DRAFT FOR DEVELOPMENT

DD ENV 1993-1-2:2001

Corrected and reprinted September 2001

Eurocode 3: Design of steel structures —

Part 1.2: General rules — Structural fire design

(together with United Kingdom National Application Document)

ICS 13.220.20; 91.010.30; 91.080.10

Committees responsible for this Draft for Development

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

British Constructional Steelwork Association

Cold Rolled Sections Association

Confederation of British Forgers

Department of the Environment, Transport and the Regions

Department of the Environment, Transport and the Regions — Construction Directorate

Department of the Environment, Transport and the Regions — Highways Agency

Health and Safety Executive

Institution of Civil Engineers

Institution of Structural Engineers

Steel Construction Institute

UK Steel Association

Welding Institute

This Draft for Development, having been prepared under the direction of the Sector Committee for Building and Civil Engineering was published under the authority of the Standards Committee and comes into effect on 15 June 2001

The following BSI references relate to the work on this Draft for Development:
Committee reference B/525/31

ISBN 0 580 33218 7

Amendments issued since publication

Amd. No	Date	Comments

Contents

National foreword

This publication has been prepared by Subcommittee B/525/31 in cooperation with B/525/4 and is the English language version of ENV 1993-1-2:1995, *Eurocode 3: Design of steel structures — Part 1.2: General rules — Structural fire design*, as published by the European Committee for Standardization (CEN). This Draft for Development also includes the United Kingdom (UK) National Application Document (NAD) to be used with the ENV in the design of buildings to be constructed in the UK.

ENV 1993-1-2:1995 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

This publication should not be regarded as a British Standard.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard.

The values for certain parameters in the ENV Eurocodes may be set by CEN Members so as to meet the requirements of national regulations. These parameters are designated by □ (boxed values) in the ENV.

During the ENV period of validity, reference should be made to the supporting documents listed in the NAD.

The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK. The NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992 (published December 1991)[1)], draws designers' attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1993-1-2:1995 has been thoroughly examined over a period of several years and is considered to offer such an alternative approach, when used in conjunction with this NAD.

Compliance with DD ENV 1993-1-2:2001 does not of itself confer immunity from legal obligations.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted into an EN.

Comments should be sent in writing to BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, proposed revised wording.

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

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xi, the ENV title page, pages 2 to 64 and a back cover.

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1) Available from The Stationery Office, PO Box 29, St Crispins House, Duke Street, Norwich NR3 1GN.

National Application Document

for use in the UK with ENV 1993-1-2:1995

Contents of National Application Document

Introduction

This National Application Document (NAD) has been prepared by Subcommittee B/525/31. It has been developed from:

a) a textual examination of ENV 1993-1-2:1995;

b) a parametric calibration against BS 5950-8, supporting standards and test data;

c) trial calculations.

It should be noted that this NAD, in common with ENV 1993-1-2 and supporting CEN standards, uses a comma (,) where a decimal point (.) would be traditionally used in the UK.

1 Scope

This NAD provides information required to enable ENV 1993-1-2:1995 *Eurocode 3 — Design of steel structures — Part 1.2: General rules — Structural fire design*, to be used for the fire resistant design of buildings to be constructed in the UK. ENV 1993-1-2:1995 is intended to be used in conjunction with DD ENV 1991-2-2:1996 and DD ENV 1993-1-1:1992, which refer to British Standards for the values of mechanical and thermal loads (actions).

2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this NAD. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

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

BS 476-21:1987, *Fire tests on building materials and structures — Part 21: Methods for determination of the fire resistance of loadbearing elements of construction.*

BS 5555:1993, *Specification for SI units and recommendations for the use of their multiples and certain other units.*

DD ENV 1991-2-2:1996, *Basis of design and actions on structures — Part 2-2: Actions on structures exposed to fire (together with United Kingdom National Application Document).*

DD ENV 1993-1-1:1992, *Eurocode 3: Design of steel structures — Part 1-1: General rules and rules for buildings (together with United Kingdom National Application Document).*

DD ENV 1993-1-3, *Eurocode 3: Design of steel structures — Part 1-3: General rules — Supplementary rules for cold formed thin gauge members and sheeting (together with United Kingdom National Application Document).*

DD ENV 1994-1-2, *Eurocode 4: Design of composite steel and concrete structures — Part 1-2: General rules — Structural fire design (together with United Kingdom National Application Document).*

3 Mechanical loading, partial factors, combination factors and other values

a) Pending the issue of EN 1993-1-1, the mechanical actions, partial factors, combination factors and other values should be determined from clauses **3** and **4** of the NAD for ENV 1993-1-1:1992.

b) The partial factors for the fire situation should be taken from Table 1 and the combination factors should be taken from Table 2.

4 Thermal loading, partial factors, combination factors and other values

a) The thermal actions should be obtained from the NAD of ENV 1991-2-2.

b) The value of $h_{net,d}$ referred to in ENV 1993-1-2:1995, **4.2.5.1**(2) should be obtained from ENV 1991-2-2, **4.2.1** making the following modifications:

1) a value of 0,45 should be used in ENV 1991-2-2:1996, **4.2.1**(2) for the factor $\gamma_{n,r}$ in accordance with the NAD of DD ENV 1991-2-2;

2) a value of 0,8 should be used in ENV 1991-2-2:1996, **4.2.1**(3) for the emissivity of steel ε_m, giving a value of 0,64 for ε_{res}.

c) The values for the adaptation factors κ_1 and κ_2 should be taken from Table 3

Table 1 — Partial factors (γ factors) for the fire situation

Reference in ENV 1993-1-2	Description	Symbol	Condition	Value	
				Boxed ENV value	Value for UK use
2.3(2)P	Partial factor for the fire situation	$\gamma_{M,fi}$	For thermal properties	1,0	1,0
2.3(3)P	Partial factor for the fire situation	$\gamma_{M,fi}$	For mechanical properties	1,0	1,0
2.4.3(4)	Partial factor for permanent action in accidental design situation	γ_{GA}	Fire	1,0	1,0[a]
[a] This value of γ_{GA} is different from the value in the NAD of DD ENV 1993-1-1:1992.					

5 Reference standards

Where ENs are referred to, appropriate BS ENs should be used. The remaining supporting standards which should be used are listed in Table 4.

Table 2 — Combination factors for the fire situation

Action	$\Psi_{1,1}$
Imposed floor loads in buildings: — storage; — escape stairs and lobbies; — all other areas.	 0,9 0,9 0,7
Imposed roof loads (including snow loads)	0
Wind loads: — height to eaves up to 8 m; — height to eaves greater than 8 m.	 0 0,3
NOTE Plant load should be taken as a permanent load.	

6 Additional recommendations

Table 3 — Adaptation factors

Reference in ENV 1993-1-2	Description	Symbol	Condition	Value	
				ENV value	Value for UK use
4.2.3.3(8)	The adaptation factor for non-uniform temperature distribution across a cross-section	κ_1	For a beam exposed on all four sides	1,0	1,0
4.2.3.3(8)	The adaptation factor for non-uniform temperature distribution across a cross-section	κ_1	For a beam exposed on three sides with a composite or concrete slab on side four	0,7	0,7
4.2.3.3(9)	The adaptation factor for non-uniform temperature distribution along a beam	κ_2	At the supports of a statically indeterminate beam	0,85	0,85
			In all other cases	1,0	1,0

Table 4 — Directly referenced supporting standards

Reference in ENV 1993-1-2	UK supporting standards
PrEN ISO 834	BS 476-20:1987
PrENV yyy 5-1	BS 476-20:1987 BS 476-21:1987[a]
PrENV yyy 5-2	
PrENV yyy 5-3	
ENV 1991-2-2	DD ENV 1991-2-2
ENV 1993-1-1	DD ENV 1993-1-1
ENV 1993-1-3	DD ENV 1993-1-3
ENV 1994-1-2	DD ENV 1994-1-2
ISO 1000	BS 5555:1993

[a] Further information can be obtained from *Fire protection for structural steel in buildings* [1].

6.1 Chapter 1 General

a) *1.1 Scope*

ENV 1993-1-2:1995 may be used to determine the resistance of stainless steel members subjected to the accidental situation of exposure to fire.

NOTE The performance of stainless steel in fire is usually significantly better than that of mild steel.

6.2 Chapter 2 Basic Principles and Rules

a) *2.1(2)P*

Provided fire protection materials satisfy the recommendations given in ENV 1993-1-2:1995, **3.3.2** the reduction factor k_y given in ENV 1993-1-2:1995, Table 3.1 may be used to calculate the member resistance. In other cases, the reduction factor k_x in ENV 1993-1-2:1995, Table 3.1 should be used.

NOTE Information on fire protection materials that satisfy the recommendations in ENV 1993-1-2:1995, **3.3.2** can be found in the publication *Fire protection for structural steel in buildings* [1].

b) *2.4.4(4)*

In general, the effects of thermal expansion may be neglected. However, consideration should be given to the effect of thermal expansion on bracing members. Further guidance is given in **6.4**b).

6.3 Chapter 4 Structural Fire Design

a) *4.2.1(5)P*

Alternatively, for the design of bolts and welds in the fire situation, the strength reduction factor may conservatively be taken as 80 % of the temperature-dependent value of $k_{x,\theta}$ given in ENV 1993-1-2:1995, Table 3.1.

b) *4.2.2(4)*

Contrary to ENV 1993-1-2:1995, **4.2.2**(4) in a fire design situation all members should be classified as for normal temperature design using the following expression for ε:

$$\varepsilon = [(235/f_y)]^{0,5}$$

where

f_y is the steel yield strength given in ENV 1993-1-1:1992, **3.2.2.1**.

c) *4.2.3.1*

For tension members, $k_{y,\theta}$ should be replaced in the expression by the temperature-dependent value of $k_{x,\theta}$ given in Table 3.1 of ENV 1993-1-2:1995.

d) *4.2.3.2(2)*

As an alternative to expression (4.6) in ENV 1993-1-2:1995, $\bar{\lambda}_{\theta,\max}$ may be conservatively calculated as:

$$\bar{\lambda}_{\theta,\max} = 1.2\ \bar{\lambda}$$

where

$\bar{\lambda}$ is the relative slenderness for a normal temperature design.

e) *4.2.3.2(4)*

For a column in a steel frame in which each storey comprises a separate fire compartment with sufficient fire resistance, the buckling length ℓ_{fi} should be taken as $0{,}7L$ where the column is continuous at both ends. Where the column is continuous at one end only, the buckling length should be taken as $0{,}85L$. ENV 1993-1-2:1995, Figure 4.1 as modified by **6.3**f) gives the buckling lengths of columns in braced frames.

f) *Figure 4.1 Buckling lengths ℓ_{fi} of columns in braced frames*

The buckling length $\ell_{fi,2}$ of the column in the intermediate storey should be taken as $0{,}7L_2$ and the buckling length $\ell_{fi,4}$ of the top storey column should be taken as $0{,}85L_4$.

g) *4.2.3.3(5)*

The constant 1,2 in expression (**4.11**) of ENV 1993-1-2:1995 should be replaced by the parameter K whose value is given by the following expression:

$$K = \frac{\lambda_{L,T,\theta,com}}{3} + 0{,}87$$

where

$\lambda_{L,T,\theta,com}$ is given in ENV 1993-1-2:1995, **4.2.3.3**(6) as modified by **6.3**h).

h) *4.2.3.3(6)*

Contrary to ENV 1993-1-2:1995, **4.2.3.3**(6) the value of the non-dimensional slenderness $\bar{\lambda}_{L,T,\theta,com}$ for the temperature $\theta_{a,com}$ should be taken as the value of the normal temperature non-dimensional slenderness $\bar{\lambda}_{LT}$.

i) *4.2.4*

The critical temperature method is an alternative to that given in ENV 1993-1-2:1995, **4.2.3** but cannot be used for the following cases:

— columns;

— tension members;

— unrestrained beams with $\bar{\lambda}_{L,T,fi} > 0{,}4$.

It can only be used for fully restrained beams, unrestrained beams where $\bar{\lambda}_{L,T,fi}$ is not greater than 0,4 and members with Class 4 cross-sections.

ENV 1993-1-2 uses the term "degree of utilization", μ_0, which allows the applied load on a member to be directly related to the critical temperature and is determined from the following expression:

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}}$$

The variables in the above expression are as defined in ENV 1993-1-2:1995, **4.2.3.5**.

For a beam bending with a uniform temperature distribution over its cross-section and along its length, $R_{fi,d,0}$ is the design moment of resistance of the member at ambient temperature.

For a beam bending with a non-uniform temperature distribution over its cross-section and along its length, $R_{fi,d,0}$ is determined by dividing the member's design moment of resistance at ambient temperature by the adaptation factors κ_1 and κ_2.

j) *4.2.4(4)*

When calculating the degree of utilization, the value of $R_{fi,d,0}$ is determined by dividing the member resistance at normal temperature by both adaptation factors κ_1 and κ_2.

k) *4.2.4(6)*

The method described in this clause may be used to design members constructed from Class 4 open sections.

6.4 Recommendations on subjects not covered in ENV 1993-1-2:1995

a) *Re-use of steel after a fire*

It may be possible to re-use steel after a fire. The guidance in BS 5950-8:1990, appendix C should be followed.

b) *Bracing members*

Bracing members provided stability to the structure in the fire design situation and should be distributed throughout the building so no substantial portion of the structural frame is solely reliant on a single plane of bracing in each of two directions approximately at right angles. Where the stability of the structure is solely dependent on a single plane of bracing or where the bracing systems are located adjacent to a single fire compartment, the temperature in the columns and compressive members forming part of the bracing system should not exceed 450 °C.

c) *Water-filled structures*

The design of water-filled structures should follow the guidance in BS 5950-8:1990, **4.7**.

NOTE Further information may be found in *Fire and steel construction — Water cooled hollow columns* [2].

d) *Portal frames*

The design of portal frames in the fire situation should follow the guidance in BS 5950-8:1990, **4.5**.

NOTE Further information may be found in *Fire and steel construction: The behaviour of steel portal frames in boundary conditions* [3].

e) *Beams with shelf angles*

The fire resistance for beams with shelf angles should be determined in accordance with the guidance in BS 5950-8:1990, appendix E.

f) *Fire resisting walls*

The guidance in BS 5950-8:1990, **4.10** should be followed for the design of fire resisting walls in the fire situation.

g) *Roofs*

Where a roof spans across a fire resisting compartment wall where it is required that strips of the roof should be fire protected on the underside of both sides of the compartment wall, care should be taken to fire stop any gaps between the top of the wall and the underside of the roof cladding to accommodate differential thermal movement in fire. Where practicable, combustible insulation should also be fire stopped along the line of the wall.

h) *Ceilings*

In addition to ENV 1993-1-2:1995, **4.2.5.3** the guidance in BS 5950-8:1990, **4.12** should also be followed for the design of ceilings in the fire situation

Bibliography

BS 5950-8:1990, *Structural use of steelwork in building — Code of practice for fire resistant design.*

[1] Association of Specialist Fire Protection Contractors and Manufacturers Limited, Steel Construction Institute. *Fire protection for structural steel in buildings* (Revised Second Edition), 1992[2)].

[2] BOND, G.V.L. *Fire and steel construction — Water cooled hollow columns*. Steel Construction Institute, 1975[2)].

[3] NEWMAN, G.M. *Fire and steel construction — The behaviour of steel portal frames in boundary conditions* (Second Edition), Steel Construction Institute, 1990[2)].
ISBN 1 870004 49 3

[2)] Available from The Steel Construction Institute, Silwood Park, Ascot, Berkshire SL5 7QN.

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EUROPEAN PRESTANDARD

PRÉNORME EUROPÉENNE

EUROPÄISCHE VORNORM

ENV 1993-1-2

September 1995

ICS 13.220.20; 91.040.00; 91.080.10

Descriptors: buildings, steel construction, structural steels, design , safety requirements, accident prevention, fire protection, fire resistance, mechanical properties, thermodynamic properties, computation, mechanical strength

English version

Eurocode 3 - Design of steel structures - Part 1-2: General rules - Structural fire design

Eurocode 3 - Calcul des structures en acier - Partie 1-2: Règles générales - Calcul du comportement au feu

Bemessung und Konstruktion von Stahlbauten - Teil 1-2: Allgemeine Regeln - Tragwerksbemessung für den Brandfall

This European Prestandard (ENV) was approved by CEN on 1993-11-05 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existance of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart,36 B-1050 Brussels

Ref. No. ENV 1993-1-2:1995 E

Contents

Foreword

Objectives of the Eurocodes

(1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship, needed to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for methods of testing their performance is available, some of the Structural Eurocodes cover some of these aspects in informative annexes.

Background to the Eurocode programme

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various member states and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(5) In 1990, after consulting their respective member states, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC 250 is responsible for all Structural Eurocodes.

Eurocode programme

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991	Eurocode 1	Basis of design and actions on structures;
EN 1992	Eurocode 2	Design of concrete structures;
EN 1993	Eurocode 3	Design of steel structures;
EN 1994	Eurocode 4	Design of composite steel and concrete structures;
EN 1995	Eurocode 5	Design of timber structures;
EN 1996	Eurocode 6	Design of masonry structures;
EN 1997	Eurocode 7	Geotechnical design;
EN 1998	Eurocode 8	Design provisions for earthquake resistance of structures;
EN 1999	Eurocode 9	Design of aluminium alloy structures.

(8) Separate sub-committees have been formed by CEN/TC 250 for the various Eurocodes listed above.

(9) This Part 1.2 of Eurocode 3 is published by CEN as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the secretariat of CEN/TC 250/SC 3 at the following address:

BSI Standards
British Standards House
389 Chiswick High Road
London W4 4AL
England

or to your national standards organization.

National Application Documents (NAD's)

(13) In view of the responsibilities of the authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by $\boxed{}$ ("boxed values"). The authorities in each member country are expected to review the "boxed values" and may substitute alternative definitive values for these safety elements for use in national application.

(14) Some of the supporting European or International Standards might not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

(15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works is located.

Matters specific to this Prestandard

(16) Work on those parts of the Structural Eurocodes covering fire resistance was initiated by the CEC and a first draft of this document was issued in 1990 as a draft "Eurocode 3 : Part 10".

(17) With the transfer of work on Structural Eurocodes to CEN, the responsibility for completing this document passed to CEN Technical Committee CEN/TC 250, sub-committee CEN/TC 250/SC 3.

(18) The scope of Eurocode 3 is defined in 1.1.1 of ENV 1993-1-1. Additional Parts of Eurocode 3 that are planned are indicated in 1.1.3 of ENV 1993-1-1.

(19) The general objectives of fire protection are to limit risks with respect to the individual and society, neighbouring property and, where required, directly exposed property, in the case of fire.

(20) The Structural Eurocodes deal with specific aspects of passive fire protection in terms of designing structures and parts thereof for adequate load-bearing capacity and for limiting fire spread as relevant.

(21) Required functions and levels of performance are generally specified by national authorities - mostly in terms of standard fire resistance rating. Where fire safety engineering for assessing passive and active measures is accepted, requirements by authorities may be less prescriptive and allow alternative strategies.

(22) This Part 1.2, together with ENV 1991-2-2, gives the supplements to ENV 1993-1-1 that are necessary so that structures designed according to this set of Structural Eurocodes may also comply with structural fire resistance requirements.

(23) Supplementary requirements concerning, for example:

- the possible installation and maintenance of sprinkler systems;
- conditions of occupancy of the building or fire compartment;
- the use of approved insulation and coating materials, including their maintenance;

are not given in this document, because they are subject to specification by national authorities.

(24) A method is included in this ENV for applying deformation criteria to the load-bearing structure where the means of protection, or the design criteria for separating members, require such consideration (see 2.1(2), 3.2.1(6), 4.2.1(6) and 4.2.2(5)). However no specific provisions are given for its application. It is intended that where any such provision is considered necessary, it should be included in the NAD.

(25) A full analytical procedure for structural fire design would take into account the behaviour of the structural system at elevated temperatures, the potential heat exposure and the beneficial effects of active fire protection systems, together with the uncertainties associated with these three features and the importance of the structure (consequences of failure).

(26) At the present time it is possible to undertake a procedure for determining adequate performance that incorporates some, if not all, of these parameters and to demonstrate that the structure, or its components, will give adequate performance in a real building fire. However, the principal current procedure in European countries is one based on results from standard fire resistance tests. The grading systems in national regulations that call for specific periods of fire resistance, take into account (though not explicitly) the features and uncertainties described above.

(27) Due to the limitations of the test method, further tests or analyses may be used. Nevertheless, the results of standard fire tests form the bulk of the input to calculation methods for structural fire design. This Prestandard therefore deals in the main with design for the standard fire resistance.

(28) Simple calculation models for steel structures are given in this document and accordingly tabulated data are not included. It is expected that tables and other design aids based on the calculation methods given in this ENV 1993-1-2 will be prepared by interested external organisations.

1 General

1.1 Scope

(1)P This Part 1.2 of ENV 1993 deals with the design of steel structures for the accidental situation of fire exposure and is intended to be used in conjunction with ENV 1993-1-1 and ENV 1991-2-2. This Part 1.2 only identifies differences from, or supplements to, normal temperature design.

(2)P This document deals only with passive methods of fire protection. Active methods are not covered.

(3)P This Part 1.2 applies to structures that, for reasons of general fire safety, are required to avoid premature collapse of the structure in exposure to fire (load-bearing function).

(4)P This Part 1.2 gives principles and application rules (see 1.2) for designing structures for specified requirements in respect of the aforementioned function and the levels of performance.

(5)P This document only applies to structures, or parts of structures, that are within the scope of ENV 1993-1-1 and are designed accordingly.

(6)P The methods given in this document may also be applied to cold-formed thin gauge steel members and sheeting within the scope of ENV 1993-1-3.

(7)P For the fire resistance of composite steel and concrete structures see ENV 1994-1-2.

(8)P The methods given in this document are applicable to any steel grade for which material properties are available.

(10)P The steel properties given in this document apply to steel grades S 235, S 275 and S 355 of EN 10025 and to all steel grades of EN 10113, EN 10155, EN 10210-1 and EN 10219-1.

1.2 Distinction between principles and application rules

(1)P Depending on the character of the individual clauses, distinction is made in this Part between principles and application rules.

(2)P The principles comprise:

- general statements and definitions for which there is no alternative;
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The principles are identified by the letter P following the paragraph number.

(4)P The application rules are generally recognised rules which follow the principles and satisfy their requirements. It is permissible to use alternative design rules different from the application rules given in the Eurocode, provided that it is shown that the alternative rule accords with the relevant principles and have at least the same reliability.

