.08\,\text{mm}, \quad u_3 = 30\,\text{mm}, \quad \text{equation (4.1) giving } z = 2.343\,\text{mm}^{0.5}
\end{align*}
\]

(1) **Fire resistance according to thermal insulation**

Using equation (D.1) and Table D.1 (from Annex D) for normal-weight concrete...
-28.8 + 1.55 \times 70 - 12.6 \times 0.809 + 0.33 \times 33.63 - 735/120 + 48 \times 33.63/120 = 88

which is greater than 60 min and so OK)

(2) Sagging moment resistance $M_{B, Rd+}$

Steel decking lower flange:

Equation (D.4) and Table D.2 (of Annex D) give:

$\theta = 951 - 1197/120 - 2.32 \times 33.63 + 86.4 \times 0.809 - 150.7 \times 0.809^2 = 834.4^\circ C$

Table 6.1 gives strength reduction factor $k = 0.093$

Area $A = 120 \text{ mm}^2$

Steel decking web:

$\theta = 661 - 833/120 - 2.96 \times 33.63 + 537.7 \times 0.809 - 351.9 \times 0.809^2 = 759.2^\circ C$

Table 6.1 gives $k = 0.159$, $A = 125.3 \text{ mm}^2$

Steel decking upper flange:

$\theta = 340 - 3269/120 - 2.62 \times 33.63 + 1148.4 \times 0.809 - 679.8 \times 0.809^2 = 708.8^\circ C$

$k = 0.219$, $A = 120 \text{ mm}^2$

Reinforcing bar in rib:

Equation (D.5) and Table D.3 (of Annex D) give:

$\theta \alpha = 1191 - 250 \times 30/55 - 240 \times 2.343 - 5.01 \times 33.63 + 1.04 \times 61.4 - 925/120$

$= 380^\circ C$

Table 6.6 gives strength reduction factor $k = 0.952$

Area = 113.1 $\text{ mm}^2$

Depth of concrete in compression:

Design strength of concrete $= 30/1.1 = 27.27 \text{ N/mm}^2$

Tensile capacity of steel decking and reinforcement bar in rib:

$120 \times 355 \times 0.093 + 125.3 \times 355 \times 0.159 + 120 \times 0.219 \times 355 + 113.1 \times 0.952 \times 460$

$= 3961.8 + 7072.6 + 9329.4 + 49529 = 69.893 \text{ N}$

Equation (4.2) of EN 1994-1-2 gives the depth of concrete in compression as:

$d_c = 69893/(0.85 \times 300 \times 27.27) = 10.05 \text{ mm}$

Taking moment about the centre of compression, equation (4.3) of EN 1994-1-2 gives the sagging moment resistance as:

$M_{B, Rd+} = 3961.8 \times (70 - 10.05/2 + 55 - 0.5) + 7072.6 \times (70 - 10.05/2 + 55/2) + 9329.4 \times (70 - 10.05/2 + 0.5) + 49529 \times (70 - 10.05/2 + 55 - 30)$

$= 6195000 \text{ N.mm} = 6.195 \text{ kN.m}$

(3) Hogging moment resistance $M_{B, Rd-}$

Hogging reinforcement area $= 193 \text{ mm}^2 \times 0.3 \text{ m} = 57.9 \text{ mm}^2$

$N = 57.9 \times 460 = 26634 \text{ N}$

Equation (D.7) and Table D.3.3 (of Annex D) give:

$\theta_{lim} = 867 - 1.9 \times 10^{-4} - 8.75 \times 33.63 - 123 \times 0.809 - 1378/120 = 457^\circ C$
Equation (D.5) gives:

\[
457 = 1191 - 250 \times 0.75 - 240 \times z - 5.01 \times 33.63 + 1.04 \times 61.4 - 925/120
\]

giving \( z = 1.809 \text{ mm}^{0.5} \).

Equation (D.9) gives:

\[
Y_I = Y_{II} = \left( \frac{1}{1.809 - \frac{4}{\sqrt{180 + 120}}} \right)^z = 9.65 \text{ mm} < 55 \text{ mm}
\]

Equation (D.10) gives:

\[
X_{II} = \frac{1}{2} \times 120 + \frac{9.65}{\sin 61.4} (\cos 61.4 - 1) = 54.27 \text{ mm}
\]

giving a total effective concrete width of 108.54 mm at this position.

For simplicity, assume the effective width of the concrete in the rib is constant. Equation (4.2) of EN 1994-1-2 gives the concrete depth in compression as:

\[
d_c = \frac{26634}{(0.85 \times 108.54 \times 27.27)} = 10.59 \text{ mm}
\]

Equation (4.3) of EN 1994-1-2 gives the hogging moment resistance as:

\[
M_{f,Rd} = 26634 \times (125 - 20 - 9.65 - 10.59/2)/1000000 = 2.4 \text{ kN.m}
\]

(4) Load-carrying capacity of the composite slab in fire

According to plastic analysis and assuming that plastic hinges form at the supports and in the mid-span of the continuous slab, the slab resistance to uniformly distributed load is calculated from:

\[
M_{f,Rd^+} + M_{f,Rd^-} = \frac{1}{8} w L^2
\]

where \( L = 4.5 \text{ m} \), giving \( w = 3.395 \text{ kN/m} \).

The resistance of the slab in fire to uniformly distributed load is

\[
3.395/0.3 = 11.32 \text{ kN/m}^2
\]

Example 10.2: Use of Annex D


Assumed design parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span</td>
<td>3.5 m</td>
</tr>
<tr>
<td>Slab thickness</td>
<td>140 mm</td>
</tr>
<tr>
<td>Concrete type</td>
<td>lightweight</td>
</tr>
<tr>
<td>Cylinder strength</td>
<td>( f_{ck} = 30 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Deck yield strength</td>
<td>( f_y = 280 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Mesh</td>
<td>A252</td>
</tr>
<tr>
<td>Mesh yield strength</td>
<td>( f_y = 500 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td>Cover</td>
<td>15 mm</td>
</tr>
<tr>
<td>Fire resistance</td>
<td>120 min</td>
</tr>
</tbody>
</table>

Consider the deck profile as follows:

\[
l_1 = 112.5 \text{ mm}; l_2 = 137.5 \text{ mm}; l_3 = 40.0 \text{ mm} \\
h_1 = 89.0 \text{ mm}; h_2 = 51.0 \text{ mm}; h_3 = 0.0 \text{ mm}
\]
Assumed design actions

Dead loads:

- Selfweight of slab \( = 2.61 \, \text{kN/m}^2 \)
- Ceiling and services \( = 0.9 \, \text{kN/m}^2 \)
- Total permanent action, \( G_k = 3.51 \, \text{kN/m}^2 \)

Imposed loads:

- Occupancy load \( = 2.5 \, \text{kN/m}^2 \)
- Movable partitions \( = 0.8 \, \text{kN/m}^2 \)
- Total variable load, \( Q_k = 3.3 \, \text{kN/m}^2 \)

Design effect at ambient temperature:

\[
E_d = 1.35(3.51) + 1.5(3.30) = 9.69 \, \text{kN/m}^2
\]

Design action in fire:

Combination factor \( \psi_{1,1} = 0.5 \) for Category B: Office area (EN 1990, Tables NA.A1.1 and NA.A1.2 (A))

Partial factors for actions \( \gamma_G = 1.35; \gamma_{Q,1} = 1.5 \)

The reduction factor

\[
\eta_i = \frac{G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} = 0.55 \quad \text{(EN 1994-1-2, 2.4.2(3))}
\]

Design action in fire

\[
E_{d,1,1} = \eta_i E_d = 5.33 \, \text{kN/m}^2 \quad \text{(EN 1994-1-2, 2.4.2(2))}
\]

Check for application limitations (EN 1994-1-2, Table D.7):

- \( l_1 = 112.5 \, \text{mm} \) within the limits 77.0–135.0 mm \( \Rightarrow \) OK
- \( l_2 = 137.5 \, \text{mm} \) within the limits 110.0–150.0 mm \( \Rightarrow \) OK
- \( l_3 = 40.0 \, \text{mm} \) within the limits 38.5–97.5 mm \( \Rightarrow \) OK
- \( h_1 = 89.0 \, \text{mm} \) within the limits 50.0–130.0 mm \( \Rightarrow \) OK
- \( h_2 = 51.0 \, \text{mm} \) within the limits 30.0–60.0 mm \( \Rightarrow \) OK

Thermal insulation (EN 1994-1-2, D.1(2), equation (D.3)):

Configuration factor \( \Phi \) of the upper flanges is given by:

\[
\Phi = \frac{1}{l_3} \left[ \sqrt{h_2^2 + \left( \frac{l_3}{2} - l_2 \right)^2} - \sqrt{h_2^2 + \left( \frac{l_3}{2} - l_1 \right)^2} \right] = \frac{1}{40} \left[ \sqrt{51^2 + \left( 40 + \frac{112.5 - 137.5}{2} \right)^2} - \sqrt{51^2 + \left( \frac{112.5 - 137.5}{2} \right)^2} \right] = 0.136
\]
The rig geometry factor $A/L_r$ is given by (EN 1994-1-2, equation (D2)):

$$\frac{A}{L_r} = \frac{h_2(l_1 + l_2) / 2}{l_2} = \frac{51(112.5 + 137.5) / 2}{137.5} = 6.375 \times 137.5 = 46.36 \text{ mm}$$

(Note: $L_r$ is the exposed area of the rib per metre of rib length which is taken as the upper flange in this case.)

For lightweight concrete (Table D.1):

- $a_0 = -79.2 \text{ min}$;  
- $a_1 = 2.18 \text{ min/mm}$;  
- $a_2 = -2.44 \text{ min}$;  
- $a_3 = 0.56 \text{ min/mm}$;  
- $a_4 = -542 \text{ mm min}$; and  
- $a_5 = 52.3 \text{ min}$.

Fire resistance with respect to thermal insulation $t_i$ (in min) is given by (equation (D.1)):

$$t_i = a_0 + a_1 h_1 + a_2 \Phi + a_3 \frac{A}{L_r} + a_4 \frac{1}{l_3} + a_5 \frac{A}{L_r} \frac{1}{l_3}$$

$$= -79.2 + 2.18 \times 89 - 2.44 \times 0.136 + 0.56 \times 46.36 - \frac{542}{40} + \frac{52.3 \times 46.36}{40}$$

$$= 187.5 \text{ min} > 120 \text{ min}$$

Load-carrying capacity

The sagging and hogging moment capacities must be calculated and compared with the free bending moment.

Free bending moment $M_0$ is given by:

$$M_0 = \frac{L^2}{8} E_{\text{fl.d}} = \frac{3.5^2}{8} \times 5.33 = 8.16 \text{ kNm per metre width}$$

Sagging moment resistance $M_{\text{fl.Rd}}$

Temperature factors for lower flange (LWC and R120):

- $b_0 = 1062^\circ \text{C}$;  
- $b_1 = -399^\circ \text{C mm}$;  
- $b_2 = -0.65^\circ \text{C mm}$;  
- $b_3 = 19.8^\circ \text{C}$; and  
- $b_4 = -43.7^\circ \text{C}$  (Table D.2)

Temperature of lower flange $\theta_{a,LF}$ is given by (equation (D.4)):

$$\theta_{a,LF} = b_0 + b_1 \frac{1}{l_3} + b_2 \frac{A}{L_r} + b_3 \Phi + b_4 \Phi^2$$

$$= 1062 - \frac{399}{40} - 0.65 \times 46.36 + 19.8 \times 0.136 - 43.7 \times 0.136^2$$

$$= 1023.8^\circ \text{C}$$

Temperature factors for web (LWC and R120):

- $b_0 = 989^\circ \text{C}$;  
- $b_1 = -629^\circ \text{C mm}$;  
- $b_2 = -1.07^\circ \text{C mm}$;  
- $b_3 = 186.1^\circ \text{C}$ and  
- $b_4 = -152.6^\circ \text{C}$  (Table D.2)

Temperature of web $\theta_{a,web}$ is given by (equation (D.4)):

$$\theta_{a,web} = b_0 + b_1 \frac{1}{l_3} + b_2 \frac{A}{L_r} + b_3 \Phi + b_4 \Phi^2$$

$$= 989 - \frac{629}{40} - 1.07 \times 46.36 + 186.1 \times 0.136 - 152.6 \times 0.136^2$$

$$= 946.1^\circ \text{C}$$
Temperature factors for upper flange (LWC and R120):

\[ b_0 = 903^\circ C; \quad b_1 = -1561^\circ C \text{ mm}; \quad b_2 = -0.92^\circ C \text{ mm}; \]
\[ b_3 = 305.2^\circ C; \quad \text{ and } \quad b_4 = -197.2^\circ C \]  
(Table D.2)

Temperature of upper flange \( \theta_{a,UF} \) is given by (equation (D.4)):

\[
\theta_{a,UF} = b_0 + b_1 \frac{1}{L_3} + b_2 \frac{A}{L_3} + b_3 \Phi + b_4 \Phi^2
\]
\[
= 903 - \frac{1561}{40} - 0.92 \times 46.36 + 305.2 \times 0.136 - 1.97 \times 0.136^2
\]
\[
= 859.2^\circ C
\]

