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EN 1994-1-2 (2005) (English): Eurocode 4: Design of composite steel and concrete structures – Part 1-2: General rules - Structural fire design [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

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English Version

Eurocode 4 - Design of composite steel and concrete structures - Part 1-2: General rules - Structural fire design

Eurocode 4 - Calcul des structures mixtes acier-béton - Partie 1-2: Règles générales - Calcul du comportement au feu

Eurocode 4 - Bemessung und Konstruktion von Verbundtragwerken aus Stahl und Beton - Teil 1-2: Allgemeine Regeln Tragwerksbemessung im Brandfall

This European Standard was approved by CEN on 4 November 2004.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

EUROPEAN COMMITTEE FOR STANDARDIZATION
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Ref. No. EN 1994-1-2:2005: E

Contents

Foreword

This European Standard EN 1994-1-2: 2005, Eurocode 4: Design of composite steel and concrete structures: Part 1-2 : General rules – Structural fire design, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI.

CEN/TC250 is responsible for all Structural Eurocodes.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by February 2006, and conflicting National Standards shall be withdrawn at latest by March 2010.

This Eurocode supersedes ENV 1994-1-2: 1994.

According to the CEN-CENELEC Internal Regulations, the National Standard Organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products – CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN1990, Eurocode : Basis of structural design

EN1991, Eurocode 1: Actions on structures

EN1992, Eurocode 2: Design of concrete structures

EN1993, Eurocode 3: Design of steel structures

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[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN1994, Eurocode 4: Design of composite steel and concrete structures

EN1995, Eurocode 5: Design of timber structures

EN1996, Eurocode 6: Design of masonry structures

EN1997, Eurocode 7: Geotechnical design

EN1998, Eurocode 8: Design of structures for earthquake resistance

EN1999, Eurocode 9: Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that EUROCODES serve as reference documents for the following purposes :

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services ;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs).

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

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[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for hENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

The National Annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, *i.e.* :

– values and/or classes where alternatives are given in the Eurocode;

– values to be used where a symbol only is given in the Eurocode;

– country specific data (geographical, climatic, etc), e.g. snow map;

– the procedure to be used where alternative procedures are given in the Eurocode;

it may also contain:

– decisions on the application of informative annexes, and

– references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products.

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific for EN 1994-1-2

EN 1994-1-2 describes the Principles, requirements and rules for the structural design of buildings exposed to fire, including the following aspects:

Safety requirements

EN 1994-1-2 is intended for clients (e.g. for the formulation of their specific requirements), designers, contractors and public authorities.

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

Construction Products Directive 89/106/EEC gives the following essential requirement for the limitation of fire risks:

"The construction works must be designed and built in such a way, that in the event of an outbreak of fire

- the load bearing resistance of the construction can be assumed for a specified period of time;

- the generation and spread of fire and smoke within the works are limited;

- the spread of fire to neighbouring construction works is limited;

- the occupants can leave the works or can be rescued by other means;

- the safety of rescue teams is taken into consideration".

[4] see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID N°1.
[5] see clauses 2.2, 3.2(4) and 4.2.3.3 of ID N°2

According to the Interpretative Document N°2 "Safety in Case of Fire[5]" the essential requirement may be observed by following various possibilities for fire safety strategies prevailing in the Member States like conventional fire scenarios (nominal fires) or “natural” (parametric) fire scenarios, including passive and/or active fire protection measures.

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

Required functions and levels of performance can be specified either in terms of nominal (standard) fire resistance rating, generally given in national regulations or, where allowed by national fire regulations, by referring to fire safety engineering for assessing passive and active measures.

Supplementary requirements concerning, for example

- the possible installation and maintenance of sprinkler systems;

- conditions on occupancy of 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 the competent authority.

Numerical values for partial factors and other reliability elements are given as recommended values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies.

Design procedures

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).

At the present time it is possible to undertake a procedure for determining adequate performance which 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 where the procedure is based on a nominal (standard) fire, the classification system, which calls for specific periods of fire resistance, takes into account (though not explicitly), the features and uncertainties described above.

Application of this Part 1-2 is illustrated below. The prescriptive approach and the performance-based approach are identified. The prescriptive approach uses nominal fires to generate thermal actions. The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters.

For design according to this part, EN 1991-1-2 is required for the determination of thermal and mechanical actions to the structure.

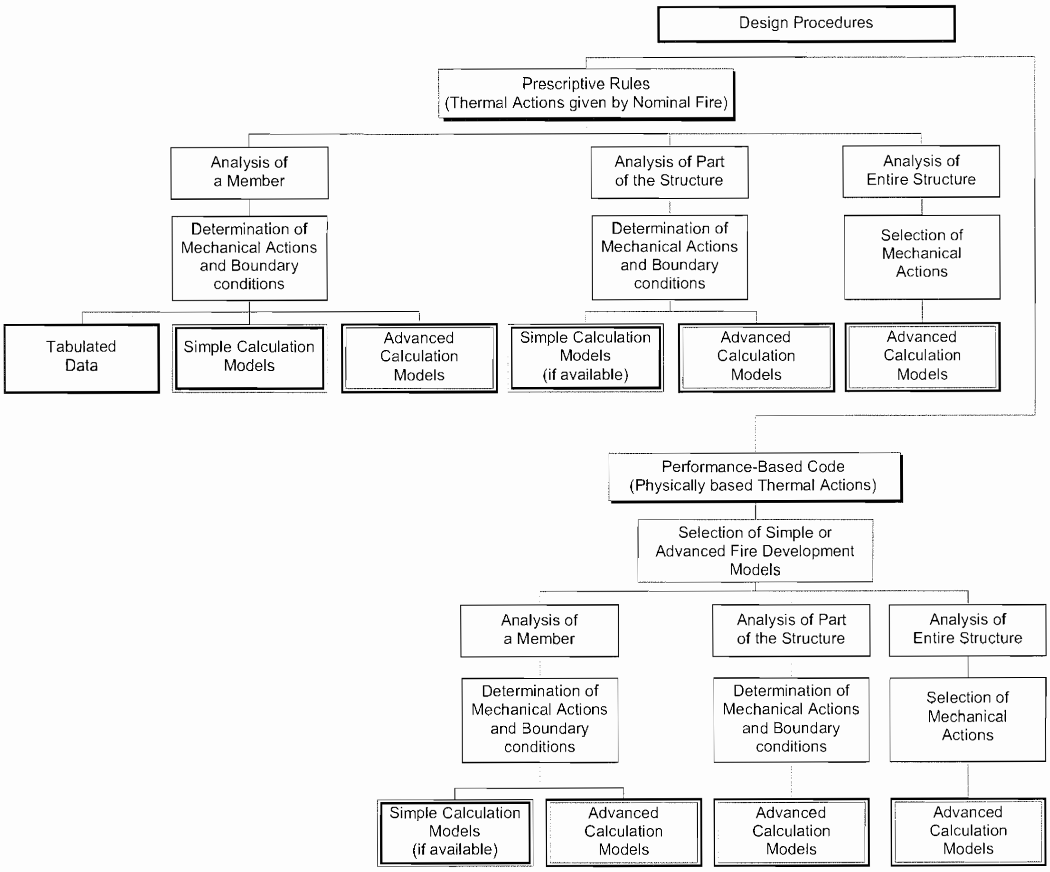


Figure 0.1: Alternative design procedures

Design aids

Apart from simple calculation models, EN 1994-1-2 gives design solutions in terms of tabulated data (based on tests or advanced calculation models) which may be used within the specified limits of validity.

It is expected, that design aids based on the calculation models given in EN 1994-1-2, will be prepared by interested external organizations.

The main text of EN 1994-1-2 together with informative Annexes A to I includes most of the principal concepts and rules necessary for structural fire design of composite steel and concrete structures.

National annex for EN 1994-1-2

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1994-1-2 should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings to be constructed in the relevant country.

National choice is allowed in EN 1994-1-2 through clauses:

- 1.1(16)
- 2.1.3(2)
- AC1 2.3 (1)P NOTE 1
- 2.3 (2)P NOTE 1
- 2.4.2 (3) NOTE 1
- 3.3.2 (9) NOTE 1
- 4.1(1)P
- 4.3.5.1 (10) NOTE 1 AC1

Section 1 General

1.1 Scope

(1) This Part 1-2 of EN 1994 deals with the design of composite steel and concrete structures for the accidental situation of fire exposure and is intended to be used in conjunction with EN 1994-1-1 and EN 1991-1-2. This Part 1-2 only identifies differences from, or supplements to, normal temperature design.

(2) This Part 1-2 of EN 1994 deals only with passive methods of fire protection. Active methods are not covered.

(3) This Part 1-2 of EN 1994 applies to composite steel and concrete structures that are required to fulfil certain functions when exposed to fire, in terms of:

- avoiding premature collapse of the structure (load bearing function);
- limiting fire spread (flame, hot gases, excessive heat) beyond designated areas (separating function).

(4) This Part 1-2 of EN 1994 gives principles and application rules (see EN 1991-1-2) for designing structures for specified requirements in respect of the aforementioned functions and the levels of performance.

(5) This Part 1-2 of EN 1994 applies to structures, or parts of structures, that are within the scope of EN 1994-1-1 and are designed accordingly. However, no rules are given for composite elements which include prestressed concrete parts.

(6) For all composite cross-sections longitudinal shear connection between steel and concrete should be in accordance with EN 1994-1-1 or be verified by tests (see also 4.3.4.1.5 and Annex I).

(7) Typical examples of concrete slabs with profiled steel sheets with or without reinforcing bars are given in Figure 1.1.

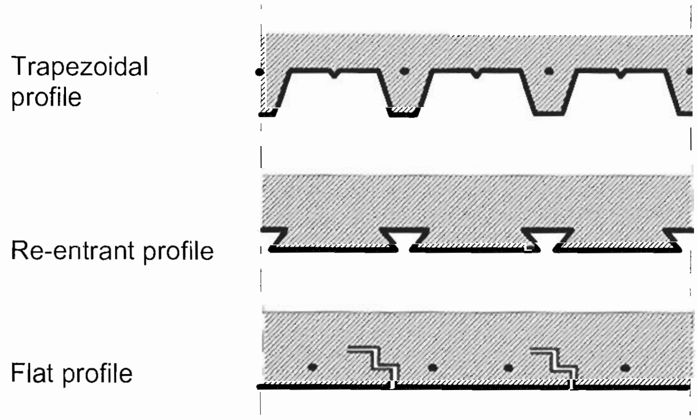
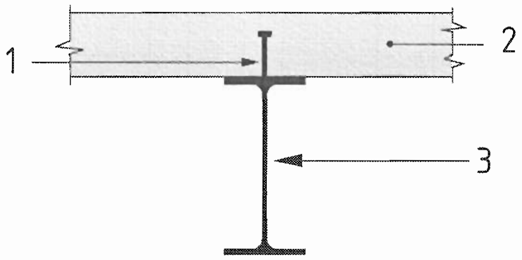


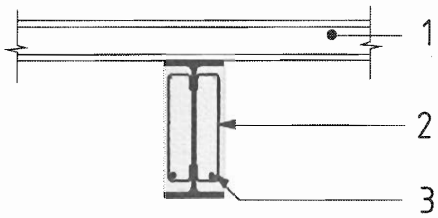
Figure 1.1 Typical examples of concrete slabs with profiled steel sheets with or without reinforcing bars

(8) Typical examples of composite beams are given in Figures 1.2 to 1.5. The corresponding constructional detailing is covered in section 5.