(5) In this Part the application rules are identified by a number in brackets, as in this paragraph.

1.3 Normative references

This European Prestandard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Prestandard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

EN 10025 *Hot rolled products of non-alloy structural steels: Technical delivery conditions;*

EN 10113 *Hot rolled products in weldable fine grade structural steels:*

Part 1: *General delivery conditions*;

Part 2: *Delivery conditions for normalized/normalized rolled steels*;

Part 3: *Delivery conditions for thermomechanically rolled steels*;

EN 10155 *Structural steels with improved atmospheric corrosion resistance - Technical delivery conditions*;

EN 10210 *Hot finished structural hollow sections of non-alloy and fine grain structural steels:*

Part 1: *Technical delivery conditions*;

EN 10219 *Cold formed welded structural hollow sections of non-alloy and fine grain structural steels:*

Part 1: *Technical delivery conditions*;

pr EN ISO 834 *Fire resistance: General requirements*;

pr ENV yyy5 *Fire tests on elements of building construction:*

Part 1: *Test method for determining the contribution to the fire resistance of structural members: by horizontal protective membranes*;

Part 2 *Test method for determining the contribution to the fire resistance of structural members: by vertical protective membranes*;

Part 4: *Test method for determining the contribution to the fire resistance of structural members: by applied protection to steel structural elements*;

ENV 1991 *Eurocode 1. Basis of design and actions on structures:*

Part 2.2: *Actions on structures exposed to fire*;

ENV 1993 *Eurocode 3. Design of steel structures:*

Part 1.1: *General rules : General rules and rules for buildings*;

Part 1.3: *General rules : Supplementary rules for cold formed thin gauge steel members and sheeting*;

ENV 1994 *Eurocode 4. Design of composite steel and concrete structures:*

Part 1.2: *General rules : Structural fire design*;

ISO 1000 *SI units.*

1.4 Definitions

For the purposes of this Part 1.2 of ENV 1993, the following definitions apply:

1.4.1 configuration factor: Solid angle within which the radiating environment can be seen from a particular point on the member surface, divided by 2π.

1.4.2 convective heat transfer coefficient: 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 surface.

1.4.3 critical temperature of structural steel: For a given load level, the temperature at which failure is expected to occur in a structural steel element for a uniform temperature distribution.

1.4.4 design fire: A specified fire development assumed for design purposes.

1.4.5 effective yield strength: For a given temperature, the stress level at which the stress-strain relationship of steel is truncated to provide a yield plateau.

1.4.6 external member: Structural member located outside the building, that may be exposed to fire through openings in the building enclosure.

1.4.7 fire compartment: A space within a building, extending over one or several floors, that is enclosed by separating members such that fire spread beyond the compartment is prevented during the relevant fire exposure.

1.4.8 fire protection material: A material that has been shown, by fire resistance tests, to be capable of remaining in position and of providing adequate thermal insulation for the fire resistance period under consideration.

1.4.9 fire resistance: The ability of a structure, a part of a structure or a member to fulfil its required functions (load bearing function and/or separating function) for a specified fire exposure and for a specified period of time.

NOTE: For steel members only the load bearing function applies.

1.4.10 fire wall: A wall separating two spaces (generally two buildings) that is designed for fire resistance and structural stability, including resistance to horizontal loading such that, in case of fire and failure of the structure on one side of the wall, fire spread beyond the wall is avoided.

1.4.11 global structural analysis (for fire): An analysis of the entire structure, when either the entire structure, or only parts of it, are exposed to fire. Indirect fire actions are considered throughout the structure.

1.4.12 indirect fire actions: Thermal expansions, thermal deformations or thermal gradients causing internal forces and moments.

1.4.13 load bearing criterion: A criterion by which the ability of a structure or member to sustain specified actions, during the relevant fire, is assessed.

1.4.14 load bearing function: The ability of a structure or a member to sustain specified actions during the relevant fire, according to defined criteria.

1.4.15 member analysis (for fire): The thermal and mechanical analysis of a structural member exposed to fire in which the member is considered as isolated, with appropriate support and boundary conditions. Indirect fire actions are not considered, except those resulting from thermal gradients.

1.4.16 net heat flux: Energy per unit time and surface area absorbed by members.

1.4.17 normal temperature design: Ultimate limit state design for ambient temperatures according to ENV 1993-1-1 for the fundamental combination according to ENV 1991-1.

1.4.18 protected members: Members for which measures are taken to reduce the temperature rise in the member due to fire.

1.4.19 section factor: 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.

1.4.20 separating member: Structural or non-structural member (wall or floor) forming part of the enclosure of a fire compartment.

1.4.21 standard fire exposure: Exposure to furnace gases with a temperature that varies with time according to the standard temperature-time curve.

1.4.22 standard fire resistance: The ability of a structure or part of it (usually only members) to fulfil required functions (load-bearing function and/or separating function), for the standard fire exposure for a stated period of time.

> **NOTE:** Standard fire resistance requirements are normally expressed in terms of periods of time, such as 30, 60 or more minutes.

1.4.23 standard temperature-time curve: The nominal temperature-time curve given in ENV 1991-2-2.

1.4.24 structural members: The load-bearing members of a structure, including bracings.

1.4.25 sub-assembly analysis (for fire): The structural analysis of parts of the structure exposed to fire, in which the respective part of the structure is considered as isolated, with appropriate support and boundary conditions. Indirect fire actions within the sub-assembly are considered, but no time-dependent interaction with other parts of the structure.

> **NOTE 1:** Where the effects of indirect fire actions within the sub-assembly are negligible, sub-assembly analysis is equivalent to member analysis.

> **NOTE 2:** Where the effects of indirect fire actions between sub-assemblies are negligible, sub-assembly analysis is equivalent to global structural analysis.

1.4.26 support and boundary conditions: Restraint conditions and applied forces and moments assumed at the supports and boundaries of a structure or part of a structure for the purposes of structural analysis.

1.4.27 temperature analysis: The 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.

1.4.28 temperature-time curves: Gas temperatures in the environment of member surfaces as a function of time. They may be:

- **nominal**: Conventional curves, adopted for classification or verification of fire resistance, such as the standard time-temperature curve;
- **parametric**: Determined on the basis of fire models and the specific physical parameters defining the conditions in the fire compartment.

1.4.29 thermal actions: Actions on the structure described by the net heat flux to the members.

1.5 Symbols

1.5.1 Supplementary to ENV 1993-1-1, the following symbols are used:

A_m is the surface area of a member per unit length;

A_p is the area of the inner surface of the fire protection material per unit length of the member;

E_a is the modulus of elasticity of steel for normal temperature design;

$E_{a,\theta}$ is the slope of the linear elastic range for steel at elevated temperature θ_a;

$E_{fi,d}$ is the design effect of actions in the fire situation;

$R_{d,\theta}$ is the design resistance at uniform elevated material temperature θ;

$R_{fi,d}$ is the design resistance in the fire situation;

$R_{fi,d,t}$ is the design value of a resistance in the fire situation, at time t;

T is the temperature [K] (cf θ temperature [°C]);

V is the volume of a member per unit length;

$X_{fi,d}$ is the design material property in the fire situation;

X_k is the characteristic value of a material property;

$X_{k,\theta}$ is the characteristic value of a material property at elevated temperature θ;

c is the specific heat [J/kgK];

d_p is the thickness of fire protection material;

$f_{p,\theta}$ is the proportional limit for steel at elevated temperature θ_a;

$f_{y,\theta}$ is the effective yield strength of steel at elevated temperature θ_a;

$\dot{h}_{net,d}$ is the design value of the net heat flux per unit area;

k_θ is the relative value of a strength or deformation property of steel at elevated temperature θ_a;

ℓ is the length at 20 °C;

$\Delta\ell$ is the temperature induced expansion;

t is the time in fire exposure [minutes];

Δt is the time interval [seconds];

η_{fi} is the reduction factor for design load level in the fire situation;

θ is the temperature [°C] (cf T temperature [K]);

κ is the adaptation factor;

λ is the thermal conductivity [W/mK];

μ_0 is the degree of utilisation at time $t = 0$.

1.5.2 Supplementary to ENV 1993-1-1, the following subscripts are used:

a	steel;
c	connection;
fi	value relevant for the fire situation;
m	member;
p	fire protection material;
t	dependent on time;
θ	dependent on temperature.

1.5.3 Additional symbols are used in annexes C and D. These are defined where they first occur.

1.6 Units

(1)P SI units shall be used in conformity with ISO 1000.

(2) Supplementary to ENV 1993-1-1, the following units are recommended for use in calculations:

- area	:	m^2 ;
- insulation thickness	:	m;
- temperature	:	°C;
- absolute temperature	:	K;
- temperature difference	:	K;
- specific heat	:	J/kgK;
- coefficient of thermal conductivity	:	W/mK.

2 Basic principles and rules

2.1 Performance requirements

(1)P Where mechanical resistance in the case of fire is required, steel structures shall be designed and constructed in such a way that they maintain their load bearing function during the relevant fire exposure.

(2)P Deformation criteria shall be applied where the means of protection, or the design criteria for separating members, require consideration of the deformation of the load bearing structure.

2.2 Actions

(1)P The thermal and mechanical actions shall be obtained from ENV 1991-2-2.

(2) Where rules given in this Part 1.2 of ENV 1993 are valid only for the standard fire exposure, this is identified in the relevant clauses.

2.3 Design values of material properties

(1)P Design values of thermal and mechanical material properties $X_{fi,d}$ are defined as follows:

- thermal properties for thermal analysis:
  - if an increase of the property is favourable for safety:

$$X_{fi,d} = X_{k,\theta} / \gamma_{M,fi} \tag{2.1a}$$

  - if an increase of the property is unfavourable for safety:

$$X_{fi,d} = \gamma_{M,fi} X_{k,\theta} \tag{2.1b}$$

- strength and deformation properties for structural analysis:

$$X_{fi,d} = k_\theta X_k / \gamma_{M,fi} \tag{2.1c}$$

where:

$X_{k,\theta}$ is the characteristic value of a material property in fire design, generally dependent on the material temperature, see section 3;

X_k is the characteristic value of a strength or deformation property (generally f_k or E_k) for normal temperature design to ENV 1993-1-1;

k_θ is the reduction factor for a strength or deformation property $(X_{k,\theta}/X_k)$, dependent on the material temperature, see 3.2.1;

$\gamma_{M,fi}$ is the partial safety factor for the relevant material property, for the fire situation.

(2)P For thermal properties of steel, the partial safety factor for the fire situation shall be taken as:

$$\gamma_{M,fi} = \boxed{1,0}$$

(3)P For mechanical properties of steel, the partial safety factor for the fire situation shall be taken as:

$$\gamma_{M,fi} = \boxed{1,0}$$

2.4 Assessment methods

2.4.1 General

(1)P The model of the structural system adopted for design to this Part 1.2 of ENV 1993 shall reflect the expected performance of the structure in fire exposure.

(2)P The structural analysis for the fire situation may be carried out using one of the following:

- global structural analysis, see 2.4.2;
- analysis of portions of the structure, see 2.4.3;
- member analysis, see 2.4.4.

(3) For verifying standard fire resistance requirements, a member analysis is sufficient.

(4)P As an alternative to the use of calculation models, design may be based on the results of tests.

2.4.2 Global structural analysis

(1)P Global structural analysis for the fire situation shall be carried out, taking into account the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffnesses.

(2)P It shall be verified that, for the relevant duration of fire exposure t :

$$E_{\mathrm{fi,d}} \leq R_{\mathrm{fi,d,t}} \qquad (2.2)$$

where:

$E_{\mathrm{fi,d}}$ is the design effect of actions for the fire situation, determined in accordance with ENV 1991-2-2, including the effects of thermal expansions and deformations;

$R_{\mathrm{fi,d,t}}$ is the corresponding design resistance at elevated temperatures.

2.4.3 Analysis of portions of the structure

(1)P As an alternative to global structural analysis of the entire structure for various fire situations, structural analysis of subassemblies comprising appropriate portions of the structure may be carried out in accordance with 2.4.2.

(2) The reactions at supports and internal forces and moments at boundaries of subassemblies applicable at time $t = 0$ may be assumed to remain unchanged throughout the fire exposure.

(3) As an alternative to carrying out a global structural analysis for the fire situation at time $t = 0$, the reactions at supports and internal forces and moments at boundaries of subassemblies may be obtained from a global structural analysis for normal temperature design by using:

$$E_{\mathrm{fi,d}} = \eta_{\mathrm{fi}} E_{\mathrm{d}} \qquad (2.3)$$

where:

E_{d} is the design value of the corresponding force or moment for normal temperature design, for the fundamental combination of actions given by expression (2.9) in ENV 1993-1-1;

η_{fi} is the reduction factor for the design load level for the fire situation.

(4) The reduction factor for the design load level for the fire situation η_{fi} is given by:

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{2.4}$$

where:

$Q_{k,1}$ is the principal variable load;

γ_{GA} is the partial factor for permanent actions in accidental design situations;

$\psi_{1,1}$ is the combination factor for frequent values, see table 9.3 in ENV 1991-1.

NOTE: Figure 2.1 shows the variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$ for different values of the factor $\psi_{1,1}$ for γ_{GA} = 1,0 with γ_G = 1,35 and γ_Q = 1,5.

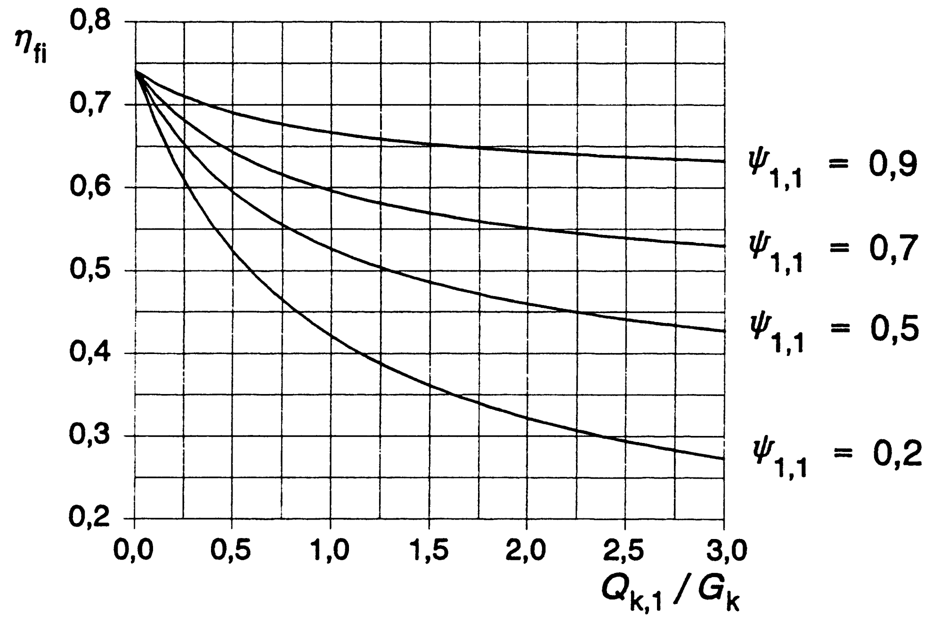


Figure 2.1: Variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$

2.4.4 Member analysis

(1)P As an alternative to global structural analysis, individual members may be analysed for the fire situation. The restraint conditions at supports and ends of members applicable at time t = 0 may generally be assumed to remain unchanged throughout the fire exposure. Where different conditions apply, this is identified in the relevant provisions.

(2) The internal forces and moments at supports and ends of members applicable at time t = 0 may be assumed to remain unchanged throughout the fire exposure.

(3) As an alternative to carrying out a global structural analysis for the fire situation at time t = 0, the internal forces and moments at supports and ends of members may be obtained from a global structural analysis for normal temperature design by using expression (2.3).

(4) Only the effects of thermal deformations resulting from thermal gradients across the cross-section need be considered. The effects of thermal expansions of the members may be neglected.

3 Material properties

3.1 General

(1)P The thermal and mechanical properties of steel shall be determined from the following. For materials not included herein, reference shall be made to the relevant CEN product standard or European Technical Approval (ETA).

(2)P The values of material properties given in section 3 shall be treated as characteristic values, see 2.3(1).

(3)P The mechanical properties of steel at 20 °C shall be taken as those given in ENV 1993-1-1 for normal temperature design.

3.2 Mechanical properties of steel

3.2.1 Strength and deformation properties

(1)P For heating rates between 2 and 50 K/min, the strength and deformation properties of steel at elevated temperatures shall be obtained from the stress-strain relationship given in figure 3.1.

(2) This relationship should be used to determine resistance to tension, compression, moment or shear.

(3) Table 3.1 gives the reduction factors, relative to the appropriate value at 20 °C, for the stress-strain relationship for steel at elevated temperatures given in figure 3.1, as follows:

- effective yield strength, relative to yield strength at 20 °C: $k_{y,\theta} = f_{y,\theta}/f_y$
- proportional limit, relative to yield strength at 20 °C: $k_{p,\theta} = f_{p,\theta}/f_y$
- slope of linear elastic range, relative to slope at 20 °C: $k_{E,\theta} = E_{a,\theta}/E_a$

(4) The variation of these three reduction factors with temperature is illustrated in figure 3.2.

(5)P Alternatively, for temperatures below 400 °C, the stress-strain relationship specified in (1) may be extended by the strain-hardening option given in annex B, provided that the proportions of the cross-section are not such that local buckling is liable to prevent attainment of the increased strain and that the member is adequately restrained to prevent buckling.

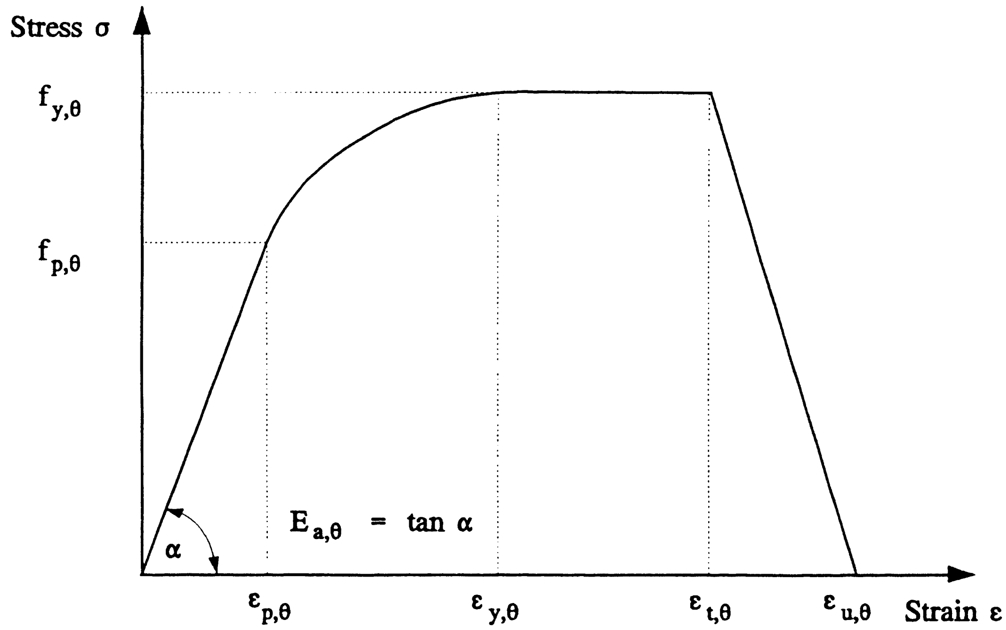
(6) Table 3.1 also gives values of a modified reduction factor $k_{x,\theta}$ for use in place of $k_{y,\theta}$ where it is necessary to satisfy deformation criteria.

3.2.2 Unit mass

(1)P The unit mass of steel ρ_a may be considered to be independent of the steel temperature. The following value may be taken:

$$\rho_a = 7850\,\mathrm{kg/m^3}$$

Strain range	Stress σ	Tangent modulus
$\varepsilon \leq \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$	$E_{a,\theta}$
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + (b/a)\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}$	$\dfrac{b(\varepsilon_{y,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}}$
$\varepsilon_{y,\theta} \leq \varepsilon \leq \varepsilon_{t,\theta}$	$f_{y,\theta}$	0
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta}\left[1 - (\varepsilon - \varepsilon_{t,\theta})/(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})\right]$	-
$\varepsilon = \varepsilon_{u,\theta}$	0,00	-
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta}/E_{a,\theta}$ $\quad\varepsilon_{y,\theta} = 0,02$	$\varepsilon_{t,\theta} = 0,15$ $\quad\varepsilon_{u,\theta} = 0,20$
Functions	$a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c/E_{a,\theta})$ $b^2 = c(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2$ $c = \dfrac{(f_{y,\theta} - f_{p,\theta})^2}{(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})}$	



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

Figure 3.1: Stress-strain relationship for steel at elevated temperatures.

Table 3.1: Reduction factors for stress-strain relationship of steel at elevated temperatures.

Steel temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20 °C			
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Modified factor (relative to f_y) for satisfying deformation criteria $k_{x,\theta} = f_{x,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20 °C	1,000	1,000	1,000	1,000
100 °C	1,000	1,000	1,000	1,000
200 °C	1,000	0,922	0,807	0,900
300 °C	1,000	0,845	0,613	0,800
400 °C	1,000	0,770	0,420	0,700
500 °C	0,780	0,615	0,360	0,600
600 °C	0,470	0,354	0,180	0,310
700 °C	0,230	0,167	0,075	0,130
800 °C	0,110	0,087	0,050	0,090
900 °C	0,060	0,051	0,0375	0,0675
1000 °C	0,040	0,034	0,0250	0,0450
1100 °C	0,020	0,017	0,0125	0,0225
1200 °C	0,000	0,000	0,0000	0,0000

NOTE: For intermediate values of the steel temperature, linear interpolation may be used.