**Effective depth:**

\[
\frac{h_2}{h_1} = \frac{51}{89} = 0.57 < 1.5; \quad h_1 = 89 > 40 \text{ mm} \quad \text{(equation (D.4(1)))}
\]
\[
h_{eff} = h_1 + 0.5h_2 \left( \frac{h_1 + h_2}{h_1 + h_3} \right) = 130.8 \text{ mm} \quad \text{(equation (D.15a))}
\]

**Plastic neutral axis:**

\[
h_{el} = 130.8 \text{ mm}
\]

\[ \text{Fig. 10.3. Plastic neutral axis} \]

Tensile resistance of steel deck is given by (4.3.1):

\[
N_p = \sum_{i=1}^{n} A_i k_{y,i} \left( \frac{f_{y,i}}{\gamma_{M.f.a}} \right)
\]
\[
= 1.2 \times (137.5 \times 0.035 + 2 \times 52.5 \times 0.051 + 40 \times 0.08) \times 280/1000
\]
\[
= 4.50 \text{ kN per 152.5 mm}
\]
\[
= 29.5 \text{kN per metre width}
\]

Assuming that the top layer of concrete slabs are at a temperature below 300°C, the depth of concrete slab in compression \( x_{pl} \) is given by:

\[
\alpha_{slab} A_i k_{c,\theta} \left( \frac{f_{c,i}}{\gamma_{M.f.c}} \right) = N_p
\]
\[
0.85 \times 30 \times 1000 \times x_{pl} = 29.52 \times 1000
\]
\[
x_{pl} = 1.16 \text{ mm}
\]

As can be seen from Table D.5, this area of concrete is at a sufficiently low temperature for full strength to be assumed.

**Lever arm Z:**

\[
Z = \frac{\left\{ \sum_{i=1}^{n} A_i k_{y,i} z_i \left( \frac{f_{y,i}}{\gamma_{M.f.a}} \right) \right\}}{N_p} = 117.02 \text{ mm}
\]
Sagging moment capacity:

\[ M_{f,Rd}^s = N_p Z = 29.52 \times 117.02 / 1000 = 3.45 \text{kNm per metre width} \]

Hogging moment resistance \( M_{f,Rd}^h \):

Normal force in hogging reinforcement:

\[ N_s = 252 \times 500 / 1000 = 126 \text{kN per metre width} \]

Limiting temperature: (equation (D.7))

\[
\theta_{\text{lim}} = d_0 + d_1 N_s + d_2 A \frac{A}{L_c} + d_3 \Phi + d_4 \frac{1}{l_3} = 1213 - 2.5 \times 10^{-4} \times 126000 - 10.09 \times 46.36 - 214 \times 0.136 - 2320 / 40 = 626.6^{\circ}\text{C}
\]

\( z \)-factor: (D.3(6), equation (D.6))

\[
u_1 = 89.8 \text{ mm}; \quad u_2 = 89.8 \text{ mm};
\]

\[
u_3 = 0.75 \times h_3 = 38.25 \text{ mm (according to D.3(6))}
\]

\[
\frac{1}{z} = \frac{1}{\sqrt{\nu_1}} + \frac{1}{\sqrt{\nu_2}} + \frac{1}{\sqrt{\nu_3}} = 0.373
\]

Fig. 10.4. Calculation of \( z \) factor

\[
Y_1 = \left( \frac{1}{z} - \frac{4}{\sqrt{h_1 + l_3}} \right)^{-1} = 419.3 \text{ mm} > h_2 (= 51 \text{ mm}) \quad (D.3(5))
\]

In the case of \( Y_1 > h_2 \), the ribs of the slab may be neglected. Table D.5 may be used to obtain the location of the isotherm of the limiting temperature \( \theta_{\text{lim}} \) as a conservative approximation.

For \( \theta_{\text{lim}} = 626.6^{\circ}\text{C} \), the depth \( x = 20.1 \text{ mm (D.3(7))} \)

(Note: for lightweight concrete, the temperatures of Table D.5 are reduced to 90% of the values given.) (D.3(9))

Fig. 10.5. Depth of limiting temperature isotherm

Depth of concrete slab in compression \( x_{pl} \):

\[
x_{pl} = \frac{126}{0.85 \times 30} = 4.94 \text{ mm}
\]
10.3. Reinforced concrete slabs

The simplified calculation method in EN 1992-1-2 is based on the reduced slab size in which the fire-damaged concrete zone of the slab is obtained in exactly the same way as described in Section 8.5. Afterwards, the slab design calculations follow EN 1992-1-1, but using concrete mechanical properties at the relevant elevated temperature.

The following worked example shows how to calculate the reduced slab thickness using both the zone method and the 500°C isotherm method.

Example 10.3: Calculation of reduced slab thickness

Calculate the reduced thickness and design strength of concrete for a solid concrete slab in fire. The design standard fire resistance rating is 90 min. The original slab thickness is 100 mm.

Calculation results

(1) Using the zone method

The slab is heated from one side so the half-thickness of the equivalent wall (w₁ in Fig. 8.6(b)) is the thickness of the slab, w = 100 mm.

Divide the slab thickness into four zones, each 25 mm in thickness. The distances from the centres of the four zones to the exposed surface of the slab are 12.5 mm, 37.5 mm, 62.5 mm and 87.5 mm. According to Fig. 4.7, the temperatures in these zones are 725°C, 420°C, 240°C and 140°C. The temperature at the unexposed surface of the slab is \( \theta_M = 110°C \). From Table 6.4 (for siliceous NWC), the strength reduction factors for these zones are 0.2625, 0.72, 0.91 and 0.98. The strength reduction factor for the unexposed surface of the slab is \( k_{c\theta_M} = 0.995 \).

From equation (8.17), the mean reduction coefficient for the slab is:

\[
k_{c,\text{mean}} = \frac{(1 - 0.2/4)}{4} (0.2625 + 0.72 + 0.91 + 0.98) = 0.682
\]

From equation (8.18), the width of the fire-damaged zone for the slab is:

\[
a_z = 100 \left(1 - \frac{0.682}{0.995}\right) = 31.5
\]

The reduced thickness of the slab is 100 – 31.5 = 68.5 mm.
Afterwards, EN 1992-1-1 should be followed, but using the above reduced thickness of the slab and the reduced design strength of concrete, which is 0.995 times that at ambient temperature.

(2) Using the 500°C isotherm method
Using Fig. 4.7, the distance of the 500°C isotherm from the exposed surface is 30 mm. This thickness should be ignored and the reduced thickness of the slab is 100 – 30 = 70 mm. Slab design should follow EN 1992-1-1, but using the reduced slab thickness and the concrete design strength at ambient temperature.
CHAPTER 11

Other forms of construction

11.1. Introduction
A number of major research studies have been conducted to investigate the fire resistance of various forms of integrated steel construction where adequate fire protection to the steelwork is provided by existing structural components so that there is no need for extra fire protection. EN 1994-1-2 contains a number of such systems, for example partially encased universal beams and columns, concrete-filled tubular columns, where concrete acts both as part of the structural load-bearing system as well as providing fire protection to the steel. However, a number of such forms of construction is not included in EN 1994-1-2 and this chapter provides a description of the following systems: slim floor beams, shelf angle beams and blocked infilled columns.