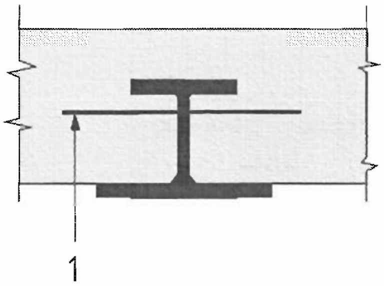
Key
1 – Shear connectors
2 – Flat concrete slab or composite slab with profiled steel sheeting
3 – Profiles with or without protection

Figure 1.2: Composite beam comprising steel beam with no concrete encasement



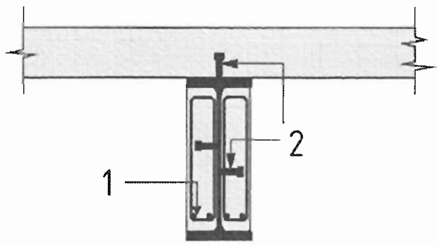
Key
1 – Optional
2 – Stirrups welded to web of profile
3 – Reinforcing bar

Figure 1.3: Steel beam with partial concrete encasement



Key
1 – Reinforcing bar

Figure 1.4: Steel beam partially encased in slab



Key
1 – Reinforcing bar
2 – Shear connectors

Figure 1.5: Composite beam comprising steel beam with partial concrete encasement

(9) Typical examples of composite columns are given in Figures 1.6 to 1.8. The corresponding constructional detailing is covered in section 5.

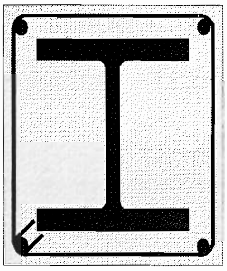
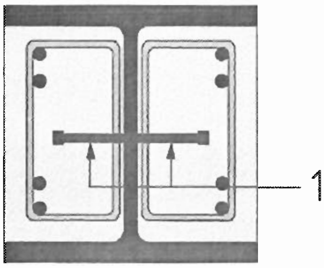


Figure 1.6:
Concrete encased profiles



Key
1 – Shear connectors welded to web of profile

Figure 1.7:
Partially encased profiles

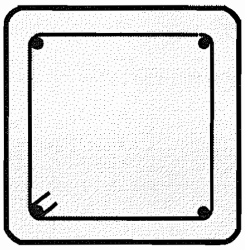


Figure 1.8:
Concrete filled profiles

(10) Different shapes, like circular or octagonal cross-sections may also be used for columns. Where appropriate, reinforcing bars may be replaced by steel sections.

(11) The fire resistance of these types of constructions may be increased by applying fire protection materials.

NOTE: The design principles and rules given in 4.2, 4.3 and 5 refer to steel surfaces directly exposed to the fire, which are free of any fire protection material, unless explicitly specified otherwise.

(12)P The methods given in this Part 1-2 of EN 1994 are applicable to structural steel grades S235, S275, S355, S420 and S460 of EN 10025, EN 10210-1 and EN 10219-1.

(13) For profiled steel sheeting, reference is made to section 3.5 of EN 1994-1-1.

(14) Reinforcing bars should be in accordance with EN 10080.

(15) Normal weight concrete, as defined in EN 1994-1-1, is applicable to the fire design of composite structures. The use of lightweight concrete is permitted for composite slabs.

(16) This part of EN 1994 does not cover the design of composite structures with concrete strength classes lower than C20/25 and LC20/22 and higher than C50/60 and LC50/55.

NOTE : Information on Concrete Strength Classes higher than C50/60 is given in section 6 of EN 1992-1-2. The use of these concrete strength classes may be specified in the National Annex.

(17) For materials not included herein, reference should be made to relevant CEN product standards or European Technical Approval (ETA).

1.2 Normative references

(1)P This European Standard 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 Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

EN 1365 -1	Fire resistance tests for loadbearing elements – Part 1: Walls
EN 1365 -2	Fire resistance tests for loadbearing elements – Part 2: Floors and roofs
EN 1365 -3	Fire resistance tests for loadbearing elements – Part 3: Beams

EN 1365 -4	Fire resistance tests for loadbearing elements – Part 4: Columns
EN 10025-1	Hot-rolled products of structural steels - Part 1: General technical delivery conditions
EN 10025-2	Hot-rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels
EN 10025-3	Hot-rolled products of structural steels - Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels
EN 10025-4	Hot-rolled products of structural steels - Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels
EN 10025-5	Hot-rolled products of structural steels - Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance
EN 10025-6	Hot-rolled products of structural steels - Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition
EN 10080	Steel for the reinforcement of concrete - Weldable reinforcing steel General
EN 10210-1	Hot finished structural hollow sections of non-alloy and fine grain structural steels – Part 1 : Technical delivery conditions
EN 10219-1	Cold formed welded structural hollow sections of non-alloy and fine grain structural steels – Part 1: Technical delivery conditions
ENV 13381-1	Test methods for determining the contribution to the fire resistance of structural members – Part 1: Horizontal protective membranes
ENV 13381-2	Test methods for determining the contribution to the fire resistance of structural members – Part 2: Vertical protective membranes
ENV 13381-3	Test methods for determining the contribution to the fire resistance of structural members – Part 3: Applied protection to concrete members
ENV 13381-4	Test methods for determining the contribution to the fire resistance of structural members – Part 4: Applied protection to steel members
ENV 13381-5	Test methods for determining the contribution to the fire resistance of structural members – Part 5: Applied protection to concrete/profiled sheet composite members
AC1 ENV 13381-6	Test methods for determining the contribution to the fire resistance of structural members – Part 6: Applied protection to concrete filled hollow steel columns AC1
EN 1990	Eurocode: Basis of structural design
EN 1991 -1-1	Eurocode 1 : Actions on Structures – Part 1.1: General Actions - Densities, self-weight and imposed loads
EN 1991 -1-2	Eurocode 1 : Actions on Structures – Part 1.2: General Actions - Actions on structures exposed to fire

EN 1991 -1-3	Eurocode 1 : Actions on Structures – Part 1.3: General Actions - Actions on structures - Snow loads
EN 1991 -1-4	Eurocode 1 : Actions on Structures – Part 1.4: General Actions - Actions on structures - Wind loads
EN 1992-1-1	Eurocode 2: Design of concrete structures - Part 1.1: General rules and rules for buildings
EN 1992-1-2	Eurocode 2: Design of concrete structures - Part 1.2: Structural fire design
EN 1993-1-1	Eurocode 3: Design of steel structures - Part 1.1: General rules and rules for buildings
EN 1993-1-2	Eurocode 3: Design of steel structures - Part 1.2: Structural fire design
EN 1993-1-5	Eurocode 3: Design of steel structures - Part 1.5: Plated structural elements
EN 1994-1-1	Eurocode 4: Design of composite steel and concrete structures - Part 1.1: General rules and rules for buildings"

1.3 Assumptions

(1)P Assumptions of EN 1990 and EN 1991-1-2 apply.

1.4 Distinction between Principles and Application Rules

(1) The rules given in EN 1990 clause 1.4 apply.

1.5 Definitions

(1)P The rules given in clauses 1.5 of EN 1990 and EN 1991-1-2 apply

(2)P The following terms are used in Part 1-2 of EN 1994 with the following meanings:

1.5.1 Special terms relating to design in general

1.5.1.1
axis distance
distance between the axis of the reinforcing bar and the nearest edge of concrete

1.5.1.2
part of structure
isolated part of an entire structure with appropriate support and boundary conditions

1.5.1.3
protected members
members for which measures are taken to reduce the temperature rise in the member due to fire

1.5.1.4
braced frame
a frame which has a sway resistance supplied by a bracing system which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system

1.5.2 Terms relating to material and products properties

1.5.2.1
failure time of protection
duration of protection against direct fire exposure; that is the time when the fire protective claddings or other protection fall off the composite member, or other elements aligned with that composite member fail due to collapse, or the alignment with other elements is terminated due to excessive deformation of the composite member

1.5.2.2
fire protection material
any material or combination of materials applied to a structural member for the purpose of increasing its fire resistance

1.5.3 Terms relating to heat transfer analysis

1.5.3.1
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

1.5.4 Terms relating to mechanical behaviour analysis

1.5.4.1
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

1.5.4.2
critical temperature of reinforcement
the temperature of the reinforcement at which failure in the element is expected to occur at a given load level

1.5.4.3
effective cross section
cross section of the member in structural fire design used in the effective cross section method. It is obtained by removing parts of the cross section with assumed zero strength and stiffness

1.5.4.4
maximum stress level
for a given temperature, the stress level at which the stress-strain relationship of steel is truncated to provide a yield plateau

1.6 Symbols

(1)P For the purpose of this Part 1-2 of EN 1994, the following symbols apply

Latin upper case letters

A	cross-sectional area or concrete volume of the member per metre of member length
$A_{a,\theta}$	cross-sectional area of the steel profile at the temperature θ
$A_{c,\theta}$	cross-sectional area of the concrete at the temperature θ
A_f	cross-sectional area of a steel flange

A_i, A_j	elemental area of the cross section with a temperature θ_i or θ_j or the exposed surface area of the part i of the steel cross-section per unit length
A/L_r	the rib geometry factor
A_i / V_i	section factor [m^{-1}] of the part i of the steel cross-section (non-protected member)
A_m	directly heated surface area of member per unit length
A_m/V	section factor of structural member
$A_{p,i}$	area of the inner surface of the fire protection material per unit length of the part i of the steel member
$A_{p,i}/ V_i$	section factor [m^{-1}] of the part i of the steel cross-section (with contour protection)
A_r	cross-sectional area of the stiffeners
A_r/V_r	section factor of stiffeners
$A_{s,\theta}$	cross-sectional area of the reinforcing bars at the temperature θ
E	integrity criterion
E 30	or E 60,...a member complying with the integrity criterion for 30, or 60... minutes in standard fire exposure
E_a	characteristic value for the modulus of elasticity of structural steel at 20°C
$E_{a,f}$	characteristic value for the modulus of elasticity of a profile steel flange
$E_{a,\theta}$	characteristic value for the slope of the linear elastic range of the stress-strain relationship of structural steel at elevated temperatures
$E_{a,\theta,\sigma}$	tangent modulus of the stress-strain relationship of the steel profile at elevated temperature θ and for stress $\sigma_{i,\theta}$
$E_{c,sec,\theta}$	characteristic value for the secant modulus of concrete in the fire situation, given by $f_{c,\theta}$ divided by $\varepsilon_{cu,\theta}$
$E_{c0,\theta}$	characteristic value for the tangent modulus at the origin of the stress-strain relationship for concrete at elevated temperatures and for short term loading
$E_{c,\theta,\sigma}$	tangent modulus of the stress-strain relationship of the concrete at elevated temperature θ and for stress $\sigma_{i,\theta}$
E_d	design effect of actions for normal temperature design
$E_{fi,d}$	design effect of actions in the fire situation, supposed to be time independent
$E_{fi,d,t}$	design effect of actions, including indirect fire actions and loads in the fire situation, at time t
$(EI)_{fi,c,z}$	flexural stiffness in the fire situation (related to the central axis Z of the composite cross-section)