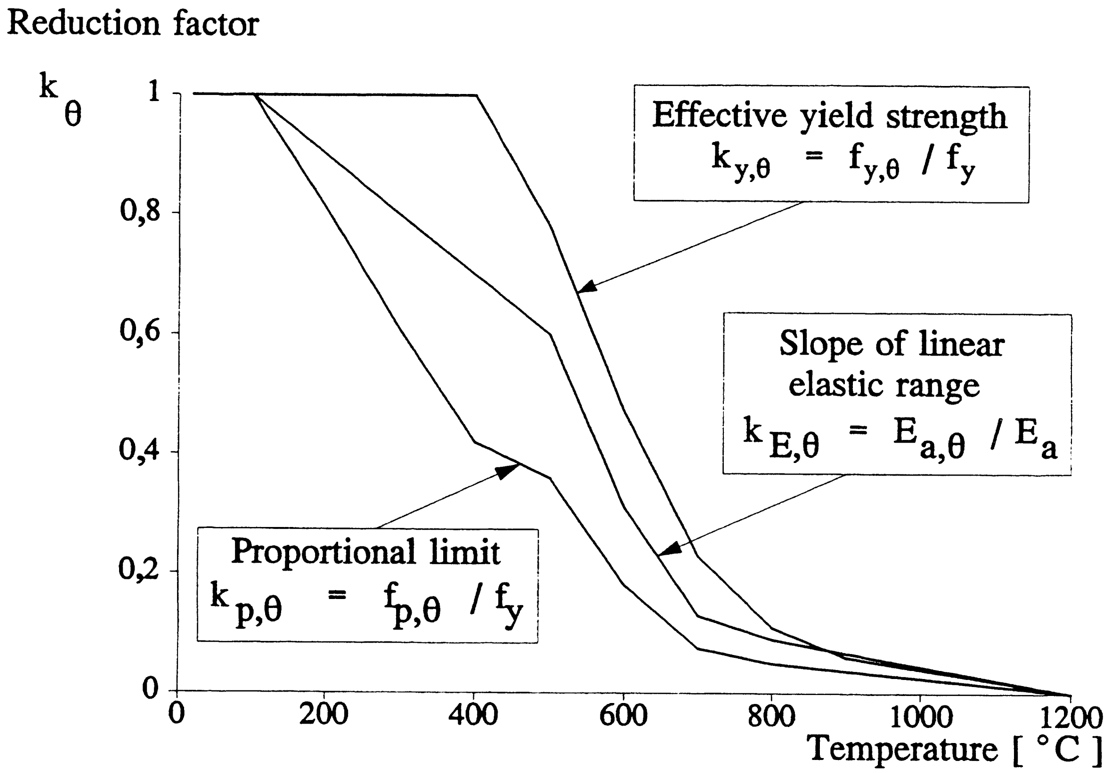


Figure 3.2: Reduction factors for the stress-strain relationship of steel at elevated temperatures

3.3 Thermal properties

3.3.1 Steel

3.3.1.1 Thermal elongation

(1)P The thermal elongation of steel $\Delta\ell / \ell$ may be determined from the following:

- for $20\,°C \leq \theta_a < 750\,°C$:

$$\Delta\ell / \ell = 1{,}2 \times 10^{-5}\,\theta_a + 0{,}4 \times 10^{-8}\,\theta_a^2 - 2{,}416 \times 10^{-4} \quad (3.1a)$$

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

$$\Delta\ell / \ell = 1{,}1 \times 10^{-2} \quad (3.1b)$$

- for $860\,°C < \theta_a \leq 1200\,°C$:

$$\Delta\ell / \ell = 2 \times 10^{-5}\,\theta_a - 6{,}2 \times 10^{-3} \quad (3.1c)$$

where:

ℓ is the length at 20 °C;

$\Delta\ell$ is the temperature induced expansion;

θ_a is the steel temperature [°C].

(2) The variation of the thermal elongation with temperature is illustrated in figure 3.3.

(3)P In simple calculation models (see 4.2) the relationship between thermal elongation and steel temperature may be considered to be constant. In this case the elongation may be determined from:

$$\Delta\ell / \ell = 14 \times 10^{-6}\,(\theta_a - 20) \quad (3.1d)$$

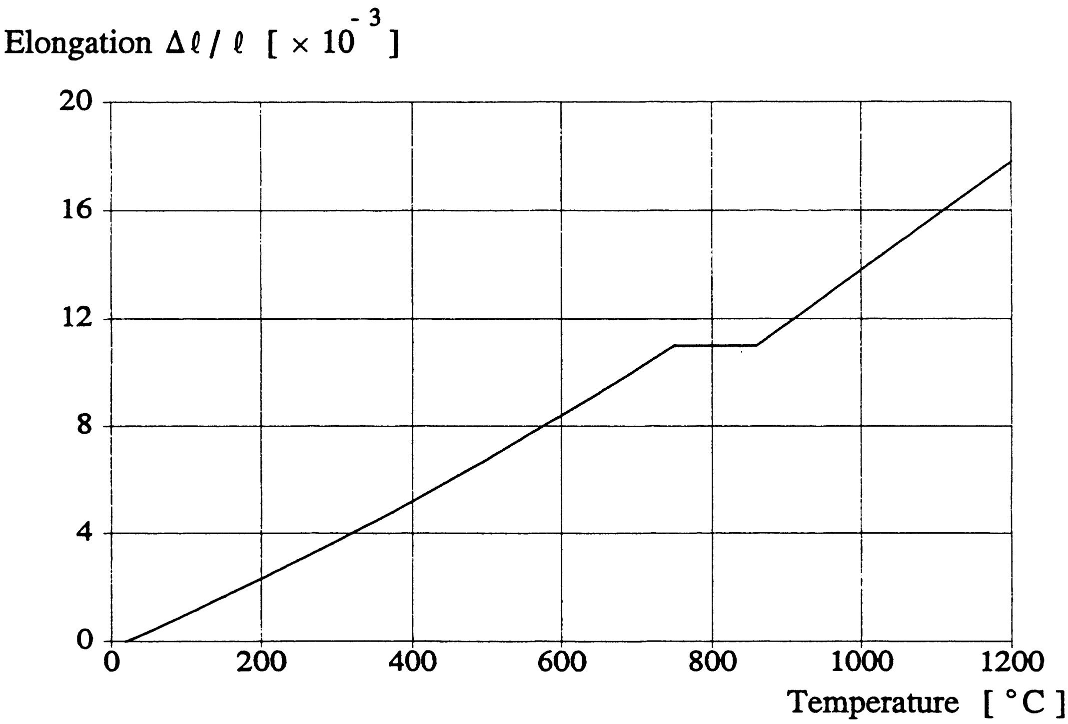


Figure 3.3: Thermal elongation of steel as a function of the temperature

3.3.1.2 Specific heat

(1)P The specific heat of steel c_a may be determined from the following:

- for $20\,°C \leq \theta_a < 600\,°C$:

$$c_a = 425 + 7{,}73 \times 10^{-1}\,\theta_a - 1{,}69 \times 10^{-3}\,\theta_a^2 + 2{,}22 \times 10^{-6}\,\theta_a^3 \text{ J/kgK} \qquad (3.2a)$$

- for $600\,°C \leq \theta_a < 735\,°C$:

$$c_a = 666 + \frac{13002}{738 - \theta_a} \text{ J/kgK} \qquad (3.2b)$$

- for $735\,°C \leq \theta_a < 900\,°C$:

$$c_a = 545 + \frac{17820}{\theta_a - 731} \text{ J/kgK} \qquad (3.2c)$$

- for $900\,°C \leq \theta_a \leq 1200\,°C$:

$$c_a = 650 \text{ J/kgK} \qquad (3.2d)$$

where:

θ_a is the steel temperature [°C].

(2) The variation of the specific heat with temperature is illustrated in figure 3.4.

(3)P In simple calculation models (see 4.2) the specific heat may be considered to be independent of the steel temperature. In this case the following value may be taken:

$$c_a = 600 \text{ J/kgK} \qquad (3.2e)$$

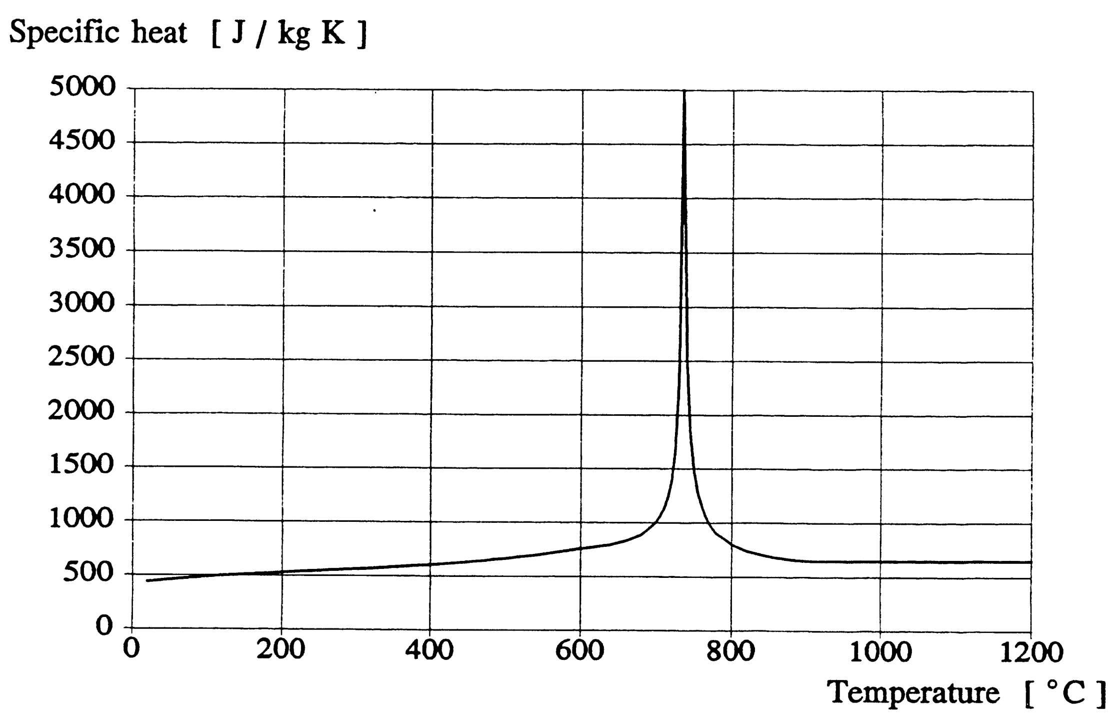


Figure 3.4: Specific heat of steel as a function of the temperature

3.3.1.3 Thermal conductivity

(1)P The thermal conductivity of steel λ_a may be determined from the following:

- for $20\,°C \leq \theta_a < 800\,°C$:

$$\lambda_a = 54 - 3{,}33 \times 10^{-2}\,\theta_a \text{ W/mK} \qquad (3.3a)$$

- for $800\,°C \leq \theta_a \leq 1200\,°C$:

$$\lambda_a = 27{,}3 \text{ W/mK} \qquad (3.3b)$$

where:

θ_a is the steel temperature [°C].

(2) The variation of the thermal conductivity with temperature is illustrated in figure 3.5.

(3)P In simple calculation models (see 4.2) the thermal conductivity may be considered to be independent of the steel temperature. In this case the following value may be taken:

$$\lambda_a = 45 \text{ W/mK} \qquad (3.3c)$$

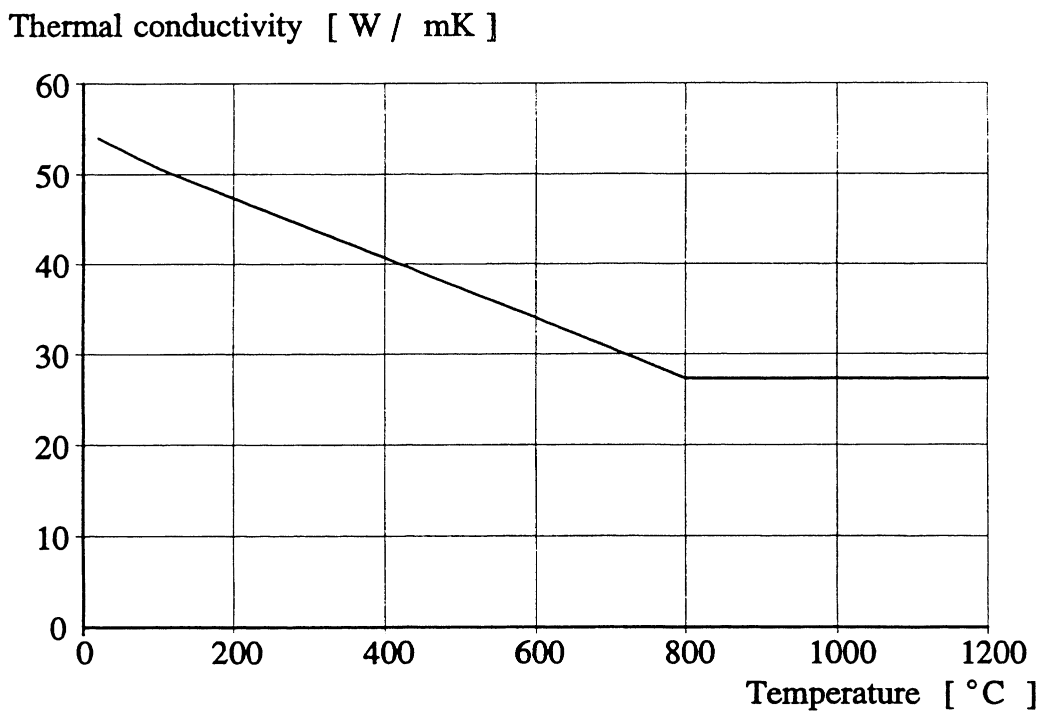


Figure 3.5: Thermal conductivity of steel as a function of the temperature

3.3.2 Fire protection materials

(1)P The properties and performance of fire protection materials shall be assessed using the test procedures given in pr ENV yyy5-1, pr ENV yyy5-2 or pr ENV yyy5-4 as appropriate.

NOTE: This assumes that these standards will include a requirement that the fire protection materials shall remain coherent and cohesive to their supports throughout the relevant fire exposure.

4 Structural fire design

4.1 General

(1)P Steelwork may be either:

- unprotected;
- insulated by fire protection material;
- protected by heat screens;
- protected by any other method that limits the temperature rise of the steel.

NOTE: Examples of other methods include water filling or partial protection in walls and floors.

(2)P The assessment of structural behaviour in a fire design situation shall be based on one of the following approaches, or on a combination of them:

- simple calculation models applied to individual members;
- advanced calculation models;
- testing.

(3)P Simple calculation models are simplified design methods which give conservative results.

(4)P Advanced calculation models are design methods in which engineering principles are applied in a realistic manner to specific applications.

(5)P Where no simple calculation model is given, it is necessary to use either a design method based on an advanced calculation model or a method based on test results.

4.2 Simple calculation models

4.2.1 General

(1)P The load-bearing function of a steel member shall be assumed to be maintained after a time t in a given fire if:

$$E_{fi,d} \leq R_{fi,d,t} \tag{4.1}$$

where:

$E_{fi,d}$ is the design effect of actions for the fire design situation, according to ENV 1991-2-2;

$R_{fi,d,t}$ is the corresponding design resistance of the steel member, for the fire design situation, at time t.

(2)P The design resistance $R_{fi,d,t}$ at time t shall be determined for the temperature distribution in the cross-section by modifying the design resistance for normal temperature design to ENV 1993-1-1, to take account of the mechanical properties of steel at elevated temperatures, see 4.2.3.

NOTE: In 4.2.3 $R_{fi,d,t}$ becomes $M_{fi,t,Rd}$, $N_{fi,t,Rd}$ etc (separately or in combination) and the corresponding values of $M_{fi,Ed}$, $N_{fi,Ed}$ etc represent $E_{fi,d}$.

(3)P Alternatively, by using a uniform temperature distribution, the verification may be carried out in the temperature domain, see 4.2.4.

(4) Net-section failure at fastener holes need not be considered, provided that there is a fastener in each hole, because the steel temperature is lower at connections due to the presence of additional material.

(5)P The resistance of connections between members need not be checked provided that the thermal resistance $(d_f/\lambda_f)_c$ of the fire protection of the connection is not less than the minimum value of the thermal resistance $(d_f/\lambda_f)_m$ of the fire protection of any of the steel members joined by that connection, where:

d_f	is	the thickness of the fire protection material — take $d_f = 0$ for unprotected members;
λ_f	is	the effective thermal conductivity of the fire protection material.

(6) Where the means of protection, or the design criteria for separating members, require the consideration of deformation criteria, see 2.1(2), the verification should be carried out as specified in 4.2.3, but substituting the reduction factors $k_{y,\theta}$ and $k_{y,\theta,max}$ for effective yield strength at temperatures θ_a and $\theta_{a,max}$ by modified reduction factors $k_{x,\theta}$ and $k_{x,\theta,max}$, see 3.2.1 and table 3.1.

4.2.2 Classification of cross-sections

(1) In a fire design situation, the classification of cross-sections as defined in 5.3 of ENV 1993-1-1 should take due account of the stress-strain relationship of steel at the relevant steel temperature.

(2) A compression member may be classified as for normal temperature design, without any change.

(3) A simply supported beam with a composite or concrete slab on the compression flange, but exposed on the other three sides, may be classified as for normal temperature design, without any change.

(4) Any other member may be classified as for normal temperature design, but using a modified value of ε in table 5.3.1 of ENV 1993-1-1, given by:

$$\varepsilon = [(235/f_y)(k_{E,\theta}/k_{y,\theta})]^{0,5} \tag{4.2}$$

(5) When considering deformation criteria, see 2.1(2), a member may be classified as for normal temperature design, without any change.

4.2.3 Resistance

4.2.3.1 Tension members

(1)P The design resistance $N_{fi,t,Rd}$ at time t of a tension member with a non-uniform temperature distribution across the cross-section may be determined from:

$$N_{fi,t,Rd} = \sum_{i=1}^{n} A_i\, k_{y,\theta,i}\, f_y/\gamma_{M,fi} \tag{4.3}$$

where:

A_i	is	an elemental area of the cross-section with a temperature θ_i;
$k_{y,\theta,i}$	is	the reduction factor for the yield strength of steel at temperature θ_i, see 3.2.1;
θ_i	is	the temperature in the elemental area A_i.

(2) The design resistance $N_{fi,t,Rd}$ at time t of a tension member with a non-uniform temperature distribution may conservatively be taken as equal to the design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform steel temperature θ_a equal to the maximum steel temperature $\theta_{a,max}$ reached at time t.

(3) The design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform temperature θ_a should be determined from:

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] \tag{4.4}$$

where:

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, see 3.2.1;

N_{Rd} is the design resistance of the gross cross-section $N_{p\ell,Rd}$ for normal temperature design, according to 5.4.3 of ENV 1993-1-1.

4.2.3.2 Compression members with Class 1, Class 2 or Class 3 cross-sections

(1) The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a Class 1, Class 2 or Class 3 cross-section should be determined from:

$$N_{b,fi,t,Rd} = [\chi_{fi} / 1,2] A\, k_{y,\theta,max} f_y / \gamma_{M,fi} \tag{4.5}$$

where:

χ_{fi} is the reduction factor for flexural buckling in the fire design situation;

$k_{y,\theta,max}$ is the reduction factor from 3.2.1 for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t.

NOTE: The constant 1,2 in this expression is a correction factor that allows for a number of effects, including the difference in the strain at failure compared to $\varepsilon_{y,\theta}$. The value is empirical.

(2) The value of χ_{fi} should be taken as the lesser of the values of $\chi_{y,fi}$ and $\chi_{z,fi}$ determined as given in 5.5.1 of ENV 1993-1-1, except using:

- buckling curve c, irrespective of the type of cross-section or the axis of buckling;
- the buckling length ℓ_{fi} for the fire design situation in place of ℓ;
- the non-dimensional slenderness $\bar{\lambda}_{\theta,max}$ for the temperature $\theta_{a,max}$, given by:

$$\bar{\lambda}_{\theta,max} = \bar{\lambda} [k_{y,\theta,max} / k_{E,\theta,max}]^{0,5} \tag{4.6}$$

where:

$k_{y,\theta,max}$ is the reduction factor from 3.2.1 for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t;

$k_{E,\theta,max}$ is the reduction factor from 3.2.1 for the slope of the linear elastic range at the maximum steel temperature $\theta_{a,max}$ reached at time t.

(3) The buckling length ℓ_{fi} of a column for the fire design situation should generally be determined as for normal temperature design. However, in a braced frame the buckling length ℓ_{fi} of a column length may be determined by considering it as fixed in direction at continuous or semi-continuous connections to the column lengths in the fire compartments above and below, provided that the fire resistance of the building components that separate these fire compartments is not less than the fire resistance of the column.

(4) In the case of a steel frame in which each storey comprises a separate fire compartment with sufficient fire resistance, in an intermediate storey the buckling length of a column $\ell_{fi} = 0,5L$ and in the top storey the buckling length $\ell_{fi} = 0,7L$, where L is the system length in the relevant storey, see figure 4.1.

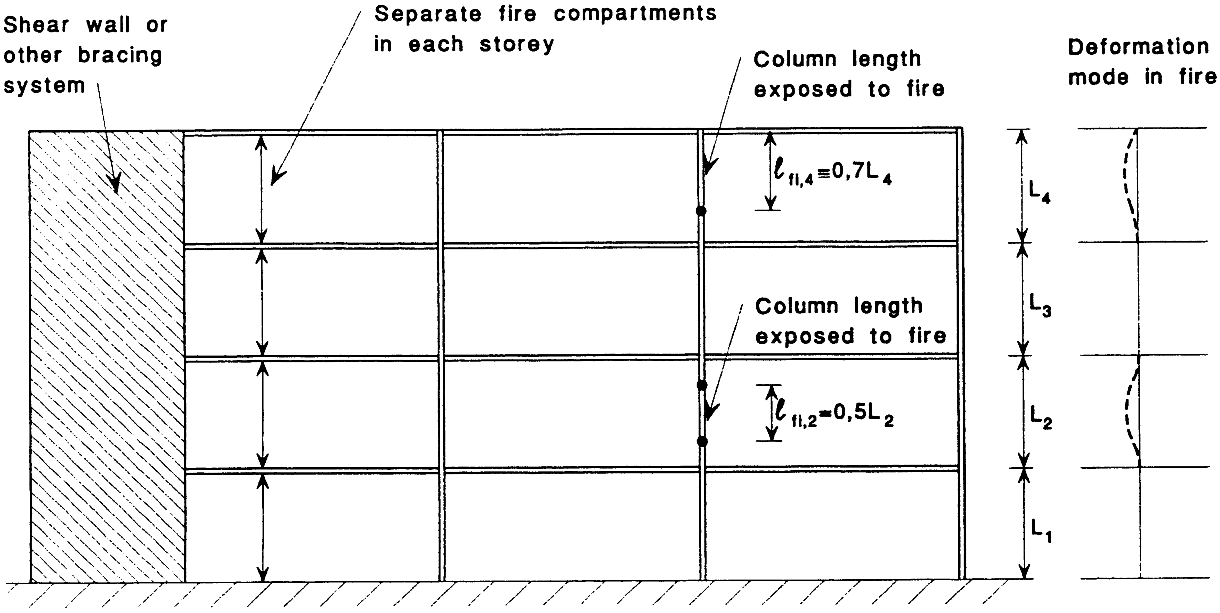


Figure 4.1: Buckling lengths ℓ_{fi} of columns in braced frames

4.2.3.3 Beams with Class 1 or Class 2 cross-sections

(1)P The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution across the cross-section may be determined from:

$$M_{fi,t,Rd} = \sum_{i=1}^{n} A_i z_i k_{y,\theta,i} f_{y,i} / \gamma_{M,fi} \tag{4.7}$$

where:

- z_i is the distance from the plastic neutral axis to the centroid of the elemental area A_i;
- $f_{y,i}$ is the nominal yield strength f_y for the elemental area A_i taken as positive on the compression side of the plastic neutral axis and negative on the tension side;
- A_i and $k_{y,\theta,i}$ are as defined in 4.2.3.1(1).