11.2. Slim floor beams
A slim floor beam is manufactured by welding a wide steel plate to the bottom flange of a universal column section. The concrete/composite slab is supported on this additional plate. Figure 11.1(a) provides an illustration of this system. The Corus asymmetric steel beam (ASB) system is similar to the slim floor beam system, the difference being that an ASB section is rolled as one and no additional welding is necessary (Fig. 11.1(b)). In both systems, concrete surrounds the upper flange and web of the steel beam and only the lower flange is exposed to fire. Thus, there is a significant residual strength in the steel beam. Results from the standard fire resistance tests indicate that this form of construction can achieve a standard fire resistance rating of 60 min without fire protection.

11.3. Shelf angle beams
A shelf angle beam is manufactured by welding a pair of steel angle sections on both sides of the web of a universal beam section. The function of these angle sections is to support the concrete floor slabs. Figure 11.2 provides an illustration of this form of construction. Under fire conditions, only the lower flange and the lower web are exposed to fire. The upper flange and the upper web are protected by the concrete floor slabs. The steel angles can also contribute to the fire resistance of the system. Results of the standard fire resistance tests suggest that this form of construction can achieve at least 30 min of standard fire resistance without fire protection.

The standard fire resistance calculation for this form of construction is given in Appendix E of the British Standard BS 5950 Part 8. This calculation method is based on plastic analysis of the steel section and the contribution of the concrete slabs is ignored. To ensure fire protection of the steel section by the concrete slabs, the construction details in Section E.1 of BS 5950 Part 8 should be observed.
Appendix E of BS 5950 Part 8 contains an approximate calculation method to determine the temperatures in the steel section including the steel angles under the standard fire condition. This method is presented below. In this method, the steel section is divided into seven blocks as shown in Fig. 11.3.

Block 1 is the lower flange. Its temperature may be calculated using equation (4.1) and the section factor of the unprotected lower flange given by:

\[
\frac{A_m}{V} = \frac{2(B_e + t_f)}{B_c t_f} \approx \frac{2}{t_f}
\]  \hspace{1cm} (11.1)

where \(t_f\) is the thickness of the lower flange.

---

**Fig. 11.1.** (a) Slim floor beam and (b) asymmetric beam

---

**Fig. 11.2.** Cross-section of shelf angle beam
Block 2 is the lower web of the steel section. An analysis of the standard fire resistance test results indicates that the temperature in block 2 is directly related to the lower flange temperature, being slightly modified by the rate of heat conduction to the concrete slab. The rate of heat conduction to the concrete slab is represented by the exposed steel web depth to flange width ratio \( D_e = \frac{B}{C} \). Table 11.1 gives temperatures in the lower web of the steel section. In Table 11.1, \( T_1 \) is the temperature of the lower flange of the steel section.

Block 3 is the exposed leg of the angle section. Its temperature is a function of the aspect ratio and is given in Table 11.2.

The exact locations of blocks 4, 5 and 6 depend on the location of the 300°C line. If the steel temperature is below 300°C, it is assumed to retain its full strength. Figure 11.4 illustrates the two possible locations of the 300°C temperature line and the definition of the three blocks.

![Fig. 11.3. Temperature blocks for shelf angle beams](image)

**Table 11.1. Temperature of the exposed web (block 2 in Fig. 11.3) of a shelf angle beam**

<table>
<thead>
<tr>
<th>Aspect ratio ( D_e/B )</th>
<th>30 min</th>
<th>60 min</th>
<th>90 min</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_e/B \leq 0.6 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
</tr>
<tr>
<td>( 0.6 &lt; D_e/B \leq 0.8 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
</tr>
<tr>
<td>( 0.8 &lt; D_e/B \leq 1.1 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
</tr>
<tr>
<td>( 1.1 &lt; D_e/B \leq 1.5 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
</tr>
<tr>
<td>( 1.5 &lt; D_e/B )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
<td>( T_1 )</td>
</tr>
</tbody>
</table>

**Table 11.2. Temperature of the exposed angle leg of a shelf angle beam**

<table>
<thead>
<tr>
<th>Aspect ratio ( D_e/B )</th>
<th>30 min</th>
<th>60 min</th>
<th>90 min</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_e/B \leq 0.6 )</td>
<td>475</td>
<td>725</td>
<td>900</td>
</tr>
<tr>
<td>( 0.6 &lt; D_e/B \leq 0.8 )</td>
<td>510</td>
<td>745</td>
<td>910</td>
</tr>
<tr>
<td>( 0.8 &lt; D_e/B \leq 1.1 )</td>
<td>550</td>
<td>765</td>
<td>925</td>
</tr>
<tr>
<td>( 1.1 &lt; D_e/B \leq 1.5 )</td>
<td>550</td>
<td>765</td>
<td>925</td>
</tr>
<tr>
<td>( 1.5 &lt; D_e/B )</td>
<td>550</td>
<td>765</td>
<td>925</td>
</tr>
</tbody>
</table>
Blocks 6 and 7 have temperatures lower than 300°C. Since the full steel strength is retained, no further calculation is necessary.

Temperatures in blocks 4 and 5 are calculated using the following equation:

\[ T_x = \frac{T_R}{G} - 300 \text{°C} \]

where \( T_R \) is the temperature in the angle root; \( x \) is the distance (in mm) from the angle root and \( G \) is the temperature gradient (in °C/mm) in this region. The temperature gradients are 2.3°C/mm, 3.8°C/mm and 4.3°C/mm for the standard fire resistance periods of 30 min, 60 min and 90 min respectively.

From equation (11.1), the location of the 300°C is given by:

\[ x = \frac{T_R}{G} - 300 \text{°C} \]

Values of \( T_R \) are given in Table 11.3.