$(EI)_{fi,eff}$	effective flexural stiffness in the fire situation
$(EI)_{fi,f,z}$	flexural stiffness of the two flanges of the steel profile in the fire situation (related to the central axis Z of the composite cross-section)
$(EI)_{fi,s,z}$	flexural stiffness of the reinforcing bars in the fire situation (related to the central axis Z of the composite cross-section)
$(EI)_{fi,eff,z}$	effective flexural stiffness (for bending around axis z) in the fire situation
$(EI)_{fi,w,z}$	flexural stiffness of the web of the steel profile in the fire situation (related to the central axis Z of the composite cross-section)
E_k	characteristic value of the modulus of elasticity
E_s	modulus of elasticity of the reinforcing bars
$E_{s,\theta}$	characteristic value for the slope of the linear elastic range of the stress-strain relationship of reinforcing steel at elevated temperatures
$E_{s,\theta,\sigma}$	tangent modulus of the stress-strain relationship of the reinforcing steel at elevated temperature θ and for stress $\sigma_{i,\theta}$
F_a	compressive force in the steel profile
F^+, F^-	total compressive force in the composite section in case of sagging or hogging bending moments
F_c	compression force in the slab
G_k	characteristic value of a permanent action
HC	hydrocarbon fire exposure curve
I	thermal insulation criterion
$I_{i,\theta}$	second moment of area, of the partially reduced part i of the cross-section for bending around the weak or strong axis in the fire situation
I 30	or I 60,... a member complying with the thermal insulation criterion for 30, or 60... minutes in standard fire exposure
L	system length of a column in the relevant storey
L_{ei}	buckling length of a column in an internal storey
L_{et}	buckling length of a column in the top storey
M	bending moment
M_{fi,Rd^+}; M_{fi,Rd^-}	design value of the sagging or hogging moment resistance in the fire situation
$M_{fi,t,Rd}$	design moment resistance in the fire situation at time t
N	number of shear connectors in one critical length,

	or axial load
N_{equ}	equivalent axial load
$N_{fi,cr}$	elastic critical load (≡ Euler buckling load) in the fire situation
$N_{fi,cr,z}$	elastic critical load (≡ Euler buckling load) around the axis Z in the fire situation
$N_{fi,pl,Rd}$	design value of the plastic resistance to axial compression of the total cross-section in the fire situation
$N_{fi,Rd}$	design value of the resistance of a member in axial compression (≡ design axial buckling load) and in the fire situation
$N_{fi,Rd,z}$	design value of the resistance of a member in axial compression, for bending around the axis Z, in the fire situation
$N_{fi,Sd}$	design value of the axial load in the fire situation
N_{Rd}	axial buckling load at normal temperature
N_s	normal force in the hogging reinforcement (A_s . f_{sy})
P_{Rd}	design shear resistance of a headed stud automatically welded
$P_{fi,Rd}$	design shear resistance in the fire situation of a shear connector
$Q_{k,1}$	characteristic value of the leading variable action 1
R	Load bearing criterion
R 30	or R 60, R90, R120, R180, R240... a member complying with the load bearing criterion for 30, 60, 90, 120, 180 or 240 minutes in standard fire exposure
R_d	design resistance for normal temperature design
$R_{fi,d,t}$	design resistance in the fire situation, at time t
$R_{fi,y,Rd}$	design crushing resistance in the fire situation
T	tensile force
V	volume of the member per unit length
$V_{fi,pl,Rd}$	design value of the shear plastic resistance in the fire situation
$V_{fi,Sd}$	design value of the shear force in the fire situation
V_i	volume of the part i of the steel cross section per unit length [m^3/m]
X	X (horizontal) axis
$X_{fi,d}$	design values of mechanical (strength and deformation) material properties in the fire situation

X_k	characteristic or nominal value of a strength or deformation property for normal temperature design
$X_{k,\theta}$	value of a material property in the fire situation, generally dependant on the material temperature
Y	Y (vertical) axis
Z	Z (column) central axis of the composite cross-section

Latin lower case letters

a_w	throat thickness of weld (connection between steel web and stirrups)
b	width of the steel section
b_1	width of the bottom flange of the steel section
b_2	width of the upper flange of the steel section
b_c	depth of the composite column made of a totally encased section, or width of concrete partially encased steel beams
$b_{c,fi}$	width reduction of the encased concrete between the flanges in the fire situation
$b_{c,fi,min}$	minimum value of the width reduction of the encased concrete between the flanges in the fire situation
b_{eff}	effective width of the concrete slab
b_{fi}	width reduction of upper flange in the fire situation
c	specific heat, or buckling curve, or concrete cover from edge of concrete to border of structural steel
c_a	specific heat of steel
c_c	specific heat of normal weight concrete
c_p	specific heat of the fire protection material
d	diameter of the composite column made of concrete filled hollow section, or diameter of the studs welded to the web of the steel profile
d_p	thickness of the fire protection material
e	thickness of profile or hollow section
e_1	thickness of the bottom flange of the steel profile
e_2	thickness of the upper flange of the steel profile
e_f	thickness of the flange of the steel profile
e_w	thickness of the web of the steel profile

ef	external fire exposure curve
$f_{ay,\theta}$	maximum stress level or effective yield strength of structural steel in the fire situation
$f_{ay,\theta cr}$	strength of steel at critical temperature θ_{cr}
$f_{ap,\theta}$; $f_{sp,\theta}$	proportional limit of structural or reinforcing steel in the fire situation
$f_{au,\theta}$	ultimate tensile strength of structural steel or steel for stud connectors in the fire situation, allowing for strain-hardening
f_{ay}	characteristic or nominal value for the yield strength of structural steel at 20°C
f_c	characteristic value of the compressive cylinder strength of concrete at 28 days and at 20°C.
$f_{c,j}$	characteristic strength of concrete part j at 20°C.
$f_{c,\theta}$	characteristic value for the compressive cylinder strength of concrete in the fire situation at temperature θ°C.
$f_{c,\theta n}$	residual compressive strength of concrete heated to a maximum temperature (with n layers)
$f_{c,\theta y}$	residual compressive strength of concrete heated to a maximum temperature
$f_{fi,d}$	design strength property in the fire situation
f_k	characteristic value of the material strength
f_{ry}, f_{sy}	characteristic or nominal value for the yield strength of a reinforcing bar at 20°C
$f_{sy,\theta}$	maximum stress level or effective yield strength of reinforcing steel in the fire situation
$f_{y,i}$	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
h	depth or height of the steel section
h_1	height of the concrete part of a composite slab above the decking
h_2	height of the concrete part of a composite slab within the decking
h_3	thickness of the screed situated on top of the concrete
h_c	depth of the composite column made of a totally encased section, or thickness of the concrete slab
h_{eff}	effective thickness of a composite slab
h_{fi}	height reduction of the encased concrete between the flanges in the fire situation
$\dot{h}_{net}$	design value of the net heat flux per unit area

$\dot{h}_{net,c}$ design value of the net heat flux per unit area by convection

$\dot{h}_{net,r}$ design value of the net heat flux per unit area by radiation

h_u thickness of the compressive zone

$h_{u,n}$ thickness of the compressive zone (with n layers)

h_v height of the stud welded on the web of the steel profile

h_w height of the web of the steel profile

$k_{c,\theta}$ reduction factor for the compressive strength of concrete giving the strength at elevated temperature $f_{c,\theta}$

$k_{E,\theta}$ reduction factor for the elastic modulus of structural steel giving the slope of the linear elastic range at elevated temperature $E_{a,\theta}$

$k_{y,\theta}$ reduction factor for the yield strength of structural steel giving the maximum stress level at elevated temperature $f_{ay,\theta}$

$k_{p,\theta}$ reduction factor for the yield strength of structural steel or reinforcing bars giving the proportional limit at elevated temperature $f_{ap,\theta}$ or $f_{sp,\theta}$

k_r , k_s reduction factor for the yield strength of a reinforcing bar

k_{shadow} correction factor for the shadow effect

$k_{u,\theta}$ reduction factor for the yield strength of structural steel giving the strain hardening stress level at elevated temperature $f_{au,\theta}$

k_θ reduction factor for a strength or deformation property dependent on the material temperature in the fire situation

ℓ length or buckling length

ℓ_1, ℓ_2, ℓ_3 specific dimensions of the re-entrant steel sheet profile or the trapezoidal steel profile

ℓ_w length (connection between steel profile and the encased concrete)

ℓ_θ buckling length of the column in the fire situation

s_s length of the rigid support (calculation of the crushing resistance of stiffeners)

t duration of fire exposure

$t_{fi,d}$ design value of standard fire resistance of a member in the fire situation

$t_{fi,requ}$ required standard fire resistance in the fire situation

t_i the fire resistance with respect to thermal insulation

u	geometrical average of the axis distances u_1 and u_2 (composite section with partially encased steel profile)
u_1 ; u_2	shortest distance from the centre of the reinforcement bar to the inner steel flange or to the nearest edge of concrete
z_i ; z_j	distance from the plastic neutral axis to the centroid of the elemental area A_i or A_j

Greek letters upper case letters

Δl	temperature induced elongation of a member
$\Delta l/l$	related thermal elongation
Δt	time interval
$\Delta\theta_{a,t}$	increase of temperature of a steel beam during the time interval Δt
$\Delta\theta_t$	increase in the gas temperature [°C] during the time interval Δt
Φ	configuration or view factor

Greek letters lower case letters

α	angle of the web
α_c	convective heat transfer coefficient
α_{slab}	coefficient taking into account the assumption of the rectangular stress block when designing slabs
γ_G	partial factor for permanent action G_k
$\gamma_{M,fi}$	partial factor for a material property in the fire situation
$\gamma_{M,fi,a}$	partial factor for the strength of structural steel in the fire situation
$\gamma_{M,fi,c}$	partial factor for the strength of concrete in the fire situation
$\gamma_{M,fi,s}$	partial factor for the strength of reinforcing bars in the fire situation
$\gamma_{M,fi,v}$	partial factor for the shear resistance of stud connectors in the fire situation
γ_Q	partial factor for variable action Q_k
γ_v	partial factor for the shear resistance of stud connectors at normal temperature
δ	eccentricity
ε	strain
ε_a	axial strain of the steel profile of the column