(2)P The plastic neutral axis of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution is the axis perpendicular to the plane of bending that satisfies the following criterion:

$$\sum_{i=1}^{n} A_i k_{y,\theta,i} f_{y,i} = 0 \tag{4.8}$$

(3) Alternatively, the design moment resistance $M_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section in a member with a non-uniform temperature distribution, may conservatively be determined from:

$$M_{fi,t,Rd} = M_{fi,\theta,Rd} / \kappa_1 \kappa_2 \tag{4.9}$$

where:

- $M_{fi,\theta,Rd}$ is the design moment resistance of the cross-section for a uniform temperature θ_a equal to the maximum temperature $\theta_{a,max}$ reached in the cross-section at time t;
- κ_1 is an adaptation factor for non-uniform temperature across the cross-section, see (8);
- κ_2 is an adaptation factor for non-uniform temperature along the beam, see (9).

(4)P The design moment resistance $M_{fi,\theta,Rd}$ of a Class 1 or Class 2 cross-section with a uniform temperature θ_a may be determined from:

$$M_{fi,\theta,Rd} = k_{y,\theta}[\gamma_{M,1}/\gamma_{M,fi}]M_{Rd} \tag{4.10}$$

where:

M_{Rd} is the plastic moment resistance of the gross cross-section $M_{p\ell,Rd}$ for normal temperature design, according to 5.4.5 of ENV 1993-1-1 or the reduced moment resistance for normal temperature design, allowing for the effects of shear if necessary, according to 5.4.7 of ENV 1993-1-1;

$k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, see 3.2.1

(5) Provided that the non-dimensional slenderness $\bar{\lambda}_{LT,\theta,com}$ for the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t does not exceed 0,4 no allowance need be made for lateral-torsional buckling. Where $\bar{\lambda}_{LT,\theta,com} > 0,4$ the design buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 1 or Class 2 cross-section should be determined from:

$$M_{b,fi,t,Rd} = [\chi_{LT,fi}/1,2]\, W_{p\ell,y} k_{y,\theta,com} f_y / \gamma_{M,fi} \tag{4.11}$$

where:

$\chi_{LT,fi}$ is the reduction factor for lateral-torsional buckling in the fire design situation;

$k_{y,\theta,com}$ is the reduction factor from 3.2.1 for the yield strength of steel at the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t.

NOTE 1: The constant 1,2 in this expression is a correction factor that allows for a number of effects. The value of 1,2 is the same as the empirically determined value for compression members.

NOTE 2: Conservatively $\theta_{a,com}$ can be assumed to be equal to the maximum temperature $\theta_{a,max}$.

(6) The value of $\chi_{LT,fi}$ should be determined as given in 5.5.2 of ENV 1993-1-1, except using the non-dimensional slenderness $\bar{\lambda}_{LT,\theta,com}$ for the temperature $\theta_{a,com}$ given by:

$$\bar{\lambda}_{LT,\theta,com} = \bar{\lambda}_{LT}[k_{y,\theta,com}/k_{E,\theta,com}]^{0,5} \tag{4.12}$$

where:

$k_{E,\theta,com}$ is the reduction factor from 3.2.1 for the slope of the linear elastic range at the maximum steel temperature in the compression flange $\theta_{a,com}$ reached at time t.

(7) The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 1 or Class 2 cross-section with a non-uniform temperature distribution may be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,max} V_{Rd}[\gamma_{M,1}/\gamma_{M,fi}]/\kappa_1\kappa_2 \tag{4.13}$$

where:

V_{Rd} is the shear resistance of the gross cross-section for normal temperature design, according to 5.4.6 of ENV 1993-1-1.

(8) The value of the adaptation factor κ_1 for non-uniform temperature distribution across a cross-section should be taken as follows:

- for a beam exposed on all four sides: $\kappa_1 = 1,0$;
- for a beam exposed on three sides, with a composite or concrete slab on side four: $\kappa_1 = \boxed{0,70}$.

(9) The value of the adaptation factor κ_2 for non-uniform temperature distribution along a beam should be taken as follows:

- at the supports of a statically indeterminate beam: $\kappa_2 = \boxed{0,85}$;
- in all other cases: $\kappa_2 = 1,0$.

4.2.3.4 Beams with Class 3 cross-sections

(1) The design moment resistance $M_{fi,t,Rd}$ at time t of a Class 3 cross-section with a non-uniform temperature distribution should be determined from:

$$M_{fi,t,Rd} = k_{y,\theta,max} M_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] / \kappa_1 \kappa_2 \tag{4.14}$$

where:

M_{Rd} is the elastic moment resistance of the gross cross-section $M_{el,Rd}$ for normal temperature design, according to 5.4.5 of ENV 1993-1-1 or the reduced moment resistance allowing for the effects of shear if necessary according to 5.4.7 of ENV 1993-1-1;

$k_{y,\theta,max}$ is the reduction factor for the yield strength of steel at the maximum steel temperature $\theta_{a,max}$ reached at time t, see 3.2.1;

κ_1 is an adaptation factor for non-uniform temperature in a cross-section, see 4.2.3.3(8);

κ_2 is an adaptation factor for non-uniform temperature along the beam, see 4.2.3.3(9).

(2) Provided that the non-dimensional slenderness $\bar{\lambda}_{LT,\theta,com}$ for the maximum temperature in the compression flange $\theta_{a,com}$ reached at time t does not exceed 0,4 no allowance need be made for lateral-torsional buckling. Where $\bar{\lambda}_{LT,\theta,com} > 0,4$ the design buckling resistance moment $M_{b,fi,t,Rd}$ at time t of a laterally unrestrained beam with a Class 3 cross-section should be determined from:

$$M_{b,fi,t,Rd} = [\chi_{LT,fi} / 1,2] \, W_{el,y} k_{y,\theta,com} f_y / \gamma_{M,fi} \tag{4.15}$$

where:

$\chi_{LT,fi}$ is as given in 4.2.3.3(6).

NOTE 1: The constant 1,2 in this expression is a correction factor that allows for a number of effects. The value of 1,2 is the same as the empirically determined value for compression members.

NOTE 2: Conservatively $\theta_{a,com}$ can be assumed to be equal to the maximum temperature $\theta_{a,max}$.

(3) The design shear resistance $V_{fi,t,Rd}$ at time t of a Class 3 cross-section with a non-uniform temperature distribution may be determined from:

$$V_{fi,t,Rd} = k_{y,\theta,max} V_{Rd} [\gamma_{M,1} / \gamma_{M,fi}] / \kappa_1 \kappa_2 \tag{4.16}$$

where:

V_{Rd} is the shear resistance of the gross cross-section for normal temperature design, according to 5.4.6 of ENV 1993-1-1.

4.2.3.5 Members with Class 1, 2 or 3 cross-sections, subject to bending and axial compression

(1) The design buckling resistance $R_{fi,t,d}$ at time t of a member subject to combined bending and axial compression should be verified by satisfying expressions (5.51) and (5.52) of ENV 1993-1-1 for a member with a Class 1 or Class 2 cross-section, or expressions (5.53) and (5.54) of ENV 1993-1-1 for a member with a Class 3 cross-section, using the modified values given in (2) and (3).

(2) The modified values of the internal forces and moments should be taken as:

$$M_{y,Sd} = M_{y,fi,Ed} \tag{4.17a}$$

$$M_{z,Sd} = M_{z,fi,Ed} \tag{4.17b}$$

$$N_{Sd} = N_{fi,Ed} \tag{4.17c}$$

(3) The resistance terms should be modified by using:

- $[\chi_{y,fi} / 1,2]$ in place of χ_y, where $\chi_{y,fi}$ is as defined in 4.2.3.2(2);
- $[\chi_{z,fi} / 1,2]$ in place of χ_z, where $\chi_{z,fi}$ is as defined in 4.2.3.2(2);
- $[\chi_{LT,fi} / 1,2]$ in place of χ_{LT}, where $\chi_{LT,fi}$ is as defined in 4.2.3.3(6);
- $k_{y,\theta,max} f_y$ in place of f_y, where $k_{y,\theta,max}$ is as defined in 4.2.3.2(1);
- $\gamma_{M,fi}$ in place of γ_{M1}.

4.2.4 Critical temperature

(1)P As an alternative to 4.2.3, verification may be carried out in the temperature domain.

(2) Except when considering deformation criteria, the critical steel temperature $\theta_{a,cr}$ at time t for a uniform temperature distribution may be determined for any degree of utilisation μ_0 at time $t = 0$ using:

$$\theta_{a,cr} = 39,19 \, \ell n \left[\frac{1}{0,9674 \, \mu_0^{3,833}} - 1 \right] + 482 \tag{4.18}$$

(3) Values of $\theta_{a,cr}$ for values of μ_0 from 0,22 to 0,80 are given in table 4.1.

(4) For members with Class 1, Class 2 or Class 3 cross-sections and for all tension members, the degree of utilisation μ_0 at time $t = 0$ may be obtained from:

$$\mu_0 = E_{fi,d} / R_{fi,d,0} \tag{4.19}$$

where:

$R_{fi,d,0}$ is the value of $R_{fi,d,t}$ for time $t = 0$, from 4.2.3;

$E_{fi,d}$ and $R_{fi,d,t}$ are as defined in 4.2.1(1).

(5) Alternatively for tension members, and for beams where lateral-torsional buckling is not a potential failure mode, μ_0 may conservatively be obtained from:

$$\mu_0 = \eta_{fi} [\gamma_{M,fi} / \gamma_{M1}] \tag{4.20}$$

where:

η_{fi} is the reduction factor defined in 2.4.3(3).

(6) For members with Class 4 cross-sections, other than tension members, it may be assumed that 4.2.1(1) is satisfied if at time t the steel temperature θ_a at all cross-sections is not more than 350 °C.

Table 4.1: Critical temperature $\theta_{a,cr}$ for values of the utilisation factor μ_0

μ_0	$\theta_{a,cr}$	μ_0	$\theta_{a,cr}$	μ_0	$\theta_{a,cr}$
0,22	711	0,42	612	0,62	549
0,24	698	0,44	605	0,64	543
0,26	685	0,46	598	0,66	537
0,28	674	0,48	591	0,68	531
0,30	664	0,50	585	0,70	526
0,32	654	0,52	578	0,72	520
0,34	645	0,54	572	0,74	514
0,36	636	0,56	566	0,76	508
0,38	628	0,58	560	0,78	502
0,40	620	0,60	554	0,80	496

4.2.5 Steel temperature development

4.2.5.1 Unprotected internal steelwork

(1) For an equivalent uniform temperature distribution in the cross-section, the increase of temperature $\Delta\theta_{a,t}$ in an unprotected steel member during a time interval Δt may be determined from:

$$\Delta\theta_{a,t} = \frac{A_m / V}{c_a \rho_a} \dot{h}_{net,d} \Delta t \tag{4.21}$$

where:

- A_m / V is the section factor for unprotected steel members;
- A_m is the exposed surface area of the member per unit length;
- V is the volume of the member per unit length;
- c_a is the specific heat of steel, from 3.3.1.2 [J/kgK];
- $\dot{h}_{net,d}$ is the design value of the net heat flux per unit area [W/m^2];
- Δt is the time interval [seconds];
- ρ_a is the unit mass of steel, from 3.2.2(1) [kg/m^3].

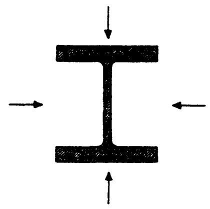
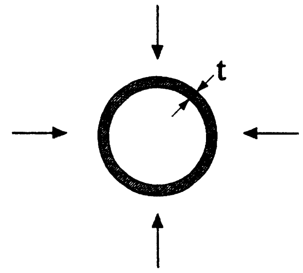
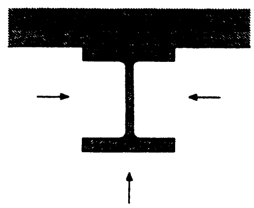
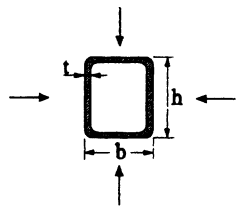
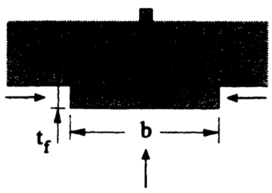
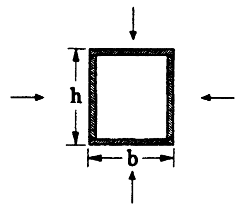
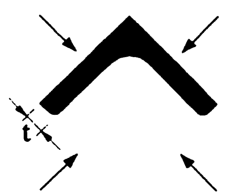
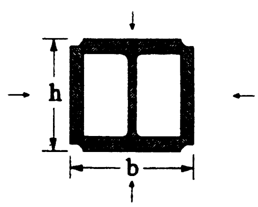
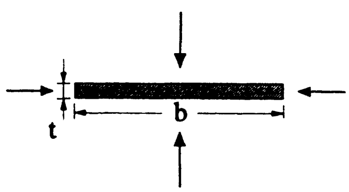
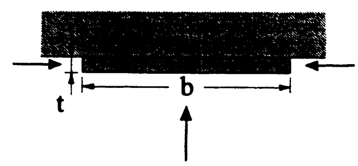
(2) The value of $\dot{h}_{net,d}$ should be obtained from ENV 1991-2-2 using $\varepsilon_f = 0{,}8$ and $\varepsilon_m = 0{,}625$ leading to $\varepsilon_{res} = 0{,}5$, where ε_f, ε_m and ε_{res} are as defined in ENV 1991-2-2.

(3) The value of Δt should not be taken as more than 5 seconds.

(4) In expression (4.21) the value of the section factor A_m / V should not be taken as less than $10\,\text{m}^{-1}$.

(5) Some expressions for calculating design values of the section factor A_m / V for unprotected steel members are given in table 4.2.

Table 4.2: Section factor A_m/V for unprotected steel members.

Open section exposed to fire on all sides: $\dfrac{A_m}{V} = \dfrac{\text{perimeter}}{\text{cross-section area}}$ 	Tube exposed to fire on all sides: $A_m/V = 1/t$ 
Open section exposed to fire on three sides: $\dfrac{A_m}{V} = \dfrac{\text{surface exposed to fire}}{\text{cross-section area}}$ 	Hollow section (or welded box section of uniform thickness) exposed to fire on all sides: If $t \ll b$: $A_m/V \approx 1/t$ 
I-section flange exposed to fire on three sides: $A_m/V = (b + 2t_f)/(bt_f)$ If $t \ll b$: $A_m/V \approx 1/t_f$ 	Welded box section exposed to fire on all sides: $\dfrac{A_m}{V} = \dfrac{2(b+h)}{\text{cross-section area}}$ 
Angle (or any open section of uniform thickness) exposed to fire on all sides: $A_m/V = 2/t$ 	I-section with box reinforcement, exposed to fire on all sides: $\dfrac{A_m}{V} = \dfrac{2(b+h)}{\text{cross-section area}}$ 
Flat bar exposed to fire on all sides: $A_m/V = 2(b+t)/(bt)$ If $t \ll b$: $A_m/V \approx 2/t$ 	Flat bar exposed to fire on three sides: $A_m/V = (b+2t)/(bt)$ If $t \ll b$: $A_m/V \approx 1/t$ 

4.2.5.2 Internal steelwork insulated by fire protection material

(1) For a uniform temperature distribution in a cross-section, the temperature increase $\Delta\theta_{a,t}$ of an insulated steel member during a time interval Δt may be obtained from:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p / V}{d_p c_a \rho_a} \frac{(\theta_{g,t} - \theta_{a,t})}{(1 + \phi/3)} \Delta t - (e^{\phi/10} - 1)\Delta\theta_{g,t} \quad \text{but} \quad \Delta\theta_{a,t} \geq 0 \qquad (4.22)$$

with:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p / V$$

where:

A_p / V	is	the section factor for steel members insulated by fire protection material;
A_p	is	the appropriate area of fire protection material per unit length of the member;
V	is	the volume of the member per unit length;
c_a	is	the specific heat of steel, from 3.3.1.2 [J/kgK];
c_p	is	the specific heat of the fire protection material [J/kgK];
d_p	is	the thickness of the fire protection material [m];
Δt	is	the time interval [seconds];
$\theta_{a,t}$	is	the steel temperature at time t;
$\theta_{g,t}$	is	the ambient gas temperature at time t;
$\Delta\theta_{g,t}$	is	the increase of the ambient gas temperature during the time interval Δt;
λ_p	is	the thermal conductivity of the fire protection material [W/mK];
ρ_a	is	the unit mass of steel, from 3.2.2 [kg/m^3];
ρ_p	is	the unit mass of the fire protection material [kg/m^3].

(2) The values of c_p, λ_p and ρ_p should be determined as specified in 3.3.2.

(3) The value of Δt should not be taken as more than 30 seconds.

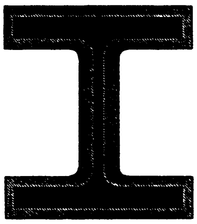
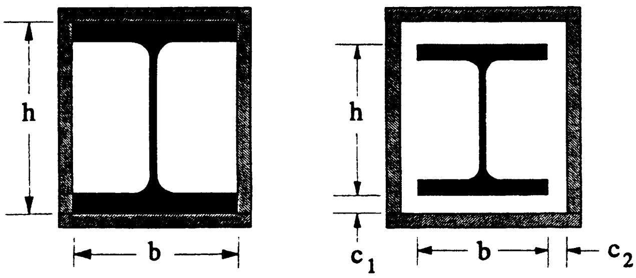
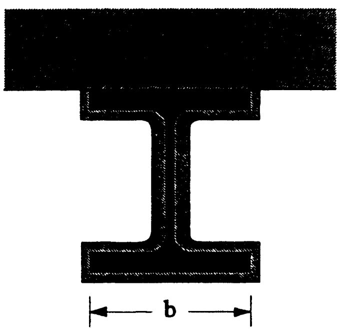
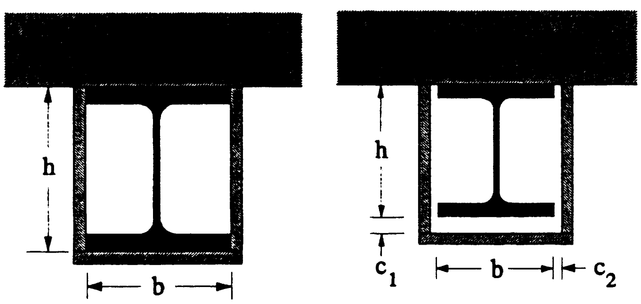
(4) The area A_p of the fire protection material should generally be taken as the area of its inner surface, but for hollow encasement with a clearance around the steel member the same value as for hollow encasement without a clearance may be adopted.

(5) Some design values of the section factor A_p / V for insulated steel members are given in table 4.3.

(6) For moist fire protection materials the calculation of the steel temperature increase $\Delta\theta_a$ may be modified to allow for a time delay in the rise of the steel temperature when it reaches 100 °C. This delay time should be determined by a method conforming with pr ENV yyy5-4.

(7) Alternatively, the uniform temperature of an insulated steel member after a given time duration of standard fire exposure may be obtained using design flow charts derived in conformity with pr ENV yyy5-4.

Table 4.3: Section factor A_p / V for steel members insulated by fire protection material

Sketch	Description	Section factor (A_p / V)
	Contour encasement of uniform thickness	$\dfrac{\text{steel perimeter}}{\text{steel cross-section area}}$
	Hollow encasement[1] of uniform thickness	$\dfrac{2(b + h)}{\text{steel cross-section area}}$
	Contour encasement of uniform thickness, exposed to fire on three sides	$\dfrac{\text{steel perimeter} - b}{\text{steel cross-section area}}$
	Hollow encasement[1] of uniform thickness, exposed to fire on three sides	$\dfrac{2h + b}{\text{steel cross-section area}}$

[1] The clearance dimensions c_1 and c_2 should not normally exceed $h/4$.

4.2.5.3 Internal steelwork in a void that is protected by heat screens

(1)P The provisions given below apply to both of the following cases:

- steel members in a void that is bordered by a floor on top and by a horizontal heat screen below, and
- steel members in a void that is bordered by vertical heat screens on both sides,

provided in both cases that there is a gap between the heat screen and the member. They do not apply if the heat screen is in direct contact with the member.

(2)P The properties and performance of the heat screens shall be determined using a test procedure conforming with pr ENV yyy5-1 or pr ENV yyy5-2 as appropriate.

(3)P The temperature development in the void in which the steel members are situated shall be determined from a standard fire test conforming with pr ENV yyy5-1 or pr ENV yyy5-2 as appropriate.

(4) For internal steelwork protected by heat screens, the calculation of the steel temperature increase $\Delta\theta_a$ should be based on the methods given in 4.2.5.1 or 4.2.5.2 as appropriate, taking the ambient gas temperature $\theta_{g,t}$ as equal to the gas temperature in the void.

(5) As an alternative to the procedure given in 4.2.5.1, $\Delta\theta_a$ may be calculated using values of the convective and radiative heat transfer coefficients α_c and α_r determined from tests conforming with pr ENV yyy5-1.

4.2.5.4 External steelwork

(1)P The temperature in external steelwork shall be determined taking into account:

- the radiative heat flux from the fire compartment
- the radiative heat flux and the convective heat flux from the flames emanating from openings
- the radiative and convective heat loss from the steelwork to the ambient atmosphere
- the sizes and locations of the structural members.