**Table 11.3. Temperature \( T_R \)**

<table>
<thead>
<tr>
<th>Aspect ratio ( D_e/B )</th>
<th>30 min</th>
<th>60 min</th>
<th>90 min</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 0 &lt; D_e/B \leq 0.6 )</td>
<td>350</td>
<td>600</td>
<td>775</td>
</tr>
<tr>
<td>( 0.6 &lt; D_e/B \leq 0.8 )</td>
<td>385</td>
<td>620</td>
<td>785</td>
</tr>
<tr>
<td>( 0.8 &lt; D_e/B \leq 1.1 )</td>
<td>425</td>
<td>640</td>
<td>800</td>
</tr>
<tr>
<td>( 1 &lt; D_e/B \leq 1.5 )</td>
<td>425</td>
<td>640</td>
<td>800</td>
</tr>
<tr>
<td>( 1.5 &lt; D_e/B )</td>
<td>425</td>
<td>640</td>
<td>800</td>
</tr>
</tbody>
</table>

**11.4. Blocked infilled columns**

The fire resistance of a bare steel column is increased significantly if autoclaved aerated concrete blocks are placed between the inner faces of the flanges. It is (conservatively) assumed that the blocks do not contribute to the load-carrying capacity of the member but provide insulation to the web of the column and the inner surface of the flanges, leading to a reduction in the temperature rise of the steel section. The situation is illustrated in Fig. 11.5.

This solution is suitable for 30 min fire resistance provided that the load ratio does not exceed 0.6 and that the section factor does not exceed 69 m⁻¹. The limiting values are based on tests carried out by British Steel and the Building Research Establishment (BRE) and are reported in BRE Digest 317.\(^{41}\) The minimum size of column required to achieve 30 min fire resistance is a \( 203 \times 203 \times 52 \text{UC} \) and the concrete blocks should have a minimum density of 475 kg/m³ in order to achieve the required level of insulation.

If fire resistance periods in excess of 30 min are required, a number of solutions are available using partially encased columns and concrete-filled rectangular hollow sections. Details of these methods are provided in a Steel Construction Institute design guide\(^{42}\) and illustrated in Fig. 11.5.
Fig. 11.5. Partially protected columns: (a) blocked-in steel column; (b) partially encased steel column (unreinforced); (c) partially encased steel column (reinforced); and (d) concrete-filled steel section
12.1. Introduction

Traditional fire design has been based around the performance of individual elements (beams, columns, walls, floor slabs). The assessment procedures (standard fire tests) do not consider the interaction between structural elements in a realistic steel- or concrete-framed building. The behaviour of connections in a fire or post-fire condition can be critical in terms of maintaining overall structural stability. The Eurocodes encourage designers to consider the behaviour of connections explicitly and provide greater flexibility in terms of available design options.

The most common approach to connection design in fire has been to ensure that the available cover and minimum dimensions of the connection is at least equivalent to that of the connected parts (concrete construction) or that the thickness of applied passive fire protection is at least equal to that used for the connected members (steel and composite construction). However, such an approach fails to consider:

- the applied load level of the connection relative to the connected members
- the ductility required to accommodate the large deformations associated with the fire limit state
- the tensile strength required to resist the large tensile forces generated during the cooling phase of a real fire.

12.2. Concrete connections

The design of concrete connections in fire is governed by the same principles and assumptions that apply to other structural members (beams, columns, walls, floor slabs); that is, fire resistance is a function of the cross-sectional dimensions and the cover to the reinforcement. This design philosophy is based on the large thermal inertia of concrete structures which is a function of their high mass and low thermal conductivity. Most concrete connections do not therefore require additional fire protection in order to fulfil the mandatory functional requirements.

Detailing of concrete structures is particularly important in relation to performance in fire. A number of authoritative guidance documents are available that set out detailing rules for enhanced performance in fire. These include:

- anchoring reinforcement
- continuous top and bottom reinforcement over supports with effective overlaps to prevent premature failure due to stress reversal
- fire stopping around service penetrations.

Connection performance is subject to the effects of both permanent and variable (dead and imposed load) actions and indirect actions arising from the effects of the fire. In general, the
former do not present any particular design problems as the load levels are generally lower at the fire limit state. However, indirect actions have a significant impact on the performance of connections in fire.

12.2.1. Increase in support moment for continuous structures
The thermal expansion of the exposed surface of a beam or floor slab induces curvature which has the effect of increasing the support moment on the unexposed face. This may lead to yielding of the top reinforcement if not considered at the design stage. Adequate overlapping of reinforcement at the support is recommended for reinforced structures. Precast structures are generally simply supported and have sufficient rotational capacity to deal with this effect. The effect of thermal curvature is illustrated in Fig. 12.1.

12.2.2. Forces due to restrained thermal expansion
When a fire compartment is located within a framed structure, the expansion of the heated elements is restrained by the surrounding cold structure. This restraint to thermal expansion leads to very high compressive forces acting on the connection and has been confirmed through large-scale testing. The situation is illustrated schematically in Fig. 12.2. Concrete connections are likely to be able to accommodate such large forces. However, on cooling, large tensile forces can also be generated and these are not normally taken into account at the design stage.

12.2.3. Eccentricity of loading due to large deflection
The effect of large deformations is to increase the eccentricity of loading at the connection. Sufficient rotational capacity must be present to accommodate the large vertical deflections typical of exposed floor slabs in a fire situation. Local damage at the support may result from the curvature of beams and floors leading to localized failure of the supporting member.

In a framed structure consisting of continuous columns, the horizontal movement of the floor slab may lead to significant lateral deformation of columns, leading to large stresses and possible shear failure. This situation is particularly significant on edge columns where the expansion towards the inside of the building is restrained by the surrounding cold structure.

Fig. 12.1. Moment distribution in a continuous structure

Fig. 12.2. Compressive forces in a framed structure due to restrained thermal expansion
This situation was illustrated in the large-scale fire test undertaken on the European concrete building at Cardington (Fig. 12.3).

There is very little guidance in EN 1992-1-2 on the design of joints other than to specify that the design of joints should be based on an overall assessment of the structural behaviour in fire.

In terms of meeting the insulation requirements for fire resistance, gaps in joints between members are restricted to 20 mm and should not be deeper than half the minimum thickness of the separating component.

12.3. Steel and composite connections

Connections (or joints to use the European terminology) to structural steelwork (whether composite or not) generally utilize standard connections with standard bolt sizes and plate thickness. This simplifies both the design and fabrication process. Guidance on the selection of appropriate connections is available in established guidance documents produced by the industry.\textsuperscript{35–47} In general, connections may be classified as either nominally pinned or fully fixed depending on their ability to transfer moments from the loaded beams into the columns. In reality, all connections are, to some extent, semi-rigid – that is, they have some measure of rotation capacity and some degree of fixity. One interesting aspect of connection behaviour in fire is that connections which are assumed to be simply supported under normal conditions actually behave as semi-rigid connections in the event of a fire.
The traditional approach to connection design in the UK has been for the structural engineer to set out the design assumptions in relation to the connections and to leave the detailed design (end-plate thickness, bolt pitch, etc.) to the fabricator. Connections are generally assumed to be simple or moment resisting with simple connections designed to resist shear loads, axial loads and notional moments. The Eurocodes present a more rigorous design approach that classifies connections according to strength and stiffness and brings in the concept of partial strength and semi-rigid connections.