$\varepsilon_{a,\theta}$	strain in the fire situation
$\varepsilon_{ae,\theta}$	ultimate strain in the fire situation
$\varepsilon_{ay,\theta}$	yield strain in the fire situation
$\varepsilon_{ap,\theta}$	strain at the proportional limit in the fire situation
$\varepsilon_{au,\theta}$	limiting strain for yield strength in the fire situation
ε_{c}	axial strain of the concrete of the column
$\varepsilon_{c,\theta}$	concrete strain in the fire situation
$\varepsilon_{ce,\theta}$	maximum concrete strain in the fire situation
$\varepsilon_{ce,\theta max}$	maximum concrete strain in the fire situation at the maximum temperature
$\varepsilon_{cu,\theta}$	concrete strain corresponding to $f_{c,\theta}$
$\varepsilon_{cu,\theta max}$	concrete strain at the maximum concrete temperature
ε_{f}	emissivity coefficient of the fire
ε_{m}	emissivity coefficient related to the surface material of the member
ε_{s}	axial deformation of the reinforcing steel of the column
ϕ_b	diameter of a bar
ϕ_s	diameter of a stirrup
ϕ_r	diameter of a longitudinal reinforcement at the corner of the stirrups
η	load level according to EN 1994-1-1
η_{fi}	reduction factor applied to E_d in order to obtain $E_{fi,d}$
$\eta_{fi,t}$	load level for fire design
θ	temperature
θ_a	temperature of structural steel
$\theta_{a,t}$	steel temperature at time t assumed to be uniform in each part of the steel cross-section
θ_c	temperature of concrete
θ_{cr}	critical temperature of a structural member
θ_i	temperature in the elemental area A_i

θ_{lim}	limiting temperature
θ_{max}	maximum temperature
θ_r	the temperature of a stiffener
θ_R	the temperature of additional reinforcement in the rib
θ_s	temperature of reinforcing steel
θ_t	gas temperature at time t
θ_v	temperature of stud connectors
θ_w	temperature in the web
λ_a	thermal conductivity of steel
λ_c	thermal conductivity of concrete
λ_p	thermal conductivity of the fire protection material
$\overline{\lambda}$	relative slenderness
$\overline{\lambda}_{\theta}$	relative slenderness of stiffeners in the fire situation
ξ	reduction factor for unfavourable permanent action G_k
ρ_a	density of steel
ρ_c	density of concrete
$\rho_{c,NC}$	density of normal weight concrete
$\rho_{c,LC}$	density of lightweight concrete
ρ_p	density of the fire protection material
σ	stress
$\sigma_{a,\theta}$	stress of the steel profile in the fire situation
$\sigma_{c,\theta}$	stress of concrete under compression in the fire situation
$\sigma_{s,\theta}$	stress of reinforcing steel in the fire situation
$\varphi_{a,\theta}$	reduction coefficient for the steel profile depending on the effect of thermal stresses in the fire situation
$\varphi_{c,\theta}$	reduction coefficient for the concrete depending on the effect of thermal stresses in the fire situation
$\varphi_{s,\theta}$	reduction coefficient for reinforcing bars depending on the effect of thermal stresses in the fire situation

χ	reduction or correction coefficient and factor
χ_z	reduction or correction coefficient and factor (for bending around axis z)
$\psi_{0,1}$	combination factor for the characteristic or rare value of a variable action
$\psi_{1,1}$	combination factor for the frequent value of a variable action
$\psi_{2,1}$	combination factor for the quasi-permanent value of a variable action
ψ_{fi}	combination factor for a variable action in the fire situation, given either by $\psi_{1,1}$ or $\psi_{2,1}$

Section 2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

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

(2)P Where compartmentation is required, the elements forming the boundaries of the fire compartment, including joints, shall be designed and constructed in such a way that they maintain their separating function during the relevant fire exposure. This shall ensure, where relevant, that:

- integrity failure does not occur;
- insulation failure does not occur.

NOTE 1: See for definition EN1991-1-2, chapters 1.5.1.8 and 1.5.1.9

NOTE 2: In case of a composite slab, the thermal radiation criterion is not relevant.

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

(4) Consideration of the deformation of the load bearing structure is not necessary in the following cases, as relevant:

- the efficiency of the means of protection has been evaluated according to 3.3.4 and
- the separating elements have to fulfill requirements according to a nominal fire exposure.

2.1.2 Nominal fire exposure

(1)P For the standard fire exposure, members shall comply with criteria R, E and I as follows:

- separating only: integrity (criterion E) and, when requested, insulation (criterion I);
- load bearing only: mechanical resistance (criterion R);
- separating and load bearing: criteria R, E and, when requested, I.

(2) Criterion "R" is assumed to be satisfied where the load bearing function is maintained during the required time of fire exposure.

(3) Criterion "I" may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K.

(4) With the external fire exposure curve the same criteria should apply, however the reference to this specific curve should be identified by the letters "ef ".

NOTE : See EN1991-1-2, chapters 1.5.3.5 and 3.2.2

(5) With the hydrocarbon fire exposure curve the same criteria should apply, however the reference to this specific curve should be identified by the letters "HC".

NOTE : See EN1991-1-2, chapters 1.5.3.11 and 3.2.3

2.1.3 Parametric fire exposure

(1) The load-bearing function is ensured when collapse is prevented during the complete duration of the fire including the decay phase or during a required period of time.

(2) The separating function with respect to insulation is ensured when

- at the time of the maximum gas temperature, the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K,
- during the decay phase of the fire, the average temperature rise over the whole of the non-exposed surface should be limited to $\Delta\theta_1$, and the maximum temperature rise at any point of that surface should not exceed $\Delta\theta_2$.

NOTE : The values of $\Delta\theta_1$ and $\Delta\theta_2$ for use in a Country may be found in its National Annex. The recommended values are $\Delta\theta_1$ = 200 K and $\Delta\theta_2$ = 240 K.

2.2 Actions

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

(2) In addition to 3.1(6) of EN 1991-1-2, the emissivity coefficient for steel and concrete related to the surface of the member should be ε_m = 0,7.

2.3 Design values of material properties

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

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

where:

X_k is the characteristic or nominal value of a strength or deformation property *(generally f_k or E_k)* for normal temperature design according to EN 1994-1-1;

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;

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

NOTE 1: For mechanical properties of steel and concrete, the recommended values of the partial factor for the fire situation are $\gamma_{M,fi,a}$ = 1,0; $\gamma_{M,fi,s}$ = 1,0; $\gamma_{M,fi,c}$ = 1,0; $\gamma_{M,fi,v}$ = 1,0. Where modifications are required, these may be defined in the relevant National Annexes of EN 1992-1-2 and EN 1993-1-2.

NOTE 2: If the recommended values are modified, tabulated data may need to be adapted.

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

- if an increase of the property is favourable for safety;

$$X_{fi,d} = X_{k,\theta}/\gamma_{M,fi} \tag{2.2a}$$

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

$$X_{fi,d} = \gamma_{M,fi}\, X_{k,\theta} \tag{2.2b}$$

where:

$X_{k,\theta}$ is the value of a material property in the fire situation, generally dependent on the material temperature, see 3.3;

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

NOTE 1: For thermal properties of steel and concrete, the recommended value of the partial factor for the fire situation is $\gamma_{M,fi}$ = 1,0; where modifications are required, these may be defined in the relevant National Annexes of EN 1992-1-2 and EN 1993-1-2.

NOTE 2: If the recommended values are modified, tabulated data may need to be adapted.

(3) The design value of the compressive concrete strength should be taken as 1,0 f_c divided by $\gamma_{M,fi,c}$, before applying the required strength reduction due to temperature and given in 3.2.2.

2.4 Verification methods

2.4.1 General

(1)P The model of the structural system adopted for design to this Part 1-2 of EN 1994 shall reflect the expected performance of the structure in fire.

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

$$E_{fi,d,t} \leq R_{fi,d,t} \tag{2.3}$$

where:

$E_{fi,d,t}$ is the design effect of actions for the fire situation, determined in accordance with EN 1991-1-2, including the effects of thermal expansions and deformations;

$R_{fi,d,t}$ is the corresponding design resistance in the fire situation.

(3) The structural analysis for the fire situation should be carried out according to 5.1.4(2) of EN 1990.

NOTE: For verifying standard fire resistance requirement, a member analysis is sufficient.

(4) Where application rules given in this Part 1-2 are valid only for the standard temperature-time curve, this is identified in the relevant clauses.

(5) Tabulated data given in 4.2 are based on the standard temperature-time curve.

(6)P As an alternative to design by calculation, fire design may be based on the results of fire tests, or on fire tests in combination with calculations, see EN 1990 clause 5.2.

2.4.2 Member analysis

(1) The effect of actions should be determined for time $t = 0$ using combination factors $\psi_{1,1}$ or $\psi_{2,1}$ according to 4.3.1(2) of EN 1991-1-2.

(2) As a simplification to (1), the effect of actions $E_{fi,d,t}$ may be obtained from a structural analysis for normal temperature design as:

$$E_{fi,d,t} = E_{fi,d} = \eta_{fi}\, E_d \tag{2.4}$$

where:

E_d is the design value of the corresponding force or moment for normal temperature design, for a fundamental combination of actions (see EN 1990)

η_{fi} is the reduction factor of E_d

(3) The reduction factor η_{fi} for load combination (6.10) in EN 1990 should be taken as:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{2.5}$$

or for load combinations (6.10a) and (6.10b) in EN 1990 as the smaller value given by the two following expressions:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} \psi_{0,1} Q_{k,1}} \tag{2.5a}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{2.5b}$$

where:

$Q_{k,1}$ is the characteristic value of the leading variable action 1

G_k is the characteristic value of a permanent action

γ_G is the partial factor for permanent actions

$\gamma_{Q,1}$ is the partial factor for variable action 1

ξ is a reduction factor for unfavourable permanent action G_k

$\psi_{0,1}$ combination factor for the characteristic value of a variable action

ψ_{fi} is the combination factor for fire situation, given either by $\psi_{1,1}$ (frequent value) or $\psi_{2,1}$ (quasi-permanent value) according to 4.3.1(2) of EN 1991-1-2

NOTE 1: An example of the variation of the reduction factor η_{fi} versus the load ratio $Q_{k,1}/G_k$ for different values of the combination factor $\psi_{fi} = \psi_{1,1}$ according to expression (2.5), is shown in Figure 2.1 with the following assumptions: γ_G = 1,35 and γ_Q = 1,5. Partial factors are specified in the relevant National Annexes of EN 1990. Equations (2.5a) and (2.5b) give slightly higher values.

NOTE 2: As a simplification the recommended value of η_{fi} = 0,65 may be used, except for imposed loads according to load category E as given in EN 1991-1-1 (areas susceptible to accumulation of goods, including access areas), where the recommended value is 0,7.