(2)P Heat screens may be provided on one, two or three sides of an external steel member in order to protect it from radiative heat transfer.

(3) Heat screens should be either:

- directly attached to that side of the steel member that it is intended to protect, or
- large enough to fully screen that side from the expected radiative heat flux.

(4) Heat screens should be non-combustible and have a fire resistance of at least EI 30 according to pr ISO EN 834.

(5) The temperature in external steelwork protected by heat screens should be determined as specified in (1), assuming that there is no radiative heat transfer to those sides that are protected by heat screens.

(6) Calculations may be based on steady state conditions resulting from a stationary heat balance using the methods given in annex C.

(7) Design using annex C of this Part 1-2 of ENV 1993 should be based on the model given in annex C of ENV 1991-2-2 describing the compartment fire conditions and the flames emanating from openings, on which the calculation of the radiative and convective heat fluxes should be based.

4.3 Advanced calculation models

4.3.1 Basis

(1)P Advanced calculation models may be used for individual members, for sub-assemblies or for entire structures.

(2)P Advanced calculation methods may be used with any type of cross-section.

(3)P Advanced calculation methods shall provide a realistic analysis of structures exposed to fire. They shall be based on fundamental physical behaviour in such a way as to lead to a reliable approximation of the expected behaviour of the relevant structural component under fire conditions.

(4)P Advanced calculation methods may include separate calculation models for the determination of:

- the development and distribution of the temperature within structural members (thermal response model);
- the mechanical behaviour of the structure or of any part of it (mechanical response model).

(5)P Any potential failure modes not covered by the advanced calculation method (including local buckling and failure in shear) shall be eliminated by appropriate means.

(6)P Advanced calculation methods may be used in association with any heating curve, provided that the material properties are known for the relevant temperature range.

(7)P The validity of any specific advanced calculation method for a particular situation shall be agreed between the client, the designer and the competent authority.

4.3.2 Thermal response

(1)P Advanced calculation methods for thermal response shall be based on the acknowledged principles and assumptions of the theory of heat transfer.

(2)P The thermal response model shall consider:

- the relevant thermal actions specified in ENV 1991-2-2;
- the variation of the thermal properties of the material with the temperature, see 3.3.

(3) The effects of non-uniform thermal exposure and of heat transfer to adjacent building components may be included where appropriate.

(4) The influence of any moisture content and of any migration of the moisture within the fire protection material may conservatively be neglected.

4.3.3 Mechanical response

(1)P Advanced calculation methods for mechanical response shall be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature.

(2)P The effects of thermally induced strains and stresses both due to temperature rise and due to temperature differentials, shall be considered.

(3)P Where relevant, the mechanical response of the model shall also take account of:

- the combined effects of mechanical actions, geometrical imperfections and thermal actions;
- the temperature dependent mechanical properties of the material, see 3.2;
- geometrical non-linear effects;
- the effects of non-linear material properties, including the beneficial effects of loading and unloading on the structural stiffness.

(4) Provided that the stress-strain relationships given in 3.2 are used, the effects of transient thermal creep need not be given explicit consideration.

(5)P The deformations at ultimate limit state implied by the calculation method shall be limited as necessary to ensure that compatibility is maintained between all parts of the structure.

(6) If necessary, the design should be based on the ultimate limit state beyond which the calculated deformations of the structure would cause failure due to the loss of adequate support to one of the members.

Annex A [informative]

Stress-strain relationships at elevated temperatures (no strain-hardening)

(1) The stress strain relationship specified in 3.2.1 is evaluated for steel grades S 235, S 275, S 355 and S 460 in tables A.1 to A.4 respectively. The variation of this relationship with temperature is illustrated in figures A.1 to A.4 for steel grades S 235, S 275, S 355 and S 460 respectively.

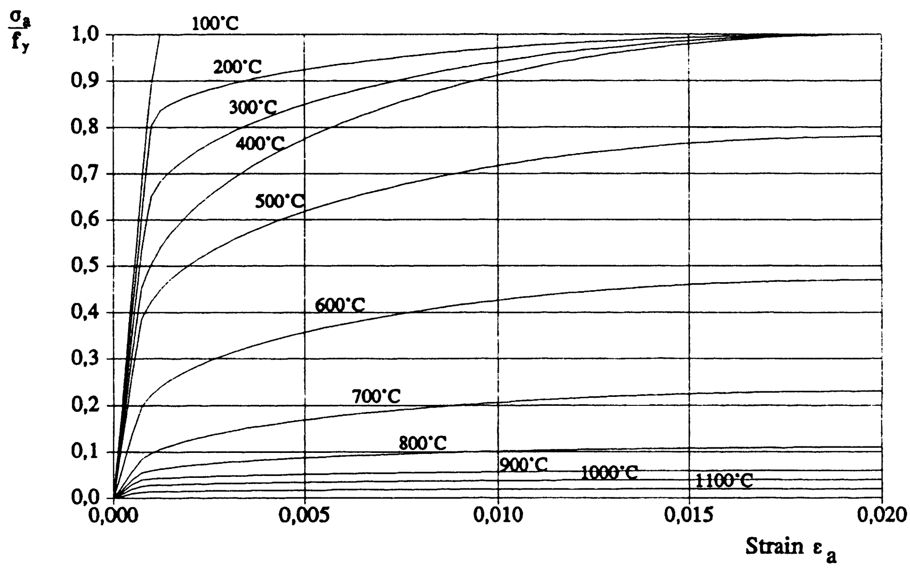


Figure A.1: Variation of stress-strain relationship with temperature for grade S 235 steel (strain-hardening not included)

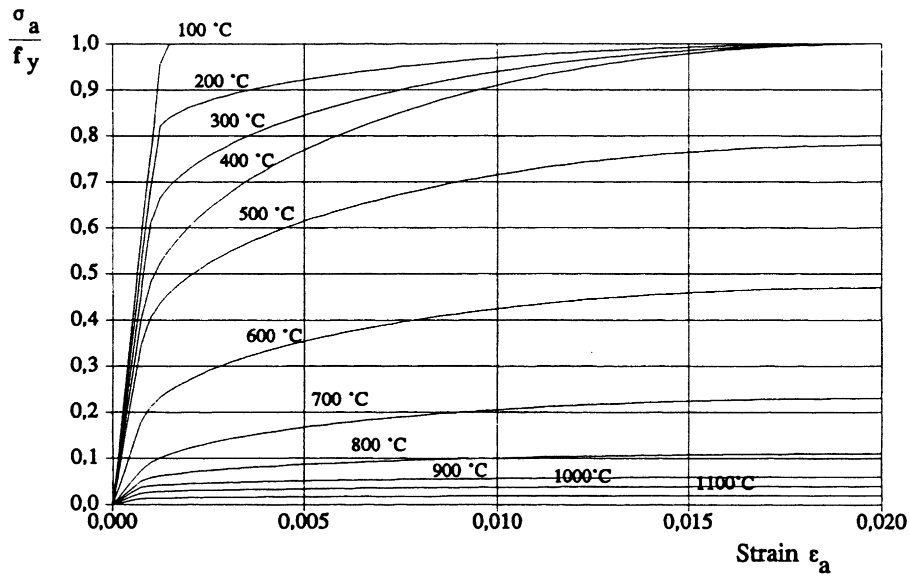


Figure A.2: Variation of stress-strain relationship with temperature for grade S 275 steel (strain-hardening not included)

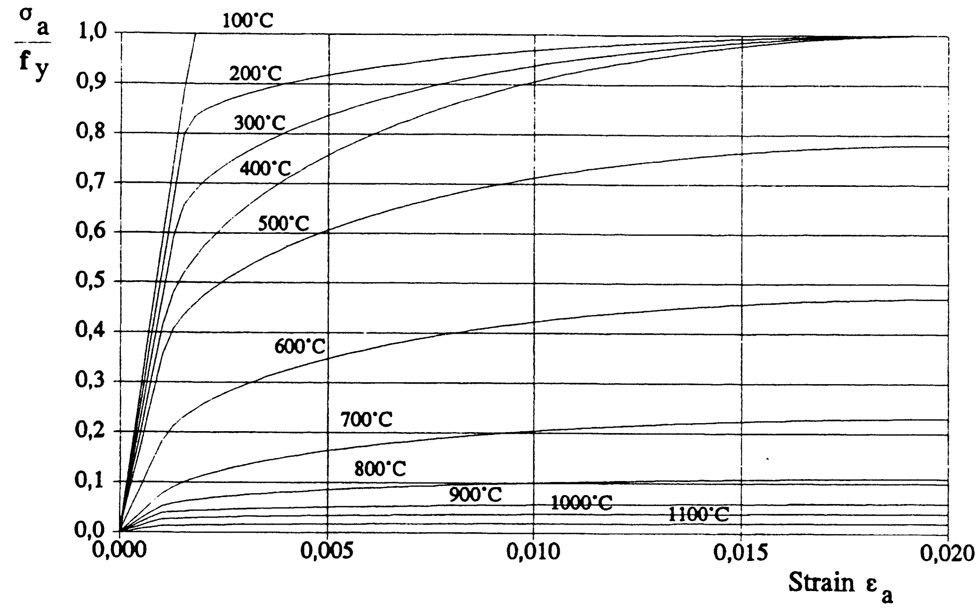


Figure A.3: Variation of stress-strain relationship with temperature for grade S 355 steel (strain-hardening not included)

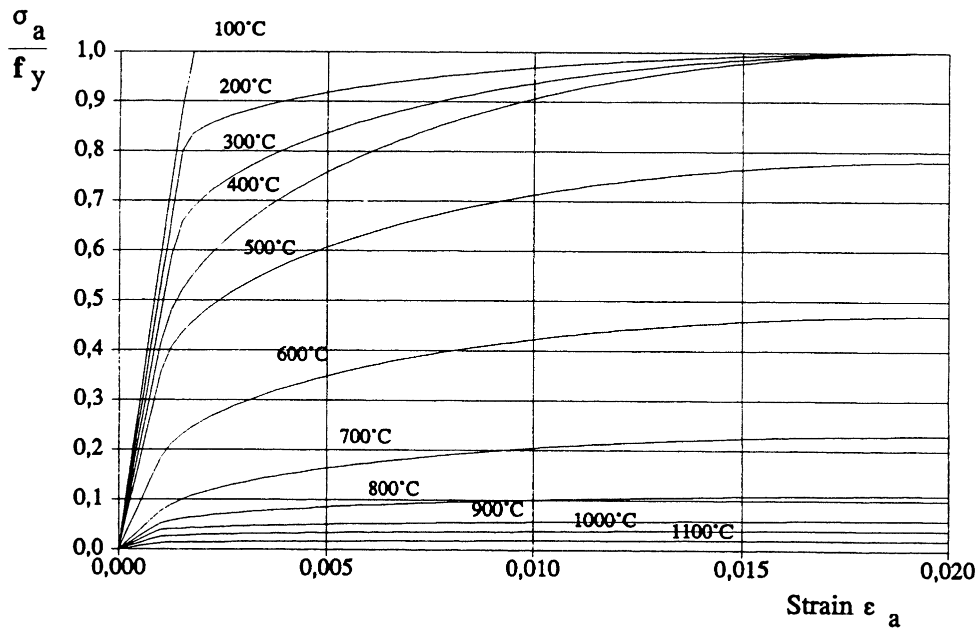


Figure A.4: Variation of stress-strain relationship with temperature for grade S 460 steel (strain-hardening not included)

Table A.1: Stress-strain relationship at elevated temperatures for grade S 235 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,447	0,402	0,357	0,313	0,268	0,139	0,058	0,040
0,0010	0,894	0,804	0,652	0,505	0,424	0,223	0,097	0,060
0,0015	1,000	0,849	0,705	0,569	0,470	0,254	0,113	0,066
0,0020	1,000	0,867	0,738	0,614	0,502	0,276	0,125	0,071
0,0025	1,000	0,880	0,763	0,650	0,528	0,295	0,135	0,074
0,0030	1,000	0,892	0,785	0,681	0,551	0,310	0,143	0,078
0,0035	1,000	0,901	0,804	0,708	0,570	0,324	0,151	0,080
0,0040	1,000	0,910	0,821	0,733	0,588	0,336	0,157	0,083
0,0045	1,000	0,917	0,836	0,755	0,604	0,347	0,163	0,085
0,0050	1,000	0,924	0,849	0,775	0,618	0,357	0,169	0,087
0,0055	1,000	0,931	0,862	0,794	0,632	0,367	0,174	0,089
0,0060	1,000	0,937	0,873	0,811	0,644	0,375	0,179	0,091
0,0065	1,000	0,942	0,884	0,827	0,656	0,383	0,183	0,092
0,0070	1,000	0,947	0,894	0,842	0,666	0,391	0,187	0,094
0,0075	1,000	0,952	0,903	0,856	0,676	0,397	0,191	0,095
0,0080	1,000	0,956	0,912	0,868	0,685	0,404	0,194	0,097
0,0085	1,000	0,960	0,920	0,880	0,694	0,410	0,197	0,098
0,0090	1,000	0,964	0,928	0,892	0,702	0,416	0,201	0,099
0,0095	1,000	0,967	0,935	0,902	0,710	0,421	0,203	0,100
0,0100	1,000	0,971	0,941	0,912	0,717	0,426	0,206	0,101
0,0110	1,000	0,977	0,953	0,930	0,730	0,435	0,211	0,103
0,0120	1,000	0,982	0,964	0,945	0,741	0,443	0,215	0,104
0,0130	1,000	0,986	0,972	0,959	0,750	0,449	0,219	0,106
0,0140	1,000	0,990	0,980	0,970	0,758	0,455	0,222	0,107
0,0150	1,000	0,993	0,986	0,979	0,765	0,460	0,224	0,108
0,0160	1,000	0,996	0,991	0,987	0,771	0,463	0,226	0,109
0,0170	1,000	0,998	0,995	0,993	0,775	0,466	0,228	0,109
0,0180	1,000	0,999	0,997	0,997	0,778	0,468	0,229	0,110
0,0190	1,000	1,000	0,999	0,999	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Table A.2: Stress-strain relationship at elevated temperatures for grade S 275 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,0000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,382	0,344	0,305	0,267	0,229	0,118	0,050	0,034
0,0010	0,764	0,687	0,611	0,482	0,407	0,212	0,091	0,058
0,0015	1,000	0,840	0,691	0,553	0,459	0,247	0,109	0,065
0,0020	1,000	0,861	0,728	0,602	0,494	0,270	0,122	0,070
0,0025	1,000	0,876	0,756	0,640	0,522	0,290	0,132	0,074
0,0030	1,000	0,888	0,779	0,672	0,545	0,306	0,141	0,077
0,0035	1,000	0,898	0,798	0,701	0,565	0,320	0,148	0,080
0,0040	1,000	0,907	0,816	0,726	0,583	0,333	0,155	0,082
0,0045	1,000	0,915	0,831	0,749	0,600	0,344	0,161	0,085
0,0050	1,000	0,922	0,845	0,770	0,615	0,354	0,167	0,087
0,0055	1,000	0,929	0,858	0,789	0,628	0,364	0,172	0,089
0,0060	1,000	0,935	0,870	0,806	0,641	0,373	0,177	0,090
0,0065	1,000	0,941	0,881	0,823	0,653	0,381	0,182	0,092
0,0070	1,000	0,946	0,892	0,838	0,664	0,389	0,186	0,094
0,0075	1,000	0,950	0,901	0,852	0,674	0,396	0,190	0,095
0,0080	1,000	0,955	0,910	0,865	0,683	0,402	0,193	0,096
0,0085	1,000	0,959	0,918	0,878	0,692	0,409	0,197	0,098
0,0090	1,000	0,963	0,926	0,889	0,701	0,414	0,200	0,099
0,0095	1,000	0,967	0,933	0,900	0,708	0,420	0,203	0,102
0,0100	1,000	0,970	0,940	0,910	0,716	0,425	0,205	0,102
0,0110	1,000	0,976	0,952	0,928	0,729	0,434	0,210	0,104
0,0120	1,000	0,981	0,963	0,944	0,740	0,442	0,215	0,105
0,0130	1,000	0,986	0,972	0,958	0,750	0,449	0,218	0,107
0,0140	1,000	0,990	0,980	0,969	0,758	0,455	0,222	0,108
0,0150	1,000	0,993	0,986	0,979	0,765	0,459	0,224	0,108
0,0160	1,000	0,996	0,991	0,985	0,769	0,462	0,226	0,109
0,0170	1,000	0,997	0,995	0,992	0,775	0,466	0,228	0,110
0,0180	1,000	0,999	0,998	0,997	0,778	0,468	0,229	0,110
0,0190	1,000	1,000	0,999	0,999	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Table A.3: Stress-strain relationship at elevated temperatures for grade S 355 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,0000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,296	0,266	0,237	0,207	0,177	0,092	0,038	0,027
0,0010	0,592	0,532	0,473	0,414	0,355	0,183	0,077	0,052
0,0015	0,887	0,799	0,657	0,520	0,435	0,230	0,100	0,062
0,0020	1,000	0,848	0,706	0,576	0,476	0,258	0,114	0,067
0,0025	1,000	0,866	0,739	0,619	0,507	0,279	0,126	0,072
0,0030	1,000	0,880	0,765	0,654	0,532	0,296	0,135	0,075
0,0035	1,000	0,892	0,786	0,685	0,554	0,312	0,144	0,078
0,0040	1,000	0,902	0,805	0,712	0,574	0,325	0,151	0,081
0,0045	1,000	0,910	0,822	0,736	0,591	0,337	0,158	0,083
0,0050	1,000	0,918	0,837	0,758	0,607	0,348	0,164	0,086
0,0055	1,000	0,925	0,851	0,778	0,621	0,359	0,169	0,088
0,0060	1,000	0,932	0,864	0,797	0,635	0,368	0,174	0,090
0,0065	1,000	0,938	0,876	0,814	0,647	0,377	0,179	0,091
0,0070	1,000	0,943	0,886	0,830	0,659	0,385	0,183	0,093
0,0075	1,000	0,948	0,896	0,845	0,669	0,399	0,187	0,094
0,0080	1,000	0,953	0,906	0,859	0,679	0,406	0,191	0,096
0,0085	1,000	0,957	0,915	0,872	0,689	0,412	0,195	0,097
0,0090	1,000	0,961	0,923	0,884	0,697	0,417	0,198	0,098
0,0095	1,000	0,965	0,930	0,896	0,705	0,423	0,201	0,099
0,0100	1,000	0,969	0,937	0,906	0,713	0,428	0,204	0,101
0,0110	1,000	0,975	0,950	0,925	0,726	0,437	0,209	0,102
0,0120	1,000	0,981	0,961	0,942	0,738	0,441	0,214	0,104
0,0130	1,000	0,985	0,971	0,956	0,748	0,448	0,218	0,106
0,0140	1,000	0,989	0,979	0,968	0,757	0,454	0,221	0,107
0,0150	1,000	0,993	0,985	0,978	0,764	0,459	0,224	0,108
0,0160	1,000	0,995	0,991	0,986	0,770	0,463	0,226	0,109
0,0170	1,000	0,997	0,995	0,992	0,774	0,466	0,228	0,109
0,0180	1,000	0,999	0,998	0,997	0,778	0,468	0,229	0,110
0,0190	1,000	1,000	0,999	0,999	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Table A.4: Stress-strain relationship at elevated temperatures for grade S 460 steel

Strain	Effective yield strength at elevated temperature, relative to yield strength at 20 °C. $k_{y,\theta} = f_{y,\theta}/f_y$							
	Steel temperature θ_a [°C]							
	100	200	300	400	500	600	700	800
0,0000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000
0,0005	0,228	0,205	0,183	0,160	0,137	0,071	0,030	0,021
0,0010	0,457	0,411	0,365	0,320	0,274	0,142	0,059	0,041
0,0015	0,685	0,616	0,548	0,465	0,395	0,205	0,087	0,057
0,0020	0,913	0,815	0,669	0,537	0,449	0,239	0,104	0,064
0,0025	1,000	0,850	0,712	0,587	0,485	0,263	0,117	0,069
0,0030	1,000	0,868	0,743	0,627	0,514	0,283	0,127	0,073
0,0035	1,000	0,882	0,769	0,661	0,538	0,300	0,137	0,076
0,0040	1,000	0,893	0,790	0,691	0,560	0,315	0,145	0,079
0,0045	1,000	0,903	0,809	0,718	0,579	0,328	0,152	0,082
0,0050	1,000	0,912	0,825	0,742	0,596	0,340	0,158	0,084
0,0055	1,000	0,920	0,841	0,764	0,611	0,351	0,164	0,086
0,0060	1,000	0,927	0,854	0,784	0,626	0,361	0,170	0,088
0,0065	1,000	0,933	0,867	0,802	0,639	0,370	0,175	0,090
0,0070	1,000	0,939	0,879	0,819	0,651	0,379	0,180	0,092
0,0075	1,000	0,945	0,890	0,835	0,663	0,387	0,184	0,094
0,0080	1,000	0,950	0,900	0,850	0,673	0,395	0,188	0,095
0,0085	1,000	0,954	0,909	0,864	0,683	0,402	0,192	0,096
0,0090	1,000	0,959	0,918	0,877	0,692	0,408	0,196	0,098
0,0095	1,000	0,963	0,926	0,889	0,701	0,414	0,199	0,099
0,0100	1,000	0,967	0,933	0,900	0,709	0,420	0,202	0,100
0,0110	1,000	0,974	0,947	0,921	0,723	0,430	0,208	0,102
0,0120	1,000	0,979	0,959	0,938	0,736	0,439	0,213	0,104
0,0130	1,000	0,984	0,969	0,953	0,747	0,446	0,217	0,105
0,0140	1,000	0,989	0,977	0,966	0,756	0,453	0,221	0,107
0,0150	1,000	0,992	0,984	0,977	0,763	0,458	0,223	0,108
0,0160	1,000	0,995	0,990	0,985	0,769	0,462	0,226	0,109
0,0170	1,000	0,997	0,994	0,992	0,774	0,466	0,228	0,109
0,0180	1,000	0,999	0,998	0,996	0,777	0,468	0,229	0,110
0,0190	1,000	0,999	0,999	1,000	0,779	0,470	0,230	0,110
0,0200	1,000	1,000	1,000	1,000	0,780	0,470	0,230	0,110

Annex B [normative]

Strain-hardening of steel at elevated temperatures

(1) For temperatures below 400 °C, the alternative strain-hardening option mentioned in 3.2.1(5) may be used as follows:

- for $0{,}02 < \varepsilon < 0{,}04$:

$$\sigma_a = 50(f_{u,\theta} - f_{y,\theta})\varepsilon + 2f_{y,\theta} - f_{u,\theta} \qquad \text{(B.1a)}$$

- for $0{,}04 \leq \varepsilon \leq 0{,}15$:

$$\sigma_a = f_{u,\theta} \qquad \text{(B.1b)}$$

- for $0{,}15 < \varepsilon < 0{,}20$:

$$\sigma_a = f_{u,\theta}[1 - 20(\varepsilon - 0{,}15)] \qquad \text{(B.1c)}$$

- for $\varepsilon \geq 0{,}20$:

$$\sigma_a = 0{,}00 \qquad \text{(B.1d)}$$

where:

$f_{u,\theta}$ is the ultimate strength at elevated temperature, allowing for strain-hardening.