The traditional approach to the design of connections in fire is to ensure there is sufficient passive fire protection over the connection. In general, this is assumed to mean at least the same thickness as that on the connected elements (beams and columns); however, the connection may be loaded to a higher level than the connected parts. As load level is connected to fire resistance, this may mean insufficient passive fire protection is applied to the connection. In addition, the connection may have insufficient ductility to accommodate the large deflections and rotations typical of the fire limit state. Many of the issues discussed above in relation to concrete connections also apply here. Of particular importance is the rotational capacity of the connection, the effects of restrained thermal expansion in generating significant compressive stresses and the tensile capacity of the connection during the cooling phase.

Two methods are given in the fire part of the Eurocode for the design of steel structures, EN 1993-1-2. The first is the simplified procedure set out in clause 4.2.1(6) whereby:

The fire resistance of a bolted or welded connection may be assumed to be sufficient provided that the following conditions are satisfied. (Note: the first condition corresponds to traditional UK design assumptions.)

1. The thermal resistance \( (d_f/\lambda_f) \) of the connection’s fire protection should be greater than the minimum value of thermal resistance \( (d_f/\lambda_f)_m \) of fire protection applied to any of the connected members.
2. The utilization of the connection should be less than the maximum value of utilization of any of the connected members.
3. The resistance of the connection at ambient temperature should satisfy the recommendations given in EN 1993-1-8 (connection design part of EN 1993).

Annex D of EN 1993-1-2 provides an alternative method for calculating the temperature distribution through the joint. Once the temperature distribution has been derived then the capacity in shear, bearing and tension is calculated using appropriate reduction factors to allow for the effects of elevated temperature.

The temperature of the connection in fire is given for beam depths less than 400 mm:

\[
\theta_h = 0.88\theta_0[1 - 0.3(h/D)]
\]

where:

- \( \theta_h \) is the temperature at height \( h \) (mm) of the steel beam (°C);
- \( \theta_0 \) is the bottom flange temperature of the steel beam remote from the connection (°C);
- \( h \) is the height of the component being considered above the bottom of the beam (mm);
- \( D \) is the depth of the beam (mm).

If the depth of the beam is greater than 400 mm:

(i) when \( h \) is less than \( D/2 \):

\[
\theta_h = 0.88\theta_0
\]

(ii) when \( h \) is greater than \( D/2 \):

\[
\theta_h = 0.88\theta_0[1 + 0.2(1 - 2h/D)]
\]

The situation is illustrated in Fig. 12.4.

This methodology has been validated against full-scale test data, as shown in Fig. 12.5.
In most current design methods the underlying assumption is that the components of the connection and the supporting members have the same rate of strength reduction. However, this is not necessarily the case. Figure 12.6 compares the strength reduction factors for members, bolts and welds.

**Fig. 12.4.** Thermal gradient within the depth of a composite connection

**Fig. 12.5.** Comparison between Eurocode prediction and test results

In most current design methods the underlying assumption is that the components of the connection and the supporting members have the same rate of strength reduction. However, this is not necessarily the case. Figure 12.6 compares the strength reduction factors for members, bolts and welds.

**Fig. 12.6.** Reduction factors for members, bolts and welds
From the figure it can be seen that, between 350°C and 1000°C, the strength of welds reduces faster than that of the connecting member. Similarly, between 100°C and 600°C the strength of bolts reduces faster than the connected member.

Most practical connections support the applied load by a combination of welds and bolts. It is therefore difficult to say with certainty which of these two components will govern at the fire limit state. Based on the data available, a connection with the same level of fire protection as the connected member will lose its strength at a faster rate.

However, the temperature profile through a connection is not the same as that of the connected member. The heating rate tends to be lower through a combination of higher thermal mass at the connection location and the effects of shielding from the connected members. It is concluded that the disadvantages associated with the strength reduction of bolts and welds are outweighed by the advantages associated with the reduced thermal gradient through the connection. Therefore, the simple rules detailed in the Eurocode do not represent a reduction in existing levels of safety.

The Eurocode connection design procedure is illustrated with reference to a worked example.

**Example 12.1: Major axis beam-to-column connection**

Consider the major axis beam-to-column connection illustrated in Figs 12.7 and 12.8. In EN 1993-1-2 two methods are presented for bolted or welded joints. The first is based on ensuring that the fire resistance of the joint is greater than or equal to that of the connected members. In general this is a conservative method as the temperature of the connection is generally less than that of the beams. However, it is also necessary to consider the utilization of the connection compared to the utilization of the member. As a simplification, the utilization of the joint and the connected members may be related to the loading and resistance at ambient temperature.

**Fig. 12.7. Details of major axis beam-to-column connection (all dimensions in mm)**
Alternatively, the resistance of the joint may be assessed according to Annex D of EN 1993-1-2 whereby the temperature of the components are calculated and reduction factors used to determine the resistance of the joint.

Consider the connection at location E1 in Fig. 12.8.

Loading

The values of the actions at the fire limit state are given in Table 12.1.

For the purpose of design, the partition load is classed as imposed to account for demountable partitions. For the fire limit state, the partition load is included in the dead load.

<table>
<thead>
<tr>
<th>Nature of loading</th>
<th>Value (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite slab</td>
<td>2.06</td>
</tr>
<tr>
<td>Structural steel sections</td>
<td>0.25</td>
</tr>
<tr>
<td>Raised floor</td>
<td>0.40</td>
</tr>
<tr>
<td>Services</td>
<td>0.25</td>
</tr>
<tr>
<td>Ceiling</td>
<td>0.15</td>
</tr>
<tr>
<td>Partitions</td>
<td>1.00</td>
</tr>
<tr>
<td>Imposed</td>
<td>2.50</td>
</tr>
</tbody>
</table>

**Permanent actions (G)**
Uniformly distributed load \( G_k = 3.11 \text{ kN/m}^2 \) (4.11 kN/m² fire limit state)

**Variable actions (Q)**
Uniformly distributed load \( Q_k = 3.50 \text{ kN/m}^2 \) (2.5 kN/m² fire limit state)

**Loading factors – ambient temperature**
Partial factor for permanent actions: \( \gamma_g = 1.35 \)
Partial factor for variable actions: \( \gamma_Q = 1.50 \) (EN 1990, Table A1.2(B))

**Loading factors – fire limit state**
For the fire limit state, partial loading factors (\( \gamma_i \)) are not applied to either permanent actions or variable actions.
Combination coefficient for variable action \( \psi_1 = 0.50 \) (EN 1990, Table A1.3)
Ambient temperature design value of actions
• Ultimate limit state

Design UDL, $F_{Ed} = (\gamma_G \times G_k) + (\gamma_Q \times Q_k) = 9.45 \text{kN/m}^2$