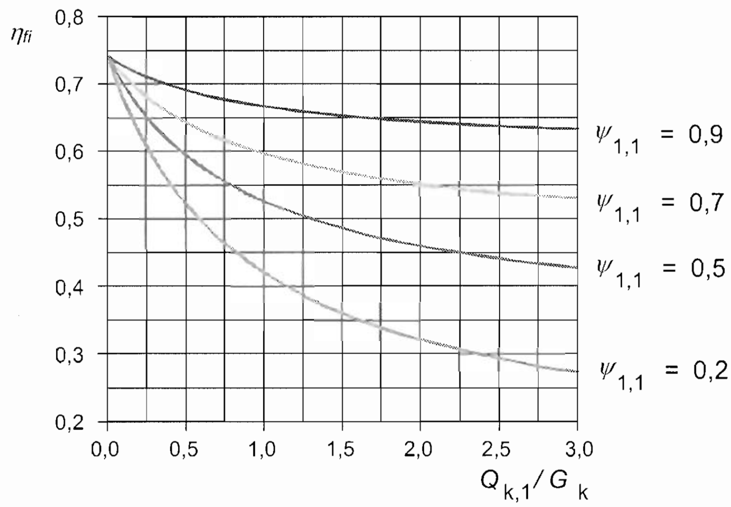


Figure 2.1: Variation of the reduction factor η_{fi} with the load ratio $Q_{k,1}/G_k$

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

(5) The boundary conditions at supports and ends of member may be assumed to remain unchanged throughout the fire exposure.

(6) Tabulated data, simplified or advanced calculation models given in 4.2, 4.3 and 4.4 respectively are suitable for verifying members under fire conditions.

2.4.3 Analysis of part of the structure

(1) The effect of actions should be determined for time *t* = *0* using combination factors $\psi_{1,1}$ or $\psi_{2,1}$ according to 4.3.1(2) of EN 1991-1-2.

(2) As an alternative to carrying out a structural analysis for the fire situation at time t=0, the reactions at supports and internal forces and moments at boundaries of part of the structure may be obtained from a structural analysis for normal temperature as given in 2.4.2.

(3) The part of the structure to be analysed should be specified on the basis of the potential thermal expansions and deformations such, that their interaction with other parts of the structure can be approximated by time-independent support and boundary conditions during fire exposure.

(4)P Within the part of the structure to be analysed, the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffness, effects of thermal expansions and deformations (indirect fire actions) shall be taken into account.

(5) The boundary conditions at supports and forces and moments at boundaries of part of the structure, may be assumed to remain unchanged throughout the fire exposure.

2.4.4 Global structural analysis

(1)P When a global structural analysis for the fire situation is carried out, the relevant failure mode in fire exposure, the temperature-dependent material properties and member stiffness as well as the effects of thermal expansions and deformations (indirect fire actions) shall be taken into account.

Section 3 Material properties

3.1 General

(1)P In fire conditions the temperature dependent properties shall be taken into account.

(2) The thermal and mechanical properties of steel and concrete should be determined from the following clauses.

(3)P The values of material properties given in 3.2 shall be treated as characteristic values, see 2.3(1)P.

(4) The mechanical properties of concrete, reinforcing and prestressing steel at normal temperature (20°C) should be taken as those given in EN 1992-1-1 for normal temperature design.

(5) The mechanical properties of steel at 20 °C should be taken as those given in EN 1993-1-1 for normal temperature design.

3.2 Mechanical properties

3.2.1 Strength and deformation properties of structural steel

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

NOTE: For the rules of this standard, it is assumed that the heating rates fall within the specified limits.

(2) The stress-strain relationships given in Figure 3.1 and Table 3.1 are defined by three parameters:

- the slope of the linear elastic range $E_{a,\theta}$;
- the proportional limit $f_{ap,\theta}$;
- the maximum stress level or effective yield strength $f_{ay,\theta}$.

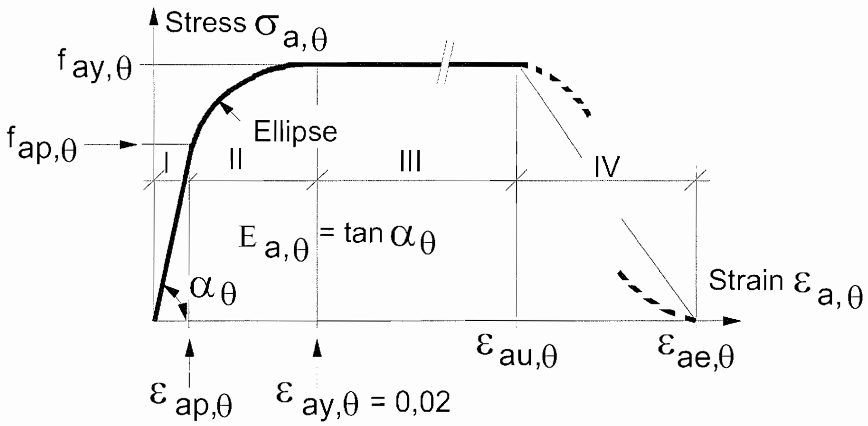


Figure 3.1: Mathematical model for stress-strain relationships of structural steel at elevated temperatures

Table 3.1: Relation between the various parameters of the mathematical model of Figure 3.1.

Strain Range	Stress σ	Tangent modulus
I / elastic $\varepsilon \le \varepsilon_{ap,\theta}$	$E_{a,\theta}\ \varepsilon_{a,\theta}$	$E_{a,\theta}$
II / transit elliptical $\varepsilon_{ap,\theta} \le \varepsilon$ $\varepsilon \le \varepsilon_{ay,\theta}$	$(f_{ap,\theta} - c) + \frac{b}{a}\sqrt{a^2 - (\varepsilon_{ay,\theta} - \varepsilon_{a,\theta})^2}$ with $a^2 = (\varepsilon_{ay,\theta} - \varepsilon_{ap,\theta})(\varepsilon_{ay,\theta} - \varepsilon_{ap,\theta} + c / E_{a,\theta})$ $b^2 = E_{a,\theta}\ (\varepsilon_{ay,\theta} - \varepsilon_{ap,\theta})c + c^2$ $c = \frac{(f_{ay,\theta} - f_{ap,\theta})^2}{E_{a,\theta}(\varepsilon_{ay,\theta} - \varepsilon_{ap,\theta}) - 2(f_{ay,\theta} - f_{ap,\theta})}$	$\frac{b(\varepsilon_{ay,\theta} - \varepsilon_{a,\theta})}{a\sqrt{a^2 - (\varepsilon_{ay,\theta} - \varepsilon_{a,\theta})^2}}$
III / plastic $\varepsilon_{ay,\theta} \le \varepsilon$ $\varepsilon \le \varepsilon_{au,\theta}$	$f_{ay,\theta}$	0

(3) Table 3.2 gives for elevated steel temperatures θ_a, the reduction factors k_θ to be applied to the appropriate value E_a or f_{ay} in order to determine the parameters in (2). For intermediate values of the temperature, linear interpolation may be used.

(4) Alternatively for temperatures below 400°C, the stress-strain relationships specified in (2) are extended by the strain hardening option given in Table 3.2, provided local instability is prevented and the ratio $f_{au,\theta}/f_{ay}$ is limited to 1,25.

NOTE: The strain-hardening option is detailed in informative Annex A.

(5) The effect of strain hardening should only be accounted for if the analysis is based on advanced calculation models according to 4.4. This is only allowed if it is proven that local failures (i.e. local buckling, shear failure, concrete spalling, etc) do not occur because of increased strains.

NOTE: Values for $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ defining the range of the maximum stress branches and decreasing branches according to Figure 3.1, may be taken from informative Annex A.

(6) The formulation of stress-strain relationships has been derived from tensile tests. These relationships may also be applied for steel in compression.

(7) In case of thermal actions according to 3.3 of EN 1991-1-2 (natural fire models), particularly when considering the decreasing temperature branch, the values specified in Table 3.2 for the stress-strain relationships of structural steel may be used as a sufficiently precise approximation.

Table 3.2: Reduction factors k_θ for stress-strain relationships of structural steel at elevated temperatures.

Steel Temperature θ_a[°C]	$k_{E,\theta} = \frac{E_{a,\theta}}{E_a}$	$k_{p,\theta} = \frac{f_{ap,\theta}}{f_{ay}}$	$k_{y,\theta} = \frac{f_{ay,\theta}}{f_{ay}}$	$k_{u,\theta} = \frac{f_{au,\theta}}{f_{ay}}$
20	1,00	1,00	1,00	1,25
100	1,00	1,00	1,00	1,25
200	0,90	0,807	1,00	1,25
300	0,80	0,613	1,00	1,25
400	0,70	0,420	1,00	
500	0,60	0,360	0,78	
600	0,31	0,180	0,47	
700	0,13	0,075	0,23	
800	0,09	0,050	0,11	
900	0,0675	0,0375	0,06	
1000	0,0450	0,0250	0,04	
1100	0,0225	0,0125	0,02	
1200	0	0	0	

3.2.2 Strength and deformation properties of concrete

(1) For heating rates between 2 and 50 K/min, the strength and deformation properties of concrete at elevated temperatures should be obtained from the stress-strain relationship given in Figure 3.2.

NOTE: For the rules of this standard, it is assumed that the heating rates fall within the specified limits.

(2)P The strength and deformation properties of uniaxially stressed concrete at elevated temperatures shall be obtained from the stress-strain relationships in EN 1992-1-2 and as presented in Figure 3.2.

(3) The stress-strain relationships given in Figure 3.2 are defined by two parameters:

- the compressive strength $f_{c,\theta}$;
- the strain $\varepsilon_{cu,\theta}$ corresponding to $f_{c,\theta}$.

(4) Table 3.3 gives for elevated concrete temperatures θ_c, the reduction factor $k_{c,\theta}$ to be applied to f_c in order to determine $f_{c,\theta}$ and the strain $\varepsilon_{cu,\theta}$. For intermediate values of the temperature, linear interpolation may be used.

NOTE: Due to various ways of testing specimens, $\varepsilon_{cu,\theta}$ shows considerable scatter, which is represented in Table B.1 of informative Annex B. Recommended values for $\varepsilon_{ce,\theta}$ defining the range of the descending branch may be taken from Annex B.

(5) For lightweight concrete (LC) the values of $\varepsilon_{cu,\theta}$, if needed, should be obtained from tests.

(6)The parameters specified in Table 3.3 hold for all qualities of concrete with siliceous aggregates. For calcareous concrete qualities the same parameters may be used. This is normally conservative. If more precise information is needed, reference should be made to Table 3.1 of EN 1992-1-2.

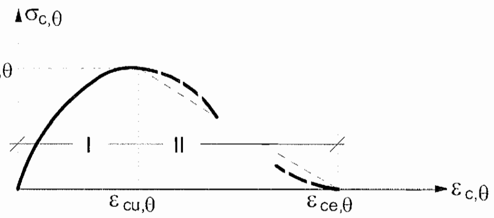
(7) In case of thermal actions according to 3.3 of EN 1991-1-2 (natural fire models), particularly when considering the decreasing temperature branch, the mathematical model for stress-strain relationships of concrete specified in Figure 3.2 should be modified.