(2) The alternative stress-strain relationship for steel, allowing for strain hardening, is illustrated in figure B.1.

(3) The ultimate strength at elevated temperature, allowing for strain hardening, should be determined as follows:

- for $\theta_a < 300$ °C:

$$f_{u,\theta} = 1{,}25 f_{y,\theta} \qquad \text{(B.2a)}$$

- for 300 °C $\leq \theta_a <$ 400 °C:

$$f_{u,\theta} = f_{y,\theta}(2 - 0{,}0025\,\theta a) \qquad \text{(B.2b)}$$

- for $\theta_a \geq 400$ °C:

$$f_{u,\theta} = f_{y,\theta} \qquad \text{(B.2c)}$$

(4) The variation of the alternative stress-strain relationship with temperature is illustrated in figure B.2.

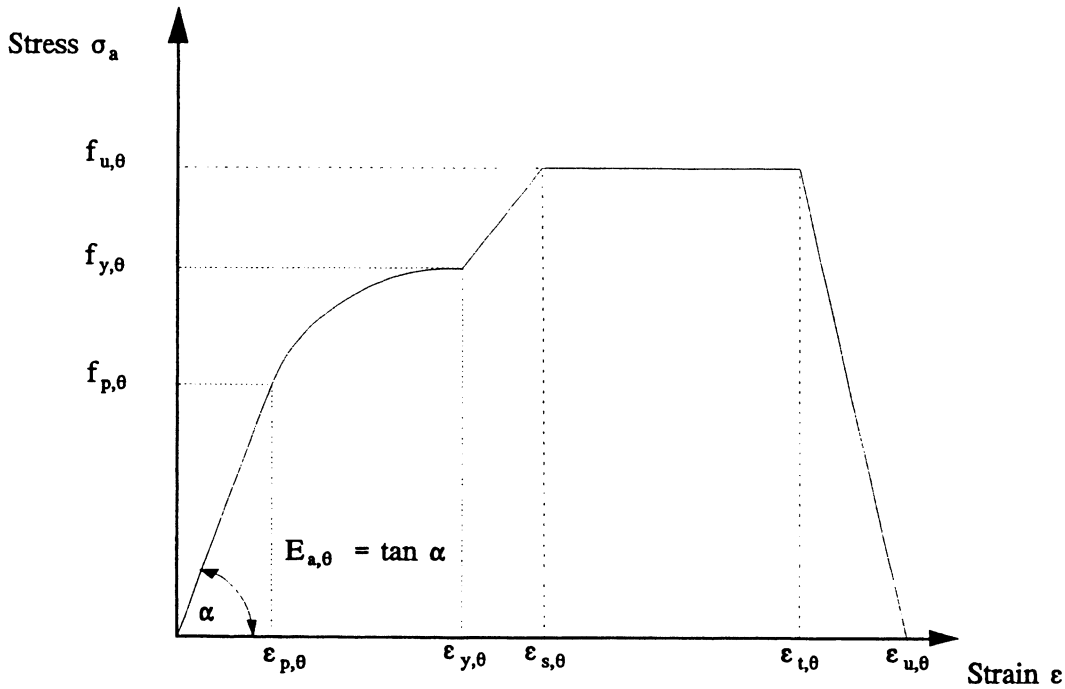


Figure B.1: Alternative stress-strain relationship for steel allowing for strain-hardening

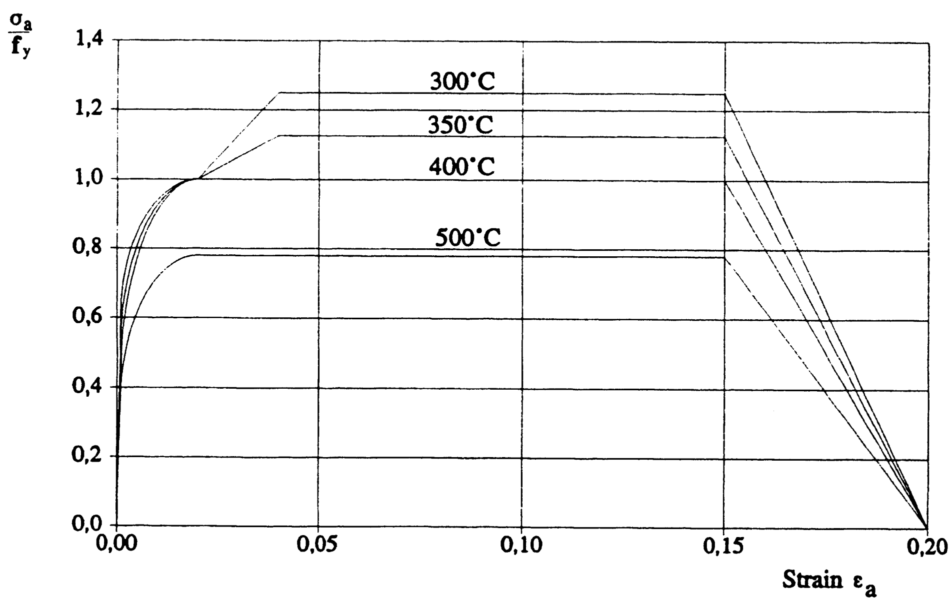


Figure B.2: Alternative stress-strain relationships for steel at elevated temperatures, allowing for strain hardening

Annex C [normative]

Heat transfer to external steelwork.

C.1 General

C.1.1 Basis

(1) In this annex C, the fire compartment is assumed to be confined to one storey only. All windows or other similar openings in the fire compartment are assumed to be rectangular.

(2) Annex C of ENV 1991-2-2 should be used to determine the temperature of the compartment fire, the dimensions and temperatures of the flames projecting from the openings, and the radiation and convection parameters.

(3) A distinction should be made between members not engulfed in flame and members engulfed in flame, depending on their locations relative to the openings in the walls of the fire compartment.

(4) A member that is not engulfed in flame should be assumed to receive radiative heat transfer from all the openings in that side of the fire compartment and from the flames projecting from all these openings.

(5) A member that is engulfed in flame should be assumed to receive convective heat transfer from the engulfing flame, plus radiative heat transfer from the engulfing flame and from the fire compartment opening from which it projects. The radiative heat transfer from other flames and from other openings may be neglected.

C.1.2 Member dimensions and faces

(1) The convention used for the dimensions d_1 and d_2 of a member and the notation used to identify its four faces are indicated in figure C.1.

C.1.3 Heat balance

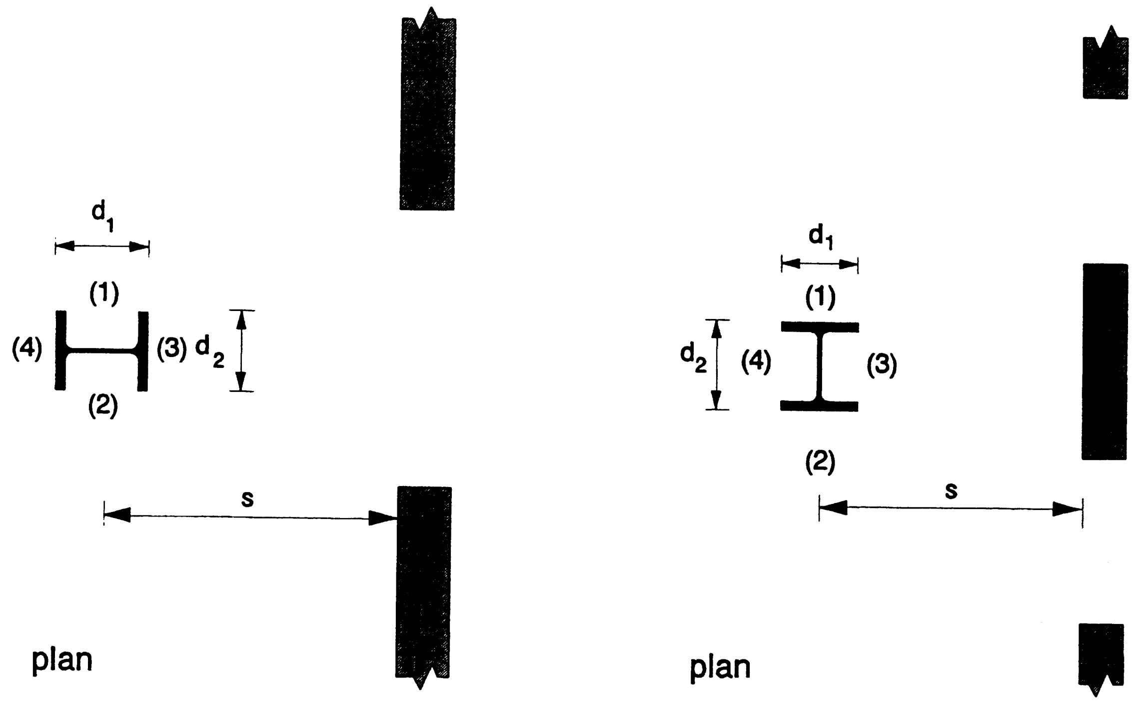
(1) For a member not engulfed in flame, the average temperature of the steel member T_m [K] should be determined from the solution of the following heat balance:

$$\sigma T_m^4 + \alpha T_m = \Sigma I_z + \Sigma I_f + 293\alpha \tag{C.1}$$

where:

σ	is	the Stefan Boltzmann constant [$56{,}7 \times 10^{-12}$ kW/m^{2}K^4];
α	is	the convective heat transfer coefficient [kW/m^{2}K];
I_z	is	the radiative heat flux from a flame [kW/m^2];
I_f	is	the radiative heat flux from an opening [kW/m^2].

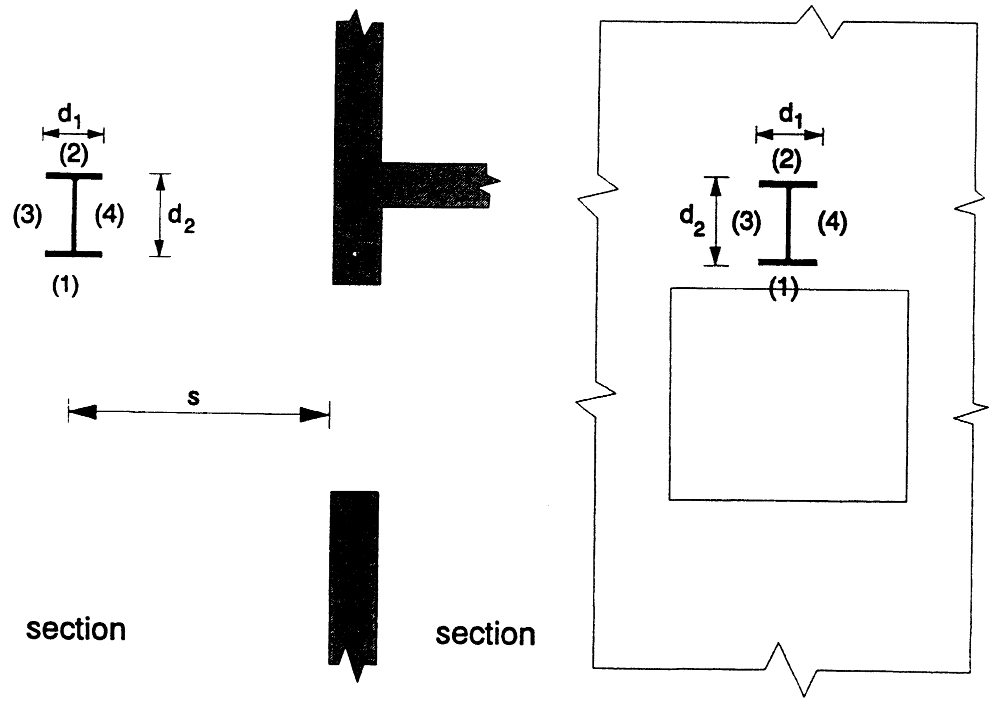
(2) The convective heat transfer coefficient α should be obtained from annex C of ENV 1991-2-2 for the 'no forced draught' or the 'forced draught' condition as appropriate, using an effective cross-sectional dimension $d = (d_1 + d_2)/2$.



1) Column opposite opening

2) Column between openings

a) Columns



1) Beam parallel to wall

2) Beam perpendicular to wall

b) Beams

Figure C.1: Member dimensions and faces

(3) For a member engulfed in flame, the average temperature of the steel member T_m [K] should be determined from the solution of the following heat balance:

$$\sigma T_m^4 + \alpha T_m = I_z + I_f + \alpha T_z \qquad \text{(C.2)}$$

where:

T_z	is	the flame temperature [K];
I_z	is	the radiative heat flux from the flame [kW/m^2];
I_f	is	the radiative heat flux from the corresponding opening [kW/m^2].

(4) The radiative heat flux I_z from flames should be determined according to the situation and type of member as follows:

- Columns not engulfed in flame: see C.2;
- Beams not engulfed in flame: see C.3;
- Columns engulfed in flame: see C.4;
- Beams fully or partially engulfed in flame: see C.5.

Other cases may be treated analogously, using appropriate adaptations of the treatments given in C.2 to C.5.

(5) The radiative heat flux I_f from an opening should be determined from:

$$I_f = \phi_f \varepsilon_f (1 - a_z) \sigma T_f^4 \qquad \text{(C.3)}$$

where:

ϕ_f	is	the overall configuration factor of the member for radiative heat transfer from that opening;
ε_f	is	the emissivity of the opening;
a_z	is	the absorptivity of the flames;
T_f	is	the temperature of the fire [K] from annex C of ENV 1991-2-2.

(6) The emissivity ε_f of an opening should be taken as unity, see annex C of ENV 1991-2-2.

(7) The absorptivity a_z of the flames should be determined from C.2 to C.5 as appropriate.

C.1.4 Overall configuration factors

(1) The overall configuration factor ϕ_f of a member for radiative heat transfer from an opening should be determined from:

$$\phi_f = \frac{(C_1 \phi_{f,1} + C_2 \phi_{f,2}) d_1 + (C_3 \phi_{f,3} + C_4 \phi_{f,4}) d_2}{(C_1 + C_2) d_1 + (C_3 + C_4) d_2} \tag{C.4}$$

where:

$\phi_{f,i}$ is the configuration factor of member face i for that opening, see annex D;

d_i is the cross-sectional dimension of member face i ;

C_i is the protection coefficient of member face i as follows:

- for a protected face: $C_i = 0$
- for an unprotected face: $C_i = 1$

(2) The configuration factor $\phi_{f,i}$ for a member face from which the opening is not visible should be taken as zero.

(3) The overall configuration factor ϕ_z of a member for radiative heat transfer from a flame should be determined from:

$$\phi_z = \frac{(C_1 \phi_{z,1} + C_2 \phi_{z,2}) d_1 + (C_3 \phi_{z,3} + C_4 \phi_{z,4}) d_2}{(C_1 + C_2) d_1 + (C_3 + C_4) d_2} \tag{C.5}$$

where:

$\phi_{z,i}$ is the configuration factor of member face i for that flame, see annex D.

(4) The configuration factors $\phi_{z,i}$ of individual member faces for radiative heat transfer from flames may be based on equivalent rectangular flame dimensions. The dimensions and locations of equivalent rectangles representing the front and sides of a flame for this purpose should be determined as given in C.2 for columns and C.3 for beams. For all other purposes, the flame dimensions from annex C of ENV 1991-2-2 should be used.

(5) The configuration factor $\phi_{z,i}$ for a member face from which the flame is not visible should be taken as zero.

(6) A member face may be protected by a heat screen, see 4.2.5.4. A member face that is immediately adjacent to the compartment wall may also be treated as protected, provided that there are no openings in that part of the wall. All other member faces should be treated as unprotected.

C.2 Column not engulfed in flame

C.2.1 Radiative heat transfer

(1) A distinction should be made between a column located opposite an opening and a column located between openings, see figure C.2.

(2) If the column is opposite an opening, see figure C.3, the radiative heat flux I_z from the flame should be determined from:

$$I_z = \phi_z \varepsilon_z \sigma T_z^4 \tag{C.6}$$

where:

ϕ_z	is	the overall configuration factor of the column for heat from the flame, see C.1.4;
ε_z	is	the emissivity of the flame, see C.2.2;
T_z	is	the flame temperature [K] from C.2.3.

(3) If the column is between openings, see figure C.4, the total radiative heat flux I_z from the flames on each side should be determined from:

$$I_z = (\phi_{z,m} \varepsilon_{z,m} + \phi_{z,n} \varepsilon_{z,n}) \sigma T_z^4 \tag{C.7}$$

where:

$\phi_{z,m}$	is	the overall configuration factor of the column for heat from flames on side m, see C.1.4;
$\phi_{z,n}$	is	the overall configuration factor of the column for heat from flames on side n, see C.1.4;
$\varepsilon_{z,m}$	is	the total emissivity of the flames on side m, see C.2.2;
$\varepsilon_{z,n}$	is	the total emissivity of the flames on side n, see C.2.2.

C.2.2 Flame emissivity

(1) If the column is opposite an opening, the flame emissivity ε_z should be determined from the expression for ε given in annex C of ENV 1991-2-2, using the flame thickness λ at the level of the top of the openings. Provided that there is no awning or balcony above the opening λ may be taken as follows:

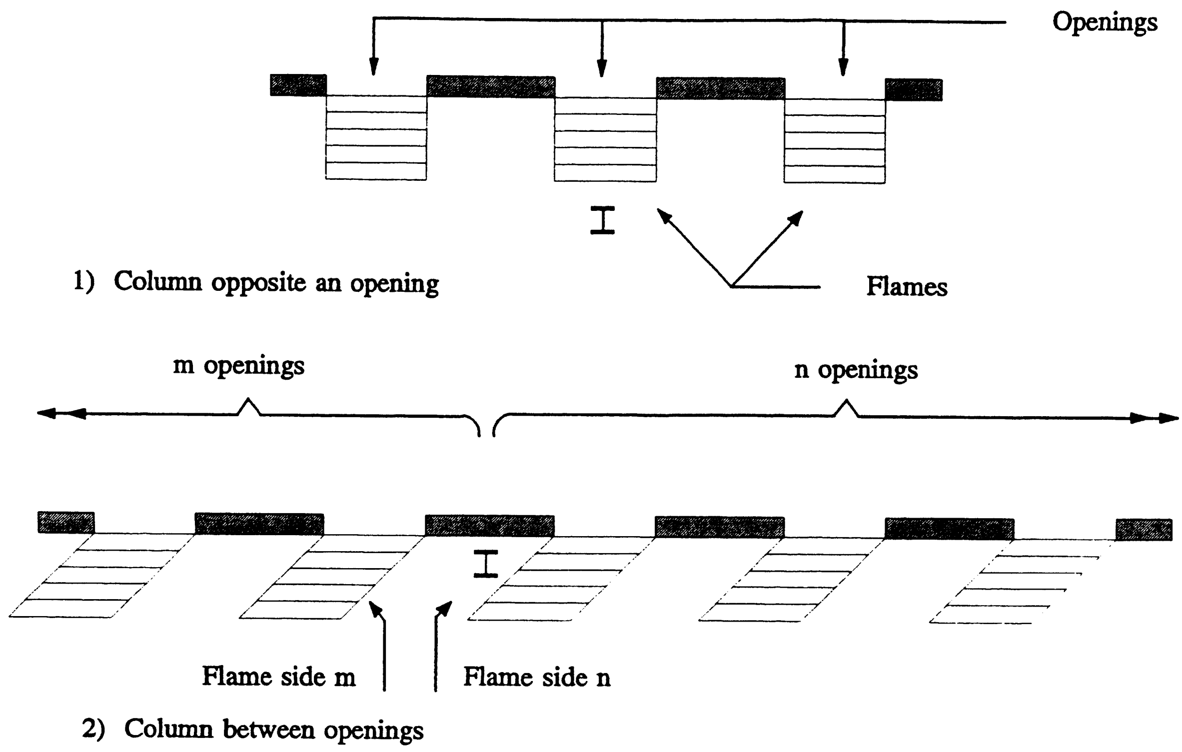
- for the 'no forced draught' condition:

$$\lambda = 2h/3 \tag{C.8a}$$

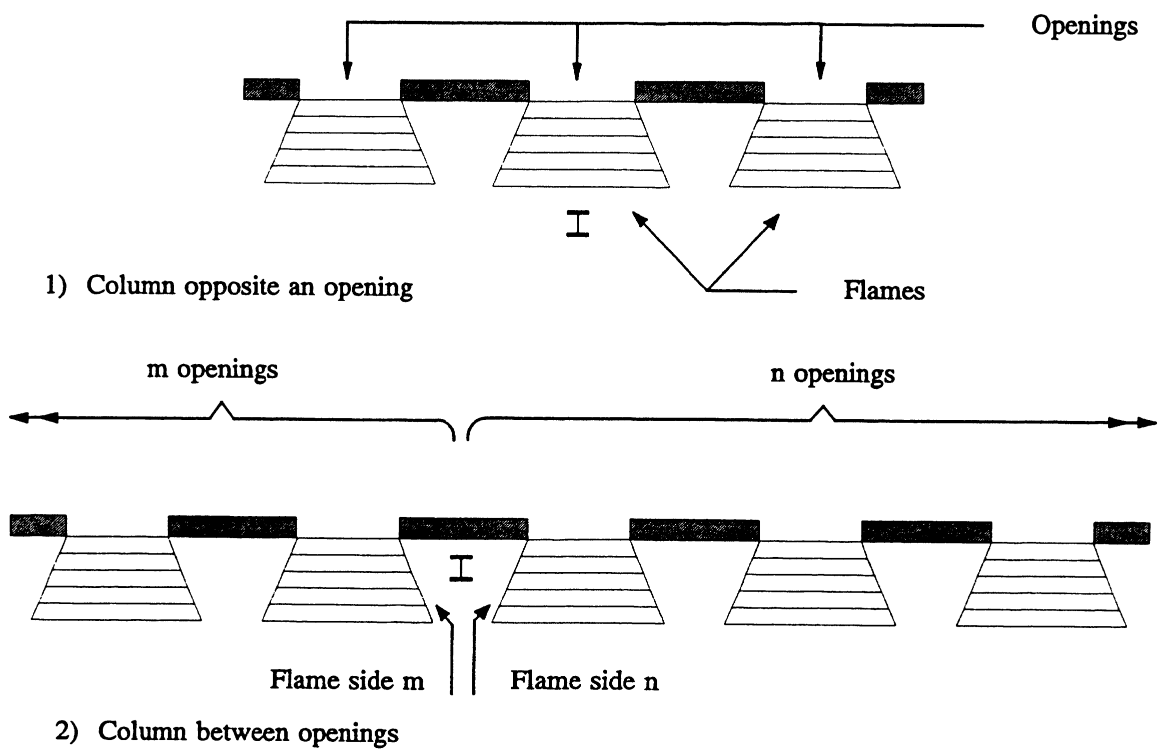
- for the 'forced draught' condition:

$$\lambda = x \quad \text{but } \lambda \leq hx/z \tag{C.8b}$$

where h, x and z are as given in annex C of ENV 1991-2-2.