**Design moment – primary beam**
The design moment on the primary beam is equal to:

$$M_{Ed} = (R \times l)/4$$

where:

$R$ is the end reactions from the secondary beams framing into the primary beam between gridlines E1 and E2:

$$R = [(l/2) \times L \times F_{Ed}] = 255.1 \text{kN}$$

Therefore:

$$M_{Ed} = (R \times l)/4 = 382.66 \text{kNm}$$

**Design shear force – primary beam**
The design shear force is equal to the end reaction on the primary beam.

$$V_{Ed}R/2 = 127.6 \text{kN}$$

**Fire limit state design value of actions**

• Ultimate limit state accidental design situation

Design UDL, $F_{Ed,fi} = G_k + (\psi_1 \times Q_k) = 5.36$ (EN1990, Table A1.3)

**Design moment – primary beam**
The design moment on the primary beam is equal to:

$$M_{Ed,fi} = (R_{fi} \times l)/4$$

where:

$R_{fi}$ is the end reaction from the secondary beams framing into the primary beam between gridlines E1 and E2:

$$R_{fi} = [(l/2) \times L \times F_{Ed,fi}] = 144.7 \text{kN}$$

Therefore:

$$M_{Ed,fi} = (R_{fi} \times l)/4 = 217 \text{kNm}$$

**Design shear force – primary beam**
The design shear force is equal to the end reaction on the primary beam.

$$V_{Ed,fi} = R_{fi}/2 = 72.3 \text{kN}$$

• **Method 1:**

$$(d_f/\lambda_f)_c \geq (d_f/\lambda_f)_m$$  \hspace{1cm} (clause 4.2.1(6))

where:

$(d_f/\lambda_f)_c$ is the relationship between the thickness of the fire protection material and the thermal conductivity of the fire protection material for the connection; and

$(d_f/\lambda_f)_m$ is the relationship between the thickness of the fire protection material and the thermal conductivity of the fire protection material for the connected member.
Resistance of connection – ambient temperature design

The connection (E1) is designed as simply supported at ambient temperature and it is acceptable to carry out the utilization check at ambient temperature. The shear capacity of the connection is assessed using the method detailed in the BCSA/SCI green book on simple connections. The shear capacity of the connection based on the shear capacity of the bolt group, the shear capacity of the end plate, the block shear capacity and the bearing capacity of the end plate are summarized in Table 12.2. As the column flange is much thicker than the end plate there is no need to consider the resistance of the column flange in bearing.

Table 12.2. End-plate shear and bearing capacity

<table>
<thead>
<tr>
<th>Resistance check</th>
<th>Formula</th>
<th>Resistance (kN)</th>
<th>Green book page ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear capacity of bolt group (F_s \leq \Sigma P_s)</td>
<td>(\Sigma p_i A_i (or 0.5k_b e_i t_p p_{bs})) for top bolt rows</td>
<td>699</td>
<td>93</td>
</tr>
<tr>
<td>Plain shear capacity of end plate (F_s/2 \leq P_s)</td>
<td>(\text{Min} (0.6p_i A_i, 0.7p_k K_p A_{net}))</td>
<td>270</td>
<td>94</td>
</tr>
<tr>
<td>Block shear (F_s/2 \leq P_s)</td>
<td>(0.6 p_k t_p [t_p + K_p (t_p + k D_b)])</td>
<td>320</td>
<td>94</td>
</tr>
<tr>
<td>Bearing (F_s/2 \leq P_s)</td>
<td>(k_{bs} d_{p} p_{bs})</td>
<td>294</td>
<td>94</td>
</tr>
</tbody>
</table>

Note: symbols used are taken from the BCSA/SCI green book. \(F_s\) is the design shear force, which equals \(V_{Ed}\) given in EC3.

Therefore the utilization of the connection is

\[
(V_{Ed}/2)/270 = 0.236
\]

This value needs to be compared to the degree of utilization of the beam connected to the column.

Resistance of primary beams – ambient temperature design

The moment capacity of the composite primary beam is:

\[ M_{c,Rd} = 515 \text{ kNm} \]

(Note: this value has been determined following the method given in SCI publication P055. The calculation process is not included in this worked example, as it is concerned with the design of the connection at the fire limit state.)

Therefore the utilization of the beam is:

\[ M_{Ed}/M_{c,Rd} = 0.74 \]

where \(M_{Ed}\) is the design moment determined previously.

Determination of fire protection thickness

The utilization of the beam is greater than that of the connection therefore it is sufficient to ensure that the fire protection is at least equivalent to that on the beam. The selection of the appropriate beam protection thickness and thermal conductivity can be made on the basis of the calculation procedure for protected steelwork detailed in Chapter 4.

- Method 2:

Annex D of EN 1992-1-2 provides a method for determining the temperature profile within the connection. This can then be used to derive reduction factors corresponding to the location of the individual components.

The first step is to calculate the temperature rise of the bottom flange (at mid-span) of the connected beam. For this example it is assumed that the required period of fire
resistance is 60 min and that the applied passive fire protection to be used is 20 mm gypsum board applied to three sides of the beam. The relevant formula for protected members is:

\[
\Delta \theta_{a,t} = \frac{\lambda_p A_p}{d_p c_a \rho_a} \left( \frac{\theta_{g,t}}{\theta_{a,t}} - \theta_{a,t} \right) \Delta t - \left( e^{\phi/10} - 1 \right) \Delta \theta_{g,t} \quad \Delta \theta_{a,t} \geq 0
\]

and

\[
\Phi = \frac{c_p \rho_p d_p A_p}{c_a \rho_a V}
\]

where:

- \( A_p/v \) is the section factor for protected steel member (136 m\(^{-1}\));
- \( c_a \) is the specific heat of the steel (600 J/kgK);
- \( c_p \) is the specific heat of the protective material (1700 J/kgK);
- \( d_p \) is the thickness of fire protection (0.02 m);
- \( \theta_{a,t} \) is the temperature of the steel at time \( t \) (°C);
- \( \theta_{g,t} \) is the temperature of the gas at time \( t \) (°C);
- \( \Delta \theta_{g,t} \) is the increase in gas temperature over the time step \( t \) (°C);
- \( \lambda_p \) is the thermal conductivity of the fire protection material (0.2 W/mK);
- \( \rho_a \) is the density of the steel (7850 kg/m\(^3\)); and
- \( \rho_p \) is the density of the protection material (800 kg/m\(^3\)).

\[ \Phi = 0.7854 \]

For the standard fire exposure and the specified protection material the temperature of the steel beam is calculated as 445°C. The time–temperature relationship is illustrated in Fig. 12.9.

![Temperature of primary beam (356 x 171UB51)](image)

(Note: this is not a particularly efficient design solution. The designer may wish to consider rationalizing the fire protection (by using a 15 mm board for example) to increase the maximum temperature in the steel beam.)