NOTE: As concrete, which has cooled down after having been heated, does not recover its initial compressive strength, the proposal of informative Annex C may be used in an advanced calculation model according to 4.4.

(8) Conservatively the tensile strength of concrete may be assumed to be zero.

(9) If tensile strength is taken into account in verifications carried out with an advanced calculation model, it should not exceed the values based on 3.2.2.2 of EN1992-1-2.

(10) In case of tension in concrete, models with a descending stress-strain branch should be considered as presented in Figure 3.2.



RANGE I:

$$\sigma_{c,\theta} = f_{c,\theta}\left[3\left(\frac{\varepsilon_{c,\theta}}{\varepsilon_{cu,\theta}}\right)\Big/\left\{2+\left(\frac{\varepsilon_{c,\theta}}{\varepsilon_{cu,\theta}}\right)^{3}\right\}\right]$$

$$\left.\begin{array}{l} k_{c,\theta} = \dfrac{f_{c,\theta}}{f_c} \\ \text{and } \varepsilon_{cu,\theta} \end{array}\right\}$$ to be chosen according to the values of Table 3.3

RANGE II:

For numerical purposes a descending branch should be adopted

Figure 3.2: Mathematical model for stress-strain relationships of concrete under compression at elevated temperatures.

Table 3.3: Values for the two main parameters of the stress-strain relationships of normal weight concrete (NC) and lightweight concrete (LC) at elevated temperatures.

Concrete Temperature	$k_{c,\theta} = f_{c,\theta}/f_c$		$\varepsilon_{cu,\theta} \cdot 10^3$
θ_c [°C]	NC	LC	NC
20	1	1	2,5

100	1	1	4,0
200	0,95	1	5,5
300	0,85	1	7,0
400	0,75	0,88	10,0
500	0,60	0,76	15,0
600	0,45	0,64	25,0
700	0,30	0,52	25,0
800	0,15	0,40	25,0
900	0,08	0,28	25,0
1000	0,04	0,16	25,0
1100	0,01	0,04	25,0
1200	0	0	-

3.2.3 Reinforcing steels

(1) The strength and deformation properties of reinforcing steels at elevated temperatures may be obtained by the same mathematical model as that presented in 3.2.1 for structural steel.

(2) For hot rolled reinforcing steel the three main parameters given in Table 3.2 may be used, except that the value of $k_{u,\theta}$ should not be greater than 1,1.

(3) The three main parameters for cold worked reinforcing steel are given in Table 3.4 (see also Table 3.2a of EN 1992-1-2).

NOTE: Prestressing steels will normally not be used in composite structures.

(4) In case of thermal actions according to 3.3 of EN 1991-1-2 (natural fire models), particularly when considering the decreasing temperature branch, the values specified in Table 3.2 for the stress-strain relationships of structural steel, may be used as a sufficiently precise approximation for hot rolled reinforcing steel.

Table 3.4: Reduction factors k_θ for stress-strain relationships of cold worked reinforcing steel

Steel Temperature θ_s [°C]	$k_{E,\theta} = \frac{E_{s\theta}}{E_s}$	$k_{p,\theta} = \frac{f_{sp,\theta}}{f_{sy}}$	$k_{y,\theta} = \frac{f_{sy,\theta}}{f_{sy}}$
20	1,00	1,00	1,00
100	1,00	0,96	1,00
200	0,87	0,92	1,00
300	0,72	0,81	1,00
400	0,56	0,63	0,94
500	0,40	0,44	0,67
600	0,24	0,26	0,40
700	0,08	0,08	0,12
800	0,06	0,06	0,11
900	0,05	0,05	0,08
1000	0,03	0,03	0,05
1100	0,02	0,02	0,03
1200	0	0	0

3.3 Thermal properties

3.3.1 Structural and reinforcing steels

(1) The thermal elongation of steel $\Delta l / l$ valid for all structural and reinforcing steel qualities, may be determined from the following:

$$\Delta l / l = -2{,}416 \cdot 10^{-4} + 1{,}2 \cdot 10^{-5}\ \theta_a + 0{,}4 \cdot 10^{-8}\, \theta_a^2 \qquad \text{for } 20°\text{C} < \theta_a \leq 750°\text{C} \qquad (3.1a)$$

$$\Delta l / l = 11 \cdot 10^{-3} \qquad \text{for } 750°\text{C} < \theta_a \leq 860°\text{C} \qquad (3.1b)$$

$$\Delta l / l = -6{,}2 \cdot 10^{-3} + 2 \cdot 10^{-5}\ \theta_a \qquad \text{for } 860°\text{C} < \theta_a \leq 1200°\text{C} \qquad (3.1c)$$

where:

l is the length at 20°C of the steel member

Δl is the temperature induced elongation of the steel member

θ_a is the steel temperature.

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

(3) In simple calculation models (see 4.3) the relationship between thermal elongation and steel temperature may be considered to be linear. In this case the elongation of steel should be determined from:

$$\Delta l / l = 14 \cdot 10^{-6}\ (\theta_a - 20) \qquad (3.1d)$$

(4) The specific heat of steel c_a valid for all structural and reinforcing steel qualities may be determined from the following:

$$c_a = 425 + 7{,}73 \cdot 10^{-1}\ \theta_a - 1{,}69 \cdot 10^{-3}\ \theta_a^2 + 2{,}22 \cdot 10^{-6}\ \theta_a^3 \quad [\text{J/kgK}] \qquad \text{for } 20 \leq \theta_a \leq 600°\text{C} \qquad (3.2a)$$

$$c_a = 666 - \left(\frac{13002}{\theta_a - 738} \right) \quad [\text{J/kgK}] \qquad \text{for } 600 < \theta_a \leq 735°\text{C} \qquad (3.2b)$$

$$c_a = 545 + \left(\frac{17820}{\theta_a - 731} \right) \quad [\text{J/kgK}] \qquad \text{for } 735 < \theta_a \leq 900°\text{C} \qquad (3.2c)$$

$$c_a = 650 \quad [\text{J/kgK}] \qquad \text{for } 900 < \theta_a \leq 1200°\text{C} \qquad (3.2d)$$

where:

θ_a is the steel temperature

(5) The variation of the specific heat with temperature is illustrated in Figure 3.4.

(6) In simple calculation models (see 4.3) the specific heat may be considered to be independent of the steel temperature. In this case the following average value should be taken:

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

(7) The thermal conductivity of steel λ_a valid for all structural and reinforcing steel qualities may be determined from the following:

$$\lambda_a = 54 - 3{,}33 \,.\, 10^{-2}\, \theta_a \quad \text{[W/mK]} \quad \text{for } 20°\text{C} \leq \theta_a \leq 800°\text{C} \qquad (3.3a)$$

$$\lambda_a = 27{,}3 \quad \text{[W/mK]} \quad \text{for } 800°\text{C} < \theta_a \leq 1200°\text{C} \qquad (3.3b)$$

where θ_a is the steel temperature.

(8) The variation of the thermal conductivity with temperature is illustrated in Figure 3.5.

(9) In simple calculation models (see 4.3) the thermal conductivity may be considered to be independent of the steel temperature. In this case the following average value should be taken:

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

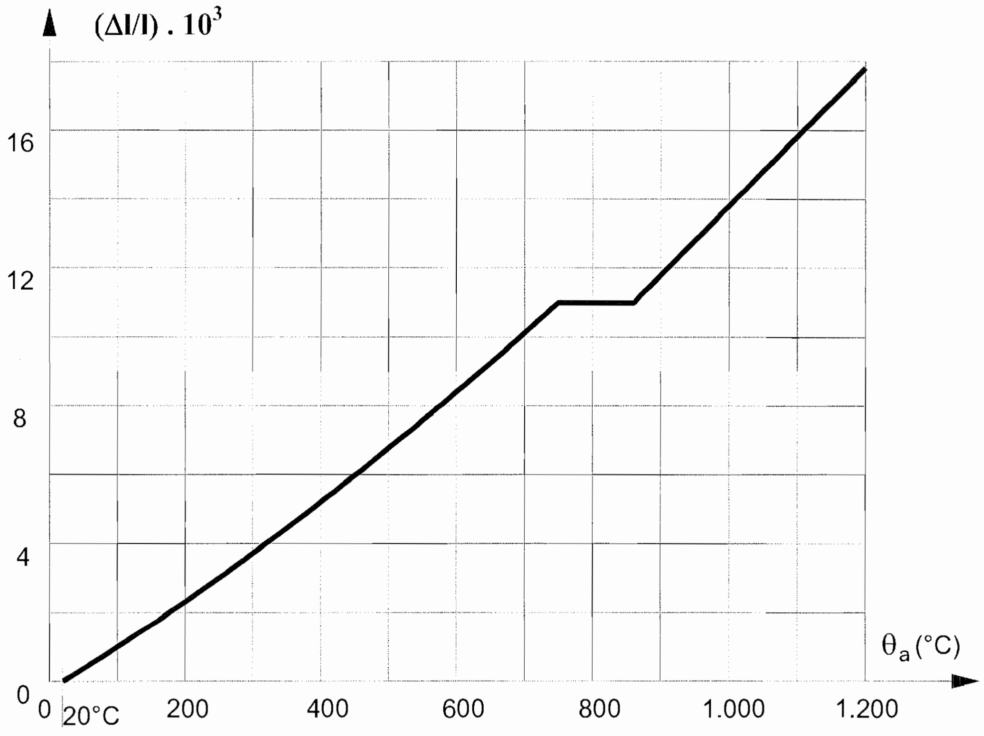


Figure 3.3: Thermal elongation of steel as a function of the temperature

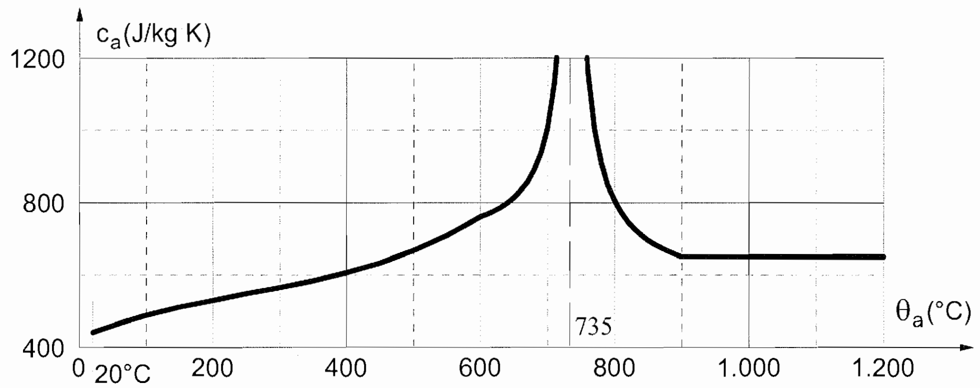


Figure 3.4: Specific heat of steel as a function of the temperature

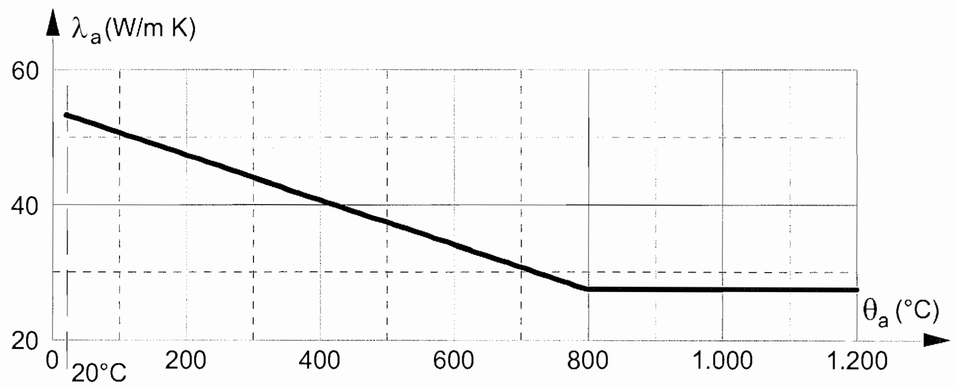


Figure 3.5: Thermal conductivity of steel as a function of the temperature

3.3.2 Normal weight concrete

(1) The thermal elongation $\Delta l / l$ of normal weight concrete and concrete with siliceous aggregates, may be determined from the following:

$$\Delta l / l = -1{,}8 \,.\, 10^{-4} + 9 \,.\, 10^{-6}\, \theta_c + 2{,}3 \,.\, 10^{-11}\, \theta_c^3 \qquad \text{for } 20°C \leq \theta_c \leq 700°C \qquad (3.4a)$$

$$\Delta l / l = 14 \,.\, 10^{-3} \qquad \text{for } 700°C < \theta_c \leq 1200°C \qquad (3.4b)$$

where:

l is the length at 20°C of the concrete member

Δl is the temperature induced elongation of the concrete member

θ_c is the concrete temperature

NOTE: For calcareous concrete, reference is made to 3.3.1(1) of EN1992-1-2.

(2) The variation of the thermal elongation with temperature is illustrated in Figure 3.6.

(3) In simple calculation models (see 4.3) the relationship between thermal elongation and concrete temperature may be considered to be linear. In this case the elongation of concrete should be determined from:

$$\Delta l / l = 18 \,.\, 10^{-6} \left(\theta_c - 20\right) \qquad (3.4c)$$

(4) The specific heat c_c of normal weight dry, siliceous or calcareous concrete may be determined from:

$$c_c = 900 \qquad \text{[J/kg K]} \qquad \text{for } 20°C \leq \theta_c \leq 100°C \qquad (3.5a)$$

$$c_c = 900 + \left(\theta_c - 100\right) \qquad \text{[J/kg K]} \qquad \text{for } 100°C < \theta_c \leq 200°C \qquad (3.5b)$$

$$c_c = 1000 + \left(\theta_c - 200\right)/2 \qquad \text{[J/kg K]} \qquad \text{for } 200°C < \theta_c \leq 400°C \qquad (3.5c)$$

$$c_c = 1100 \qquad \text{[J/kg K]} \qquad \text{for } 400°C < \theta_c \leq 1200°C \qquad (3.5d)$$

where θ_c is the concrete temperature [°C].

NOTE: The variation of c_c as a function of the temperature may be approximated by:

$$c_{c,\theta} = 890 + 56{,}2\left(\theta_c / 100\right) - 3{,}4\left(\theta_c / 100\right)^2 \qquad (3.5e)$$

(5) The variation of the specific heat with temperature according to (3.5e) is illustrated in Figure 3.7.

(6) In simple calculation models (see 4.3) the specific heat may be considered to be independent of the concrete temperature. In this case the following value should be taken:

$$c_c = 1000 \qquad \text{[J/kg K]} \qquad (3.5f)$$

(7) The moisture content of concrete should be taken equal to the equilibrium moisture content. If these data are not available, moisture content should not exceed 4 % of the concrete weight.

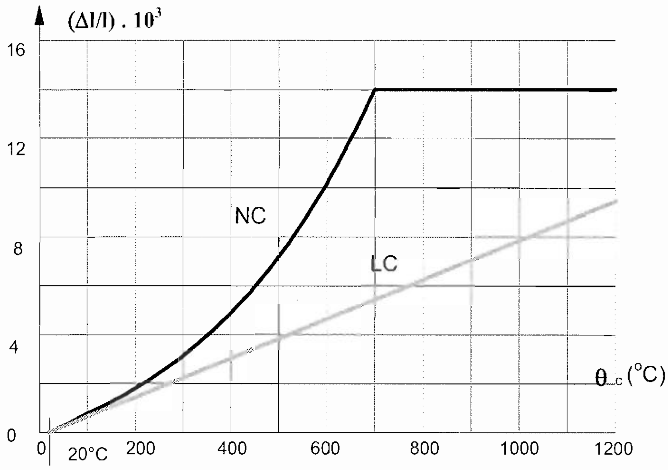


Figure 3.6: Thermal elongation of normal weight concrete (NC) and lightweight concrete (LC) as a function of the temperature

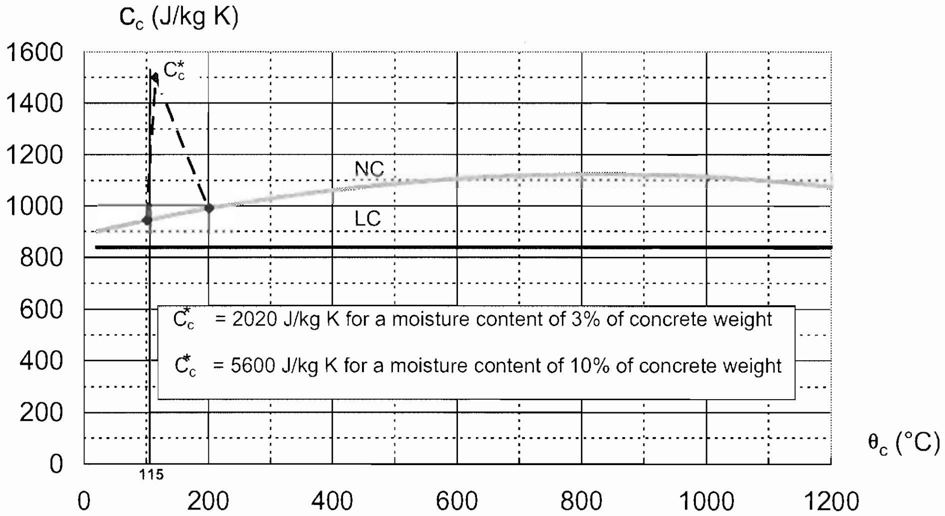


Figure 3.7: Specific heat of normal weight concrete (NC) and lightweight concrete (LC) as a function of the temperature

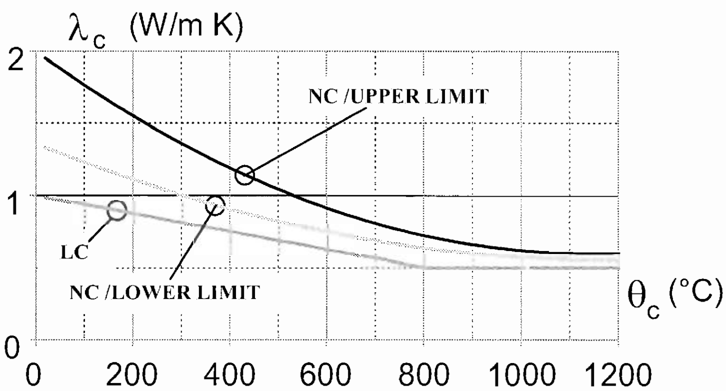


Figure 3.8: Thermal conductivity of normal weight concrete (NC) and lightweight concrete (LC) as a function of the temperature

(8) Where the moisture content is not considered on the level of the heat balance, the equations given in (4) for the specific heat may be completed by a peak value, shown in Figure 3.7, situated between 100°C and 200°C such as at 115°C:

$c_c^* = 2020$ for a moisture content of 3% of concrete weight and [J/kg K] (3.5g)

$c_c^* = 5600$ for a moisture content of 10% of concrete weight. [J/kg K] (3.5h)

The last situation may occur for hollow sections filled with concrete.

(9) The thermal conductivity λ_c of normal weight concrete may be determined between the lower and upper limits given in (10).

NOTE 1: The value of thermal conductivity may be set by the National Annex within the range defined by the lower and upper limits.

NOTE 2: The upper limit has been derived from tests of steel-concrete composite structural elements. The use of the upper limit is recommended.

(10) The upper limit of thermal conductivity λ_c of normal weight concrete may be determined from:

$$\lambda_c = 2 - 0{,}2451\,(\theta_c / 100) + 0{,}0107\,(\theta_c / 100)^2 \quad \text{[W/mK]} \quad \text{for } 20°\text{C} \leq \theta_C \leq 1200°\text{C} \qquad (3.6a)$$

where θ_C is the concrete temperature.

The lower limit of thermal conductivity λ_c of normal weight concrete may be determined from:

$$\lambda_c = 1{,}36 - 0{,}136\,(\theta_c / 100) + 0{,}0057\,(\theta_c / 100)^2 \quad \text{[W/mK]} \quad \text{for } 20°\text{C} \leq \theta_C \leq 1200°\text{C} \qquad (3.6b)$$

where θ_C is the concrete temperature.

(11) The variation of the thermal conductivity with temperature is illustrated in Figure 3.8.

(12) In simple calculation models (see 4.3) the thermal conductivity may be considered to be independent of the concrete temperature. In this case the following value should be taken:

$$\lambda_c = 1{,}60 \quad \text{[W/mK]} \qquad (3.6c)$$

3.3.3 Lightweight concrete

(1) The thermal elongation $\Delta l / l$ of lightweight concrete may be determined from:

$$\Delta l / l = 8 \,.\, 10^{-6}\,(\theta_c - 20) \qquad (3.7)$$

where:

l is the length at room temperature of the lightweight concrete member

Δl is the temperature induced elongation of the lightweight concrete member

θ_c is the lightweight concrete temperature [°C].

(2) The specific heat c_c of lightweight concrete may be considered to be independent of the concrete temperature:

$$c_c = 840 \qquad \text{[J/kg K]} \qquad (3.8)$$

(3) The thermal conductivity λ_c of lightweight concrete may be determined from the following:

$$\lambda_c = 1{,}0 - (\theta_c / 1600) \qquad \text{[W/mK]} \qquad \text{for } 20°\text{C} \le \theta_C \le 800°\text{C} \qquad (3.9a)$$

$$\lambda_c = 0{,}5 \qquad \text{[W/mK]} \qquad \text{for } \theta_C > 800°\text{C} \qquad (3.9b)$$

(4) The variation with temperature of the thermal elongation, the specific heat and the thermal conductivity are illustrated in Figures 3.6, 3.7 and 3.8.