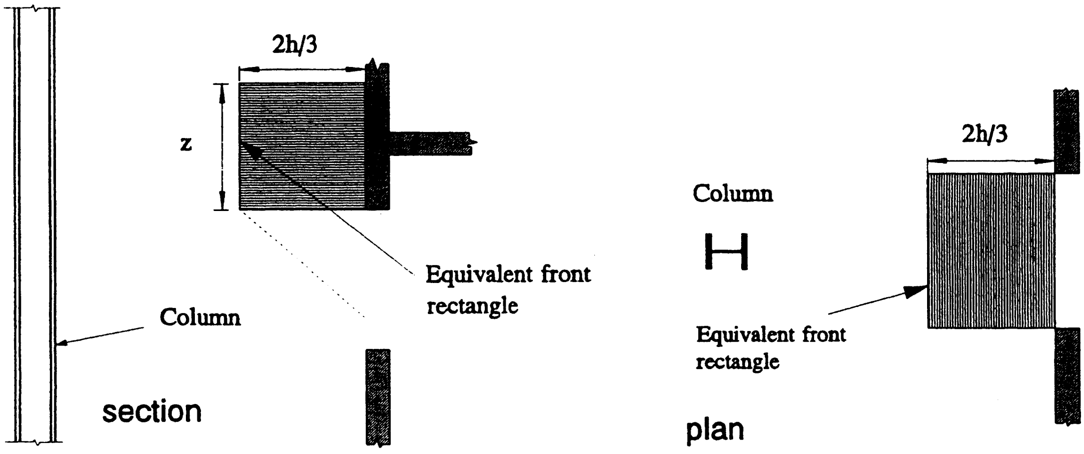


a) 'No forced draught' condition

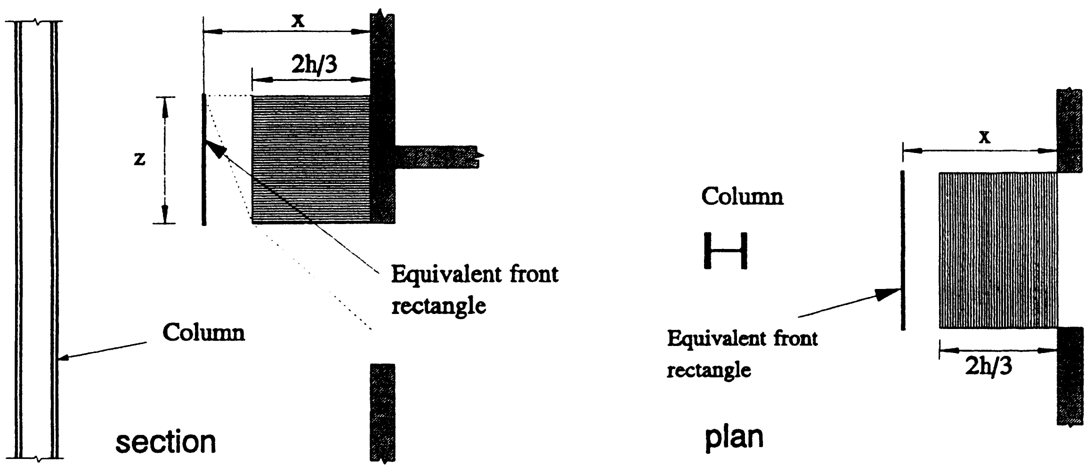


b) 'Forced draught' condition

Figure C.2: Column positions

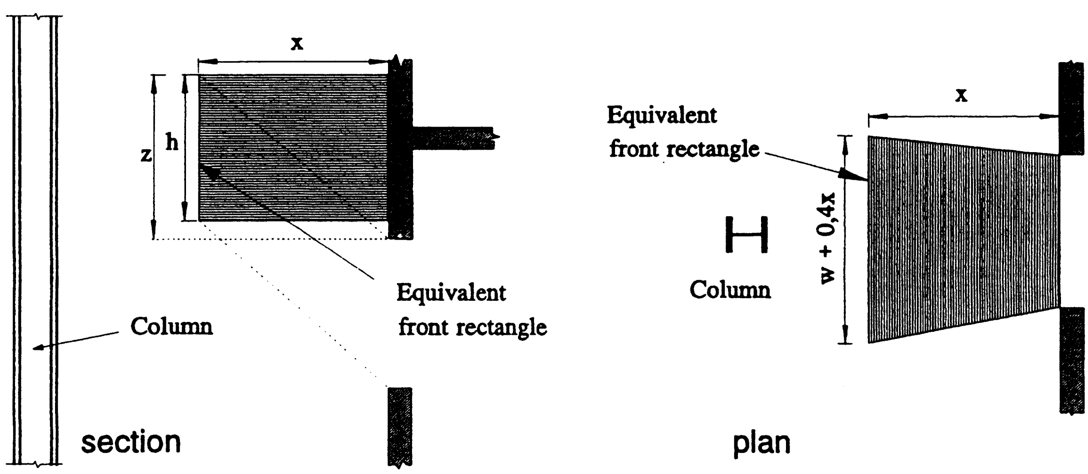


1) wall above and h < 1,25w



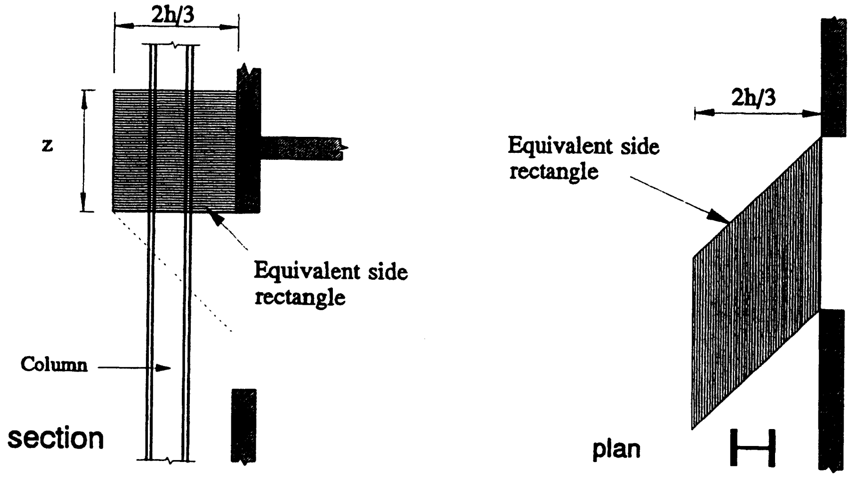
2) wall above and h > 1,25w or no wall above

a) **'No forced draught'**

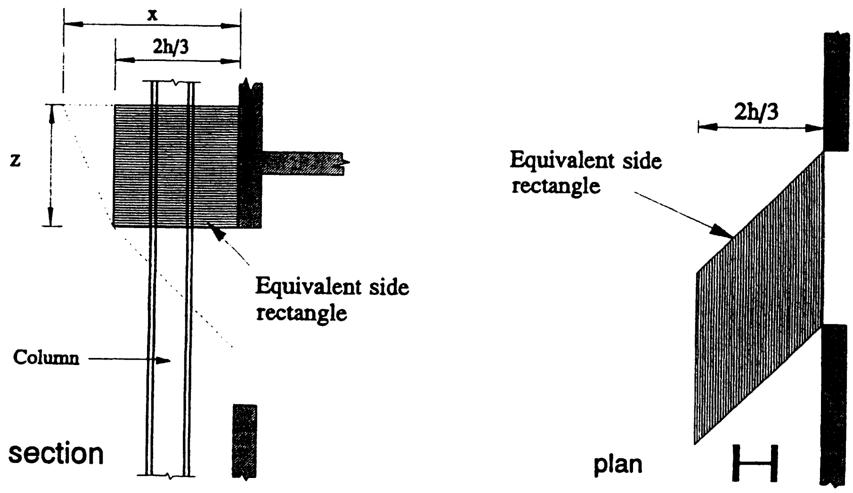


b) **'Forced draught'**

Figure C.3: Column opposite opening

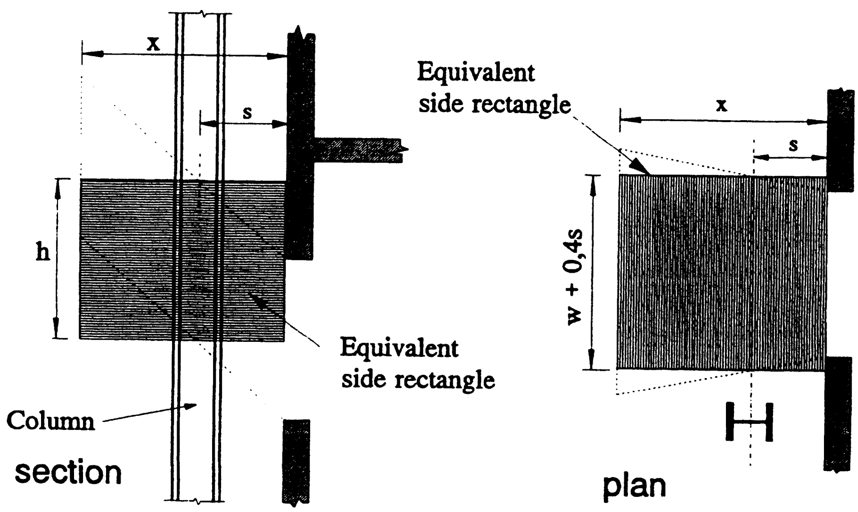


1) wall above and h < 1,25w



2) wall above and h > 1,25w or no wall above

a) 'No forced draught'



b) 'Forced draught'

Figure C.4: Column between openings

(2) If the column is between two openings, the total emissivities $\varepsilon_{z,m}$ and $\varepsilon_{z,n}$ of the flames on sides m and n should be determined from the expression for ε given in annex C of ENV 1991-2-2 using a value for the total flame thickness λ as follows:

- for side m: $$\lambda = \sum_{i=1}^{m} \lambda_i \quad \text{(C.9a)}$$

- for side n: $$\lambda = \sum_{i=1}^{n} \lambda_i \quad \text{(C.9b)}$$

where:

m is the number of openings on side m;

n is the number of openings on side n;

λ_i is the flame thickness for opening i.

(3) The flame thickness λ_i should be taken as follows:

- for the 'no forced draught' condition:

$$\lambda_i = w_i \quad \text{(C.10a)}$$

- for the 'forced draught' condition:

$$\lambda_i = w_i + 0{,}4s \quad \text{(C.10b)}$$

where:

w_i is the width of the opening;

s is the horizontal distance from the centreline of the column to the wall of the fire compartment, see figure C.1.

C.2.3 Flame temperature

(1) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex C of ENV 1991-2-2, for the 'no forced draught' condition or the 'forced draught' condition as appropriate, at a distance ℓ from the opening, measured along the flame axis, as follows:

- for the 'no forced draught' condition:

$$\ell = h/2 \quad \text{(C.11a)}$$

- for the 'forced draught' condition:

  - for a column opposite an opening:

$$\ell = 0 \quad \text{(C.11b)}$$

  - for a column between openings ℓ is the distance along the flame axis to a point at a horizontal distance s from the wall of the fire compartment. Provided that there is no awning or balcony above the opening:

$$\ell = sX/x \quad \text{(C.11c)}$$

where X and x are as given in annex C of ENV 1991-2-2.

C.2.4 Flame absorptivity

(1) For the 'no forced draught' condition, the flame absorptivity a_z should be taken as zero.

(2) For the 'forced draught' condition, the flame absorptivity a_z should be taken as equal to the emissivity ε_z of the relevant flame, see C.2.2.

C.3 Beam not engulfed in flame

C.3.1 Radiative heat transfer

(1) Throughout C.3 it is assumed that the level of the bottom of the beam is not below the level of the top of the openings in the fire compartment.

(2) A distinction should be made between a beam that is parallel to the external wall of the fire compartment and a beam that is perpendicular to the external wall of the fire compartment, see figure C.5.

(3) If the beam is parallel to the external wall of the fire compartment, the average temperature of the steel member T_m should be determined for a point in the length of the beam directly above the centre of the opening. For this case the radiative heat flux I_z from the flame should be determined from:

$$I_z = \phi_z \varepsilon_z \sigma T_z^4 \tag{C.12}$$

where:

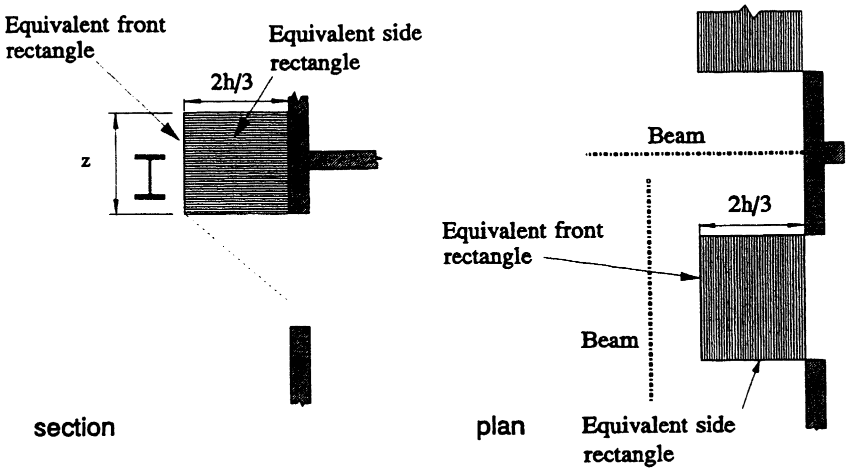
ϕ_z	is	the overall configuration factor for the flame directly opposite the beam, see C.1.4;
ε_z	is	the flame emissivity, see C.3.2;
T_z	is	the flame temperature from C.3.3 [K].

(4) If the beam is perpendicular to the external wall of the fire compartment, the average temperature in the beam should be determined at a series of points every 100 mm along the length of the beam. The average temperature of the steel member T_m should then be taken as the maximum of these values. For this case the radiative heat flux I_z from the flames should be determined from:

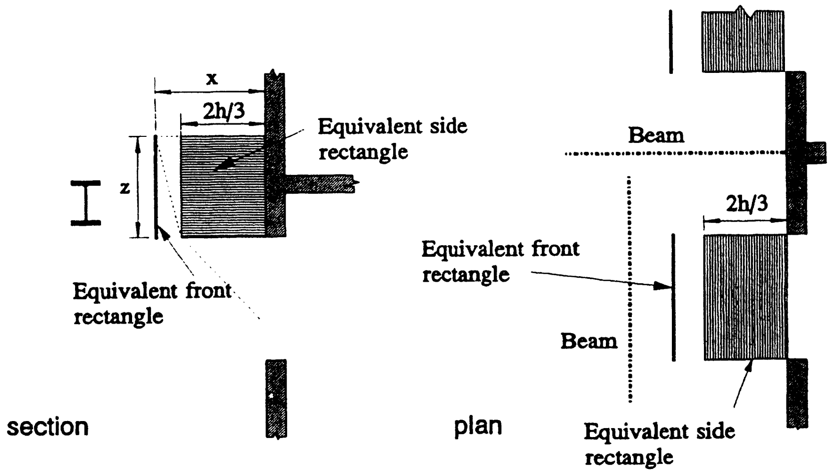
$$I_z = (\phi_{z,m} \varepsilon_{z,m} + \phi_{z,n} \varepsilon_{z,n}) \sigma T_z^4 \tag{C.13}$$

where:

$\phi_{z,m}$	is	the overall configuration factor of the beam for heat from flames on side m, see C.3.2;
$\phi_{z,n}$	is	the overall configuration factor of the beam for heat from flames on side n, see C.3.2;
$\varepsilon_{z,m}$	is	the total emissivity of the flames on side m, see C.3.3;
$\varepsilon_{z,n}$	is	the total emissivity of the flames on side n, see C.3.3;
T_z	is	the flame temperature [K], see C.3.4.

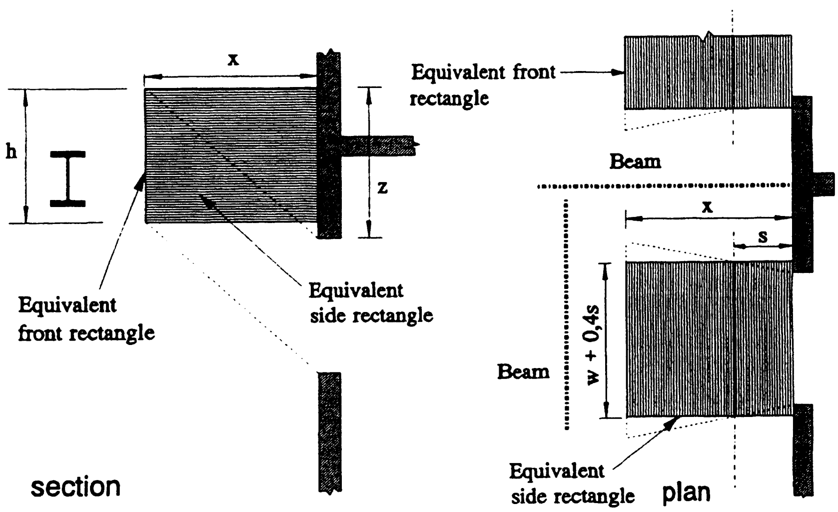


1) wall above and h < 1,25w



2) wall above and h > 1,25w or no wall above

a) 'No forced draught'



b) 'Forced draught'

Figure C.5: Beam not engulfed in flame

C.3.2 Flame emissivity

(1) If the beam is parallel to the external wall of the fire compartment, above an opening, the flame emissivity ε_z should be determined from the expression for ε given in annex C of ENV 1991-2-2, using a value for the flame thickness λ at the level of the top of the openings. Provided that there is no awning or balcony above the opening λ may be taken as follows:

- for the 'no forced draught' condition:

$$\lambda = 2h/3 \tag{C.14a}$$

- for the 'forced draught' condition:

$$\lambda = x \quad \text{but} \quad \lambda \leq hx/z \tag{C.14b}$$

where h, x and z are as given in annex C of ENV 1991-2-2

(2) If the beam is perpendicular to the external wall of the fire compartment, between two openings, the total emissivities $\varepsilon_{z,m}$ and $\varepsilon_{z,n}$ of the flames on sides m and n should be determined from the expression for ε given in annex C of ENV 1991-2-2 using a value for the flame thickness λ as follows:

- for side m:

$$\lambda = \sum_{i=1}^{m} \lambda_i \tag{C.15a}$$

- for side n:

$$\lambda = \sum_{i=1}^{n} \lambda_i \tag{C.15b}$$

where:

m is the number of openings on side m;

n is the number of openings on side n;

λ_i is the width of opening i.

(3) The flame thickness λ_i should be taken as follows:

- for the 'no forced draught' condition:

$$\lambda_i = w_i \tag{C.16a}$$

- for the 'forced draught' condition:

$$\lambda_i = w_i + 0{,}4s \tag{C.16b}$$

where:

w_i is the width of the opening;

s is the horizontal distance from the wall of the fire compartment to the point under consideration on the beam, see figure C.5.

C.3.3 Flame temperature

(1) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex C of ENV 1991-2-2, for the 'no forced draught' or 'forced draught' condition as appropriate, at a distance ℓ from the opening, measured along the flame axis, as follows:

- for the 'no forced draught' condition:

$$\ell = h/2 \tag{C.17a}$$

- for the 'forced draught' condition:
  - for a beam parallel to the external wall of the fire compartment, above an opening:

$$\ell = 0 \tag{C.17b}$$

  - for a beam perpendicular to the external wall of the fire compartment, between openings ℓ is the distance along the flame axis to a point at a horizontal distance s from the wall of the fire compartment. Provided that there is no awning or balcony above the opening:

$$\ell = sX/x \tag{C.17c}$$

where X and x are as given in annex C of ENV 1991-2-2.

C.3.4 Flame absorptivity

(1) For the 'no forced draught' condition, the flame absorptivity a_z should be taken as zero.

(2) For the 'forced draught' condition, the flame absorptivity a_z should be taken as equal to the emissivity ε_z of the relevant flame, see C.3.2.