Here the depth of the beam is less than or equal to 400 mm, therefore:

\[
\theta_h = 0.88 \theta_o [1 - 0.3(h/D)]
\]

where:

- \( \theta_h \) is the temperature at height \( h \) (mm) of the steel beam;
\( \theta_0 \) is the bottom flange temperature of the steel beam at mid-span (445°C); 
\( h \) is the height of the component being considered above the bottom of the beam (mm); and 
\( D \) is the depth of the beam (mm) (355 mm).

The temperature of the critical components is illustrated graphically in Fig. 12.10. The values are summarized in the Table 12.3 below.

The temperatures at each location are used to derive reduction factors for the individual components either from Table D.1 for bolts or from Table 3.1 for the end plate and column.

**Table 12.3. Temperature of critical components**

<table>
<thead>
<tr>
<th>Description</th>
<th>Distance from bottom flange (mm)</th>
<th>Temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom flange of steel beam at mid-span</td>
<td>0</td>
<td>445</td>
</tr>
<tr>
<td>Bottom flange of the steel beam in the vicinity of the connection</td>
<td>0</td>
<td>392</td>
</tr>
<tr>
<td>Bottom edge of end plate</td>
<td>75</td>
<td>367</td>
</tr>
<tr>
<td>Bottom bolt row</td>
<td>115</td>
<td>354</td>
</tr>
<tr>
<td>2nd bolt row from bottom</td>
<td>175</td>
<td>334</td>
</tr>
<tr>
<td>3rd bolt row from bottom</td>
<td>235</td>
<td>314</td>
</tr>
<tr>
<td>Top bolt row</td>
<td>295</td>
<td>294</td>
</tr>
<tr>
<td>Top edge of end plate</td>
<td>335</td>
<td>281</td>
</tr>
</tbody>
</table>

The original checks are then repeated using the reduction factors for elevated temperature and compared to the reduced load applied at the fire limit state. In this case the design shear force is reduced according to the fire limit state load factors.

The reduction factors for the individual components are summarized in Table 12.4 and the corresponding resistance checks are summarized in Table 12.5.

The utilization of the connection at the fire limit state is:

\[
\frac{(V_{Ed,f})}{270} = 0.14
\]
Table 12.4. Reduction factors

<table>
<thead>
<tr>
<th>Component</th>
<th>Reduction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>End plate (based on bottom temperature)</td>
<td>1.0</td>
</tr>
<tr>
<td>Bottom bolt row</td>
<td>0.83</td>
</tr>
<tr>
<td>2nd bolt row from bottom</td>
<td>0.885</td>
</tr>
<tr>
<td>3rd bolt row from bottom</td>
<td>0.86</td>
</tr>
<tr>
<td>Top bolt row</td>
<td>0.933</td>
</tr>
</tbody>
</table>

In this example the connection is utilized less at the fire limit state than at ambient temperature. This is because the reduction in the applied load at the fire limit state is greater than the reduction in material properties of the connection components.

It should be noted that for moment connections it is more likely that the utilization of the connection would be higher than that of the connected beam, and that for unprotected connections the reduction in the strength of the connection components would be much greater.

Table 12.5. Resistance at elevated temperature

<table>
<thead>
<tr>
<th>Resistance check</th>
<th>Formula</th>
<th>Resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear capacity of bolt group</td>
<td>( \sum p_i A_i \times k_b ) (or ( k_b \times 0.5 \times c_i t_b p_{bs} )) for top bolt rows</td>
<td>611</td>
</tr>
<tr>
<td>Plain shear capacity of end plate</td>
<td>Min ( k_y \times 0.6 p_i A_i + k_y \times 0.7 p_i K_{A\text{net}} )</td>
<td>270</td>
</tr>
<tr>
<td>Block shear ( F_s / 2 \leq P_s )</td>
<td>( k_y \times 0.6 p_i t_p (L_v + K_e (L_t - kD_t)) )</td>
<td>320</td>
</tr>
<tr>
<td>Bearing ( F_s / 2 \leq P_{bs} )</td>
<td>( k_b k_{bs} d_{ps} p_{bs} )</td>
<td>258</td>
</tr>
</tbody>
</table>

Note: the symbols used are taken from the BCSA/SCI green book.46 \( F_s \) is the design shear force, which equals \( V_{Ed,b} \) given in EC3.
CHAPTER 13

General discussion

13.1. Introduction
The design methodologies presented in the structural Eurocodes provide a framework to facilitate the performance-based design of structures in fire while enabling accepted prescriptive solutions to be adopted where required.

The calculation methods provide a more rational basis for the fire engineering design of structures and provide greater flexibility to engineers, architects and end users in relation to the design of new buildings and the refurbishment/reuse of the existing building stock.

In general, this increased flexibility is achieved at the cost of increased design effort. Structural fire engineering covers a wide spectrum of approaches to the nature and effects of the loading and the means of ensuring adequate resistance for the required duration. On the one hand there is a simple reliance on values from published tables based on a simplified assessment of both the effects of the fire and the load acting on the structure at the time of the fire. At the other extreme the designer may choose to model the fire using complex computational fluid dynamics techniques and analyse the entire building using non-linear finite-element analysis. The design solution adopted will depend on the particular circumstances of the project and the requirements of the client and regulatory authorities. It is necessary to consider the financial implications of adopting a more sophisticated approach to the design of structures in fire. Such methods can only be justified where significant savings in material or enhanced levels of safety (over and above those required by National regulations) are required. The general recommendation is to use the simplest approach commensurate with the requirements for the building. Although the Eurocodes only set out the general principles associated with advanced fire engineering methods, more detailed guidance is now available.

The minimum fire resistance requirements are defined in National regulations based on a consideration of life safety of building occupants, those in the vicinity of the building and the fire service. Other issues such as property protection, protection of the environment and business continuity are not considered. Therefore, in certain cases, a level of safety over and above that required by National regulations may be appropriate.

13.2. Guidance on selection of appropriate design method
The hierarchy in terms of complexity of design methods is tabulated data followed by simplified calculation methods followed by advanced calculation methods. For the designer the tabulated approach should be the first port of call. This is particularly relevant in relation to concrete and composite structures. Simplified calculation methods are appropriate for steel and composite buildings and concrete buildings where the dimensions or cover required do not meet the specified fire resistance period or where renovation of an existing structure involves a change of use resulting in a new fire resistance category being applied to the
building. Calculation methods can be used to demonstrate performance under specific conditions and may provide substantial savings (for example in applied passive fire protection to steel structures) in certain circumstances. Advanced calculation methods (typically non-linear finite-element models) may be used where the structure is very complex and where the provisions of the National regulations are not applicable. Examples of such structures would include sports stadia, exhibition halls and airport terminals.
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