C.4 Column engulfed in flame

(1) The radiative heat flux I_z from the flames should be determined from:

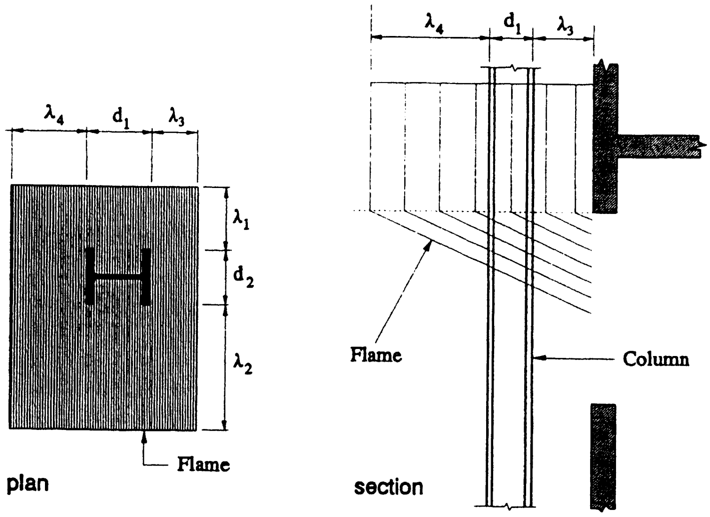
$$I_z = \frac{(I_{z,1} + I_{z,2})d_1 + (I_{z,3} + I_{z,4})d_2}{2(d_1 + d_2)} \tag{C.18}$$

with:

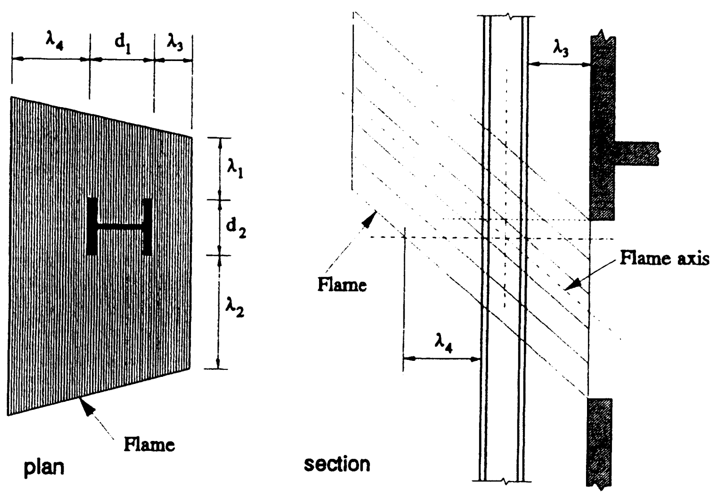
$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_z^4$$
$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_z^4$$
$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma T_o^4$$
$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma T_z^4$$

where:

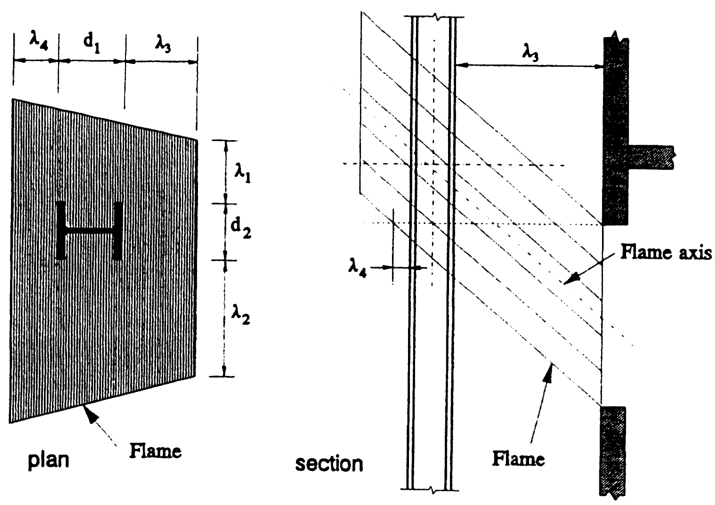
$I_{z,i}$	is	the radiative heat flux from the flame to column face i;
$\varepsilon_{z,i}$	is	the emissivity of the flames with respect to face i of the column;
i	is	the column face indicator (1), (2), (3) or (4);
C_i	is	the protection coefficient of member face i, see C.1.4;
T_z	is	the flame temperature [K];
T_o	is	the flame temperature at the opening [K] from annex C of ENV 1991-2-2.



a) **'No forced draught' condition**



1) Flame axis intersects column axis below top of opening



2) Flame axis intersects column axis above top of opening

b) **'Forced draught' condition**

Figure C.6: Column engulfed in flame

(2) The emissivity of the flames $\varepsilon_{z,i}$ for each of the faces 1, 2, 3 and 4 of the column should be determined from the expression for ε given in annex C of ENV 1991-2-2, using a flame thickness λ equal to the dimension λ_i indicated in figure C.6 corresponding to face i of the column.

(3) For the 'no forced draught' condition the values of λ_i at the level of the top of the opening should be used, see figure C.6(a).

(4) For the 'forced draught' condition, if the level of the intersection of the flame axis and the column centreline is below the level of the top of the opening, the values of λ_i at the level of the intersection should be used, see figure C.6(b)(1). Otherwise the values of λ_i at the level of the top of the opening should be used, see figure C.6(b)(2), except that if $\lambda_4 < 0$ at this level, the values at the level where $\lambda_4 = 0$ should be used.

(5) The flame temperature T_z should be taken as the temperature at the flame axis obtained from the expression for T_z given in annex C of ENV 1991-2-2 for the 'no forced draught' or 'forced draught' condition as appropriate, at a distance ℓ from the opening, measured along the flame axis, as follows:

- for the 'no forced draught' condition:

$$\ell = h/2 \tag{C.19a}$$

- for the 'forced draught' condition, ℓ is the distance along the flame axis to the level where λ_i is measured. Provided that there is no balcony or awning above the opening:

$$\ell = (\lambda_3 + 0{,}5 d_1) X/x \quad \text{but } \ell \leq 0{,}5 hX/z \tag{C.19b}$$

where h, X, x and z are as given in annex C of ENV 1991-2-2.

(6) The absorptivity a_z of the flames should be determined from:

$$a_z = \frac{\varepsilon_{z,1} + \varepsilon_{z,2} + \varepsilon_{z,3}}{3} \tag{C.20}$$

where $\varepsilon_{z,1}$, $\varepsilon_{z,2}$ and $\varepsilon_{z,3}$ are the emissivities of the flame for column faces 1, 2, and 3.

C.5 Beam fully or partially engulfed in flame

C.5.1 Radiative heat transfer

C.5.1.1 General

(1) Throughout C.5 it is assumed that the level of the bottom of the beam is not below the level of the top of the adjacent openings in the fire compartment.

(2) A distinction should be made between a beam that is parallel to the external wall of the fire compartment and a beam that is perpendicular to the external wall of the fire compartment, see figure C.7.

(3) If the beam is parallel to the external wall of the fire compartment, its average temperature T_m should be determined for a point in the length of the beam directly above the centre of the opening.

(4) If the beam is perpendicular to the external wall of the fire compartment, the value of the average temperature should be determined at a series of points every 100 mm along the length of the beam. The maximum of these values should then be adopted as the average temperature of the steel member T_m .

(5) The radiative heat flux I_z from the flame should be determined from:

$$I_z = \frac{(I_{z1} + I_{z2})\, d_1 + (I_{z3} + I_{z4})\, d_2}{2\,(d_1 + d_2)} \tag{C.21}$$

where:

$I_{z,i}$ is the radiative heat flux from the flame to beam face i;

i is the beam face indicator (1), (2), (3) or (4).

C.5.1.2 'No forced draught' condition

(1) For the 'no forced draught' condition, a distinction should be made between those cases where the top of the flame is above the level of the top of the beam and those where it is below this level.

(2) If the top of the flame is above the level of the top of the beam:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{C.22a}$$

$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \tag{C.22b}$$

$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.22c}$$

$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.22d}$$

where:

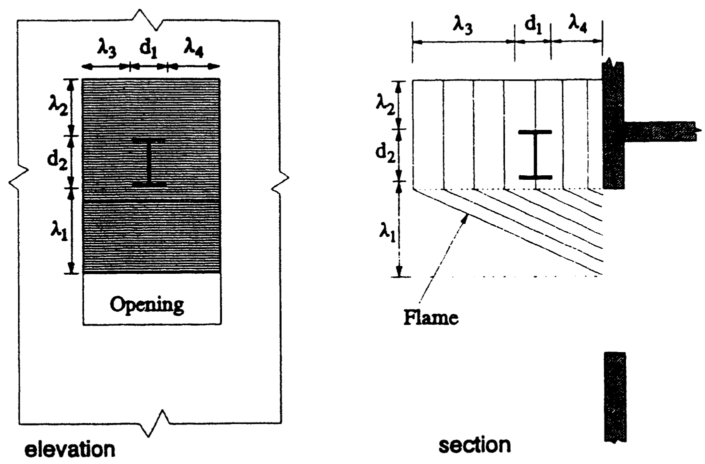
$\varepsilon_{z,i}$ is the emissivity of the flame with respect to face i of the beam, see C.5.2;

T_o is the temperature at the opening [K] from annex C of ENV 1991-2-2;

$T_{z,1}$ is the flame temperature [K] from annex C of ENV 1991-2-2, level with the bottom of the beam;

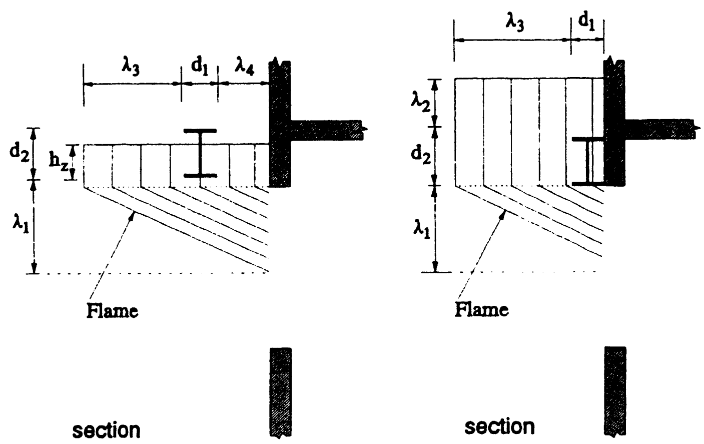
$T_{z,2}$ is the flame temperature [K] from annex C of ENV 1991-2-2, level with the top of the beam.

(3) In the case of a beam parallel to the external wall of the fire compartment C_4 may be taken as zero if the beam is immediately adjacent to the wall, see figure C.7.



1) Beam perpendicular to wall

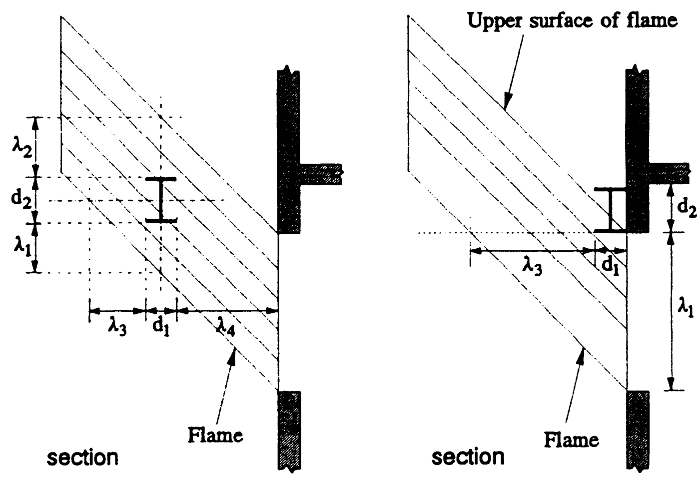
2) Beam parallel to wall



3) Top of flame below top of beam

4) Beam immediately adjacent to wall

a) 'No forced draught' condition



1) Beam not adjacent to wall

2) Beam immediately adjacent to wall

b) 'Forced draught' condition

Figure C.7: Beam engulfed in flame

(4) If the top of the flame is below the level of the top of the beam:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{C.23a}$$

$$I_{z,2} = 0 \tag{C.23b}$$

$$I_{z,3} = (h_z / d_2) C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_x^4)/2 \tag{C.23c}$$

$$I_{z,4} = (h_z / d_2) C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_x^4)/2 \tag{C.23d}$$

where:

T_x is the flame temperature at the flame tip [813 K];

h_z is the height of the top of the flame above the bottom of the beam.

C.5.1.3 'Forced draught' condition

(1) For the 'forced draught' condition, in the case of beams parallel to the external wall of the fire compartment a distinction should be made between those immediately adjacent to the wall and those not immediately adjacent to it, see figure C.7.

(2) For a beam parallel to the wall, but not immediately adjacent to it, or for a beam perpendicular to the wall:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{C.24a}$$

$$I_{z,2} = C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \tag{C.24b}$$

$$I_{z,3} = C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.24c}$$

$$I_{z,4} = C_4 \varepsilon_{z,4} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.24d}$$

(3) If the beam is parallel to the wall and immediately adjacent to it, only the bottom face should be taken as engulfed in flame but one side and the top should be taken as exposed to radiative heat transfer from the upper surface of the flame, see figure C.7(b)(2). Thus:

$$I_{z,1} = C_1 \varepsilon_{z,1} \sigma T_o^4 \tag{C.25a}$$

$$I_{z,2} = \phi_{z,2} C_2 \varepsilon_{z,2} \sigma T_{z,2}^4 \tag{C.25b}$$

$$I_{z,3} = \phi_{z,3} C_3 \varepsilon_{z,3} \sigma (T_{z,1}^4 + T_{z,2}^4)/2 \tag{C.25c}$$

$$I_{z,4} = 0 \tag{C.25d}$$

where $\phi_{z,i}$ is the configuration factor relative to the upper surface of the flame, for face i of the beam, from annex D.

C.5.2 Flame emissivity

(1) The emissivity of the flame ε_{zi} for each of the faces 1, 2, 3 and 4 of the beam should be determined from the expression for ε given in annex C of ENV 1991-2-2, using a flame thickness λ equal to the dimension λ_i indicated in figure C.7 corresponding to face i of the beam.

C.5.3 Flame absorptivity

(1) The absorptivity of the flame a_z should be determined from:

$$a_z = 1 - e^{-0,3h} \tag{C.26}$$

Annex D [informative]

Configuration factor

(1) The configuration factor ϕ is defined in 1.4.1. It measures the fraction of the total radiative heat leaving a given radiating surface that arrives at a given receiving surface. Its value depends on the size of the radiating surface, on the distance from the radiating surface to the receiving surface and on their relative orientation.

(2) In this annex all radiating surfaces are assumed to be rectangular in shape. They comprise the windows and other openings in fire compartment walls and the equivalent rectangular surfaces of flames, see C.1.4.

(3) In calculating the configuration factor for a given situation, a rectangular envelope should first be drawn around the cross-section of the member receiving the radiative heat transfer, as indicated in figure D.1. The value of ϕ should then be determined for the mid-point P of each face of this rectangle.

(4) The configuration factor for each receiving surface should be determined as the sum of the contributions from each of the zones on the radiating surface (normally four) that are visible from the point P on the receiving surface, as indicated in figures D.2 and D.3. These zones should be defined relative to the point X where a horizontal line perpendicular to the receiving surface meets the plane containing the radiating surface. No contribution should be taken from zones such as the shaded zones on figure D.3 that are not visible from the point P.

(5) If the point X lies outside the radiating surface, the effective configuration factor should be determined by adding the contributions of the two rectangles extending from X to the farther side of the radiating surface, then subtracting the contributions of the two rectangles extending from X to the nearer side of the radiating surface.

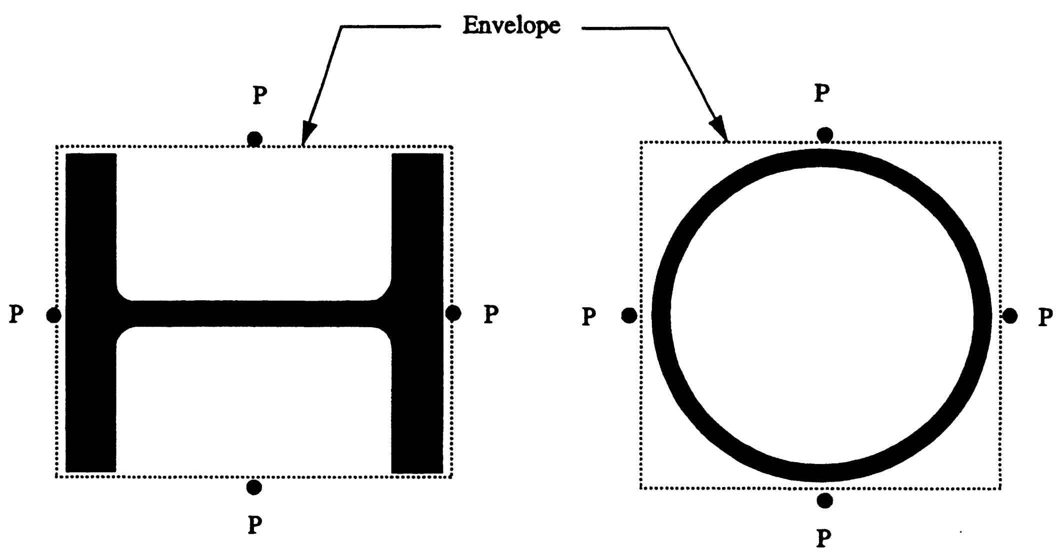


Figure D.1: Envelope of receiving surfaces

(6) The contribution of each zone should be determined as follows:

a) receiving surface parallel to radiating surface:

$$\phi = \frac{1}{2\pi}\left[\frac{a}{(1+a^2)^{0,5}}\tan^{-1}\left[\frac{b}{(1+a^2)^{0,5}}\right] + \frac{b}{(1+b^2)^{0,5}}\tan^{-1}\left[\frac{a}{(1+b^2)^{0,5}}\right]\right] \quad \text{(D.1)}$$

with:

$a = h/s$

$b = w/s$

where:

s is the distance from P to X;

h is the height of the zone on the radiating surface;

w is the width of that zone.

b) receiving surface perpendicular to radiating surface:

$$\phi = \frac{1}{2\pi}\left[\tan^{-1}(a) - \frac{1}{(1+b^2)^{0,5}}\tan^{-1}\left[\frac{a}{(1+b^2)^{0,5}}\right]\right] \quad \text{(D.2)}$$

c) receiving surface in a plane at angle θ to the radiating surface:

$$\phi = \frac{1}{2\pi}\left[\tan^{-1}(a) - \frac{(1-b\cos\theta)}{(1+b^2-2b\cos\theta)^{0,5}}\tan^{-1}\left[\frac{a}{(1+b^2-2b\cos\theta)^{0,5}}\right]\right.$$
$$\left. + \frac{a\cos\theta}{(a^2+\sin^2\theta)^{0,5}}\left[\tan^{-1}\left[\frac{(b-\cos\theta)}{(a^2+\sin^2\theta)^{0,5}}\right] + \tan^{-1}\left[\frac{\cos\theta}{(a^2+\sin^2\theta)^{0,5}}\right]\right]\right] \quad \text{(D.3)}$$

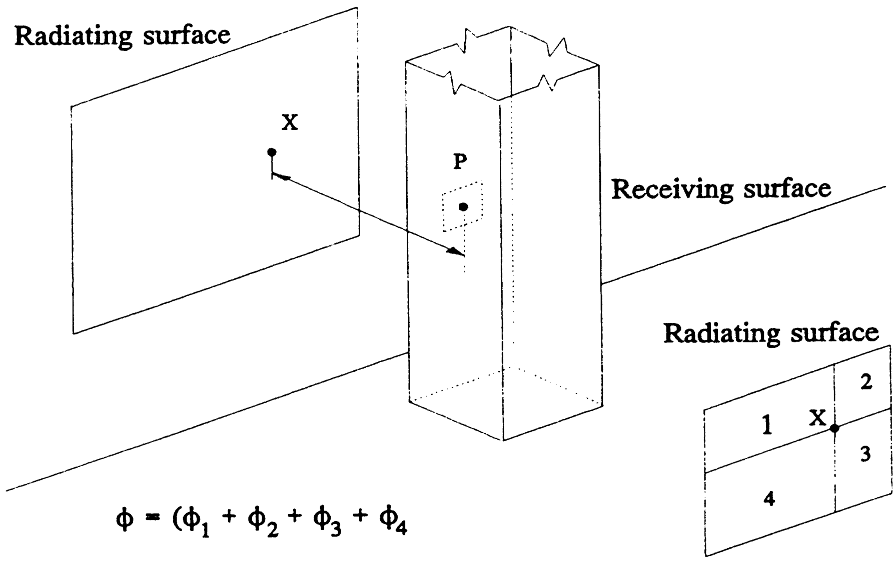


Figure D.2: Receiving surface in a plane parallel to that of the radiating surface

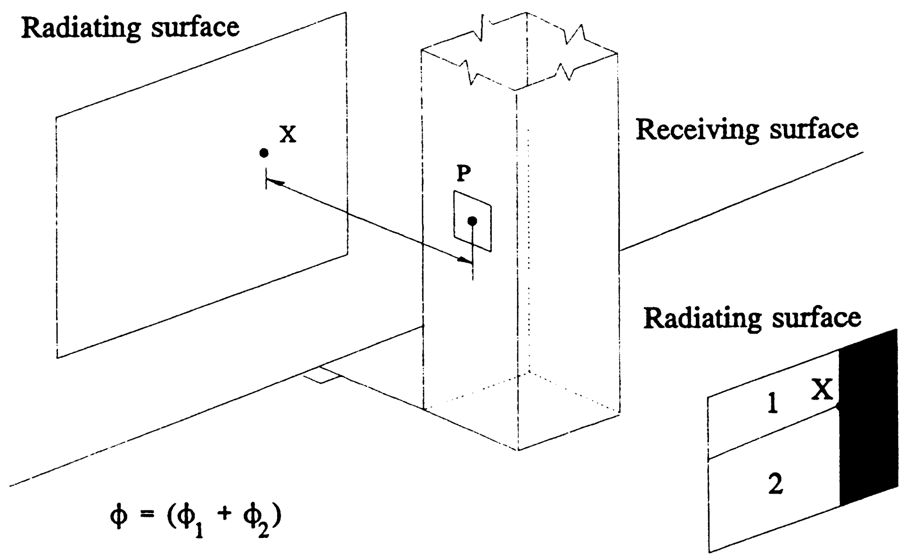


Figure D.3: Receiving surface perpendicular to the plane of the radiating surface

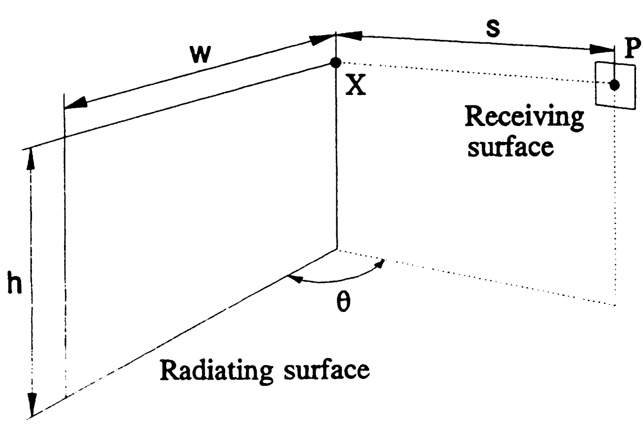


Figure D.4: Receiving surface in a plane at angle θ to that of the radiating surface

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