(5) The moisture content of concrete should be taken equal to the equilibrium moisture content. If these data are not available, the moisture content should not exceed 5 % of the concrete weight.

3.3.4 Fire protection materials

(1)P The properties and performance of fire protection materials shall be assessed using the test procedures given in ENV 13381-1, ENV 13381-2, ENV 13381-4, ENV 13381-5 and ENV 13381-6

3.4 Density

(1)P The density of steel ρ_a shall be considered to be independent of the steel temperature. The following value shall be taken:

$$\rho_a = 7850 \qquad \text{[kg/m}^3\text{]} \qquad (3.10)$$

(2) For static loads, the density of concrete ρ_c may be considered to be independent of the concrete temperature. For calculation of the thermal response, the variation of ρ_c in function of the temperature may be considered according to 3.3.2(3) of EN1992-1-2.

NOTE: The variation of ρ_c in function of the temperature may be approximated by

$$\rho_{c,\theta} = 2354 - 23{,}47\,(\theta_c / 100) \qquad (3.11)$$

(3) For unreinforced normal weight concrete (NC) the following value may be taken:

$$\rho_{c,NC} = 2300 \qquad \text{[kg/m}^3\text{]} \qquad (3.12a)$$

(4)P The density of unreinforced lightweight concrete (LC), considered in this Part 1-2 of EN 1994 for structural fire design, shall be in the range of:

$$\rho_{c,LC} = 1600 \ \ to\ \ 2000 \qquad \text{[kg/m}^3\text{]} \qquad (3.12b)$$

Section 4 Design procedures

4.1 Introduction

(1)P The assessment of structural behaviour in a fire design situation shall be based on the requirements of section 5, Constructional details, and on one of the following permitted design procedures:

- recognized design solutions called tabulated data for specific types of structural members;
- simple calculation models for specific types of structural members;
- advanced calculation models for simulating the behaviour of the global structure (see 2.4.4), of parts of the structure (see 2.4.3) or only of a structural member (see 2.4.2).

NOTE: The decision on the use of advanced calculation models in any Country may be found in the National Annex.

(2)P Application of tabulated data and simple calculation models is confined to individual structural members, considered as directly exposed to fire over their full length. Thermal action is taken in accordance with standard fire exposure, and the same temperature distribution is assumed to exist along the length of the structural members. Extrapolation outside the range of experimental evidence is not allowed.

(3) Tabulated data and simple calculation models should give conservative results compared to relevant tests or advanced calculation models.

(4)P Application of advanced calculation models deals with the response to fire of structural members, subassemblies or complete structures and allows - where appropriate - the assessment of the interaction between parts of the structure which are directly exposed to fire and those which are not exposed.

(5)P In advanced calculation models, engineering principles shall be applied in a realistic manner to specific applications.

(6)P Where no tabulated data or simple calculation models are applicable, it is necessary to use either a method based on an advanced calculation model or a method based on test results.

(7)P Load levels are defined by the ratio between the relevant design effect of actions and the design resistance:

$$\eta = \frac{E_d}{R_d} \leq 1{,}0 \text{ ; load level referring to EN 1994-1-1,} \tag{4.1}$$

where:

E_d is the design effect of actions for normal temperature design and

R_d is the design resistance for normal temperature design;

$$\eta_{fi,t} = \frac{E_{fi,d,t}}{R_d} \text{ ; load level for fire design,}$$

where:

$E_{fi,d,t}$ is the design effect of actions in the fire situation, at time t.

(8)P For a global structural analysis (entire structures) the mechanical actions shall be combined using the accidental combination given in 4.3 of EN 1991-1-2.

(9)P For any type of structural analysis according to 2.4.2, 2.4.3 and 2.4.4, load bearing failure "R" is reached, when the design resistance in the fire situation $R_{fi,d,t}$ has decreased to the level of the design effect of actions in the fire situation $E_{fi,d,t}$.

(10) For the design model "Tabulated data" of 4.2, $R_{fi,d,t}$ may be calculated by $R_{fi,d,t} = \eta_{fi,t}\ R_d$.

(11) The simple calculation models for slabs and beams may be based on known temperature distributions through the cross-section, as given in 4.3 and on material properties, as given in section 3.

(12) For slabs and beams where temperature distributions are determined by other appropriate methods or by tests, the resistance of the cross-sections may be calculated directly using the material properties given in section 3, provided instability or other premature failure effects are prevented.

(13) For a beam connected to a slab, the resistance to longitudinal shear provided by transverse reinforcement should be determined from 6.6.6, of EN 1994-1-1. In this case the contribution of the profiled steel sheeting should be ignored when its temperature exceeds 350°C. The effective width b_{eff} at elevated temperatures may be taken as the value in 5.4.1.2 of EN 1994-1-1.

(14) Rule (13) holds if the axis distance of these transverse reinforcements satisfies column 3 in Table 5.8 of EN 1992-1-2.

(15) In this document, columns subjected to fire conditions are assumed to be equally heated all around their cross-section, whereas beams supporting a floor are supposed to be heated only from the three lower sides.

(16) For beams connected to slabs with profiled steel sheets a three side fire exposure may be assumed, when at least 85 % of the upper side of the steel profile is directly covered by the steel sheet.

4.2 Tabulated data

4.2.1 Scope of application

(1) The following rules refer to member analysis according to 2.4.2. They are only valid for the standard fire exposure.

(2) The data given hereafter depend on the load level $\eta_{fi,t}$ following (7)P, (9)P and (10) of 4.1.

(3) The design effect of actions in the fire situation, assumed to be time-independent, may be taken as $E_{fi,d}$ according to (2) of 2.4.2.

(4)P It shall be verified that $E_{fi,d,t} \leq R_{fi,d,t}$

(5) For the tabulated data given in the Tables 4.1 to 4.7, linear interpolation is permitted for all physical parameters.

NOTE: When at present classification is impossible, this is marked by "-" in the tables.

4.2.2 Composite beam comprising steel beam with partial concrete encasement

(1) Composite beams comprising a steel beam with partial concrete encasement (Figure 1.5) may be classified in function of the load level $\eta_{fi,t}$, the beam width b and the additional reinforcement A_S related to the area of bottom flange A_f as given in Table 4.1.

(2) The values given in Table 4.1 are valid for simply supported beams.

(3) When determining R_d and $R_{fi,d,t} = \eta_{fi,t}\, R_d$ in connection with Table 4.1, the following conditions should be observed:

- the thickness of the web e_W does not exceed 1/15 of the width b;
- the thickness of the bottom flange e_f does not exceed twice the thickness of the web e_W;
- the thickness of the concrete slab h_C is at least 120 mm;
- the additional reinforcement area related to the total area between the flange $A_s / (A_c + A_s)$ does not exceed 5 %;
- the value of R_d is calculated on the basis of EN 1994-1-1 provided that:

  the effective slab width b_{eff} does not exceed 5 m,

  the additional reinforcement A_s is not taken into account.

(4) The values given in Table 4.1 are valid for the structural steel grade S355. If another structural steel grade is used, the minimum values for the additional reinforcement given in Table 4.1 should be factored by the ratio of the yield point of this other steel grade to the yield point of grade S355.

(5) The values given in Table 4.1 are valid for the steel grade S500 used for the additional reinforcement A_S.

(6) The values given in Tables 4.1 and 4.2 are valid for beams connected to flat reinforced concrete slabs.

(7) The values given in Tables 4.1 and 4.2 may be used for beams connected to composite floors with profiled steel sheets, if at least 85 % of the upper side of the steel profile is directly covered by the steel sheet. If not, void fillers have to be used on top of the beams.

(8) The material used for void fillers should be suitable for fire protection of steel (see ENV 13381-4 and/or ENV 13381-5).

(9) Additional reinforcement has to be placed as close as possible to the bottom flange taking into account the axis distances u_1 and u_2 of Table 4.2.

Table 4.1: Minimum cross-sectional dimensions b and minimum additional reinforcement in relation to the area of flange A_s / A_f, for composite beams comprising steel beams with partial concrete encasement.

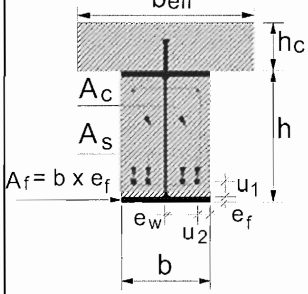
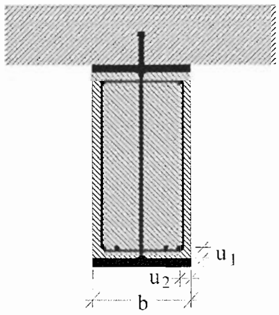
	 Condition for application: slab: $h_c \geq 120$ mm $b_{eff} \leq 5$ m steel section: $b / e_w \geq 15$ $e_f / e_w \leq 2$ additional reinforcement area, related to total area between the flanges: $A_s/(A_c + A_s) \leq 5\%$	**Standard Fire Resistance**				
		R30	**R60**	**R90**	**R120**	**R180**
1	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq 0{,}3$					
	min b [mm] and additional reinforcement A_s in relation to the area of flange A_s / A_f					
1.1	$h \geq 0{,}9 \times$ min b	70/0,0	100/0,0	170/0,0	200/0,0	260/0,0
1.2	$h \geq 1{,}5 \times$ min b	60/0,0	100/0,0	150/0,0	180/0,0	240/0,0
1.3	$h \geq 2{,}0 \times$ min b	60/0,0	100/0,0	150/0,0	180/0,0	240/0,0
2	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq 0{,}5$					
	min b [mm] and additional reinforcement A_s in relation to the area of flange A_s / A_f					
2.1	$h \geq 0{,}9 \times$ min b	80/0,0	170/0,0	250/0,4	270/0,5	-
2.2	$h \geq 1{,}5 \times$ min b	80/0,0	150/0,0	200/0,2	240/0,3	300/0,5
2.3	$h \geq 2{,}0 \times$ min b	70/0,0	120/0,0	180/0,2	220/0,3	280/0,3
2.4	$h \geq 3{,}0 \times$ min b	60/0,0	100/0,0	170/0,2	200/0,3	250/0,3
3	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq 0{,}7$					
	min b [mm] and additional reinforcement A_s in relation to the area of flange A_s / A_f					
3.1	$h \geq 0{,}9 \times$ min b	80/0,0	270/0,4	300/0,6	-	-
3.2	$h \geq 1{,}5 \times$ min b	80/0,0	240/0,3	270/0,4	300/0,6	-
3.3	$h \geq 2{,}0 \times$ min b	70/0,0	190/0,3	210/0,4	270/0,5	320/1,0
3.4	$h \geq 3{,}0 \times$ min b	70/0,0	170/0,2	190/0,4	270/0,5	300/0,8

Table 4.2: Minimum axis distance for additional reinforcement of composite beams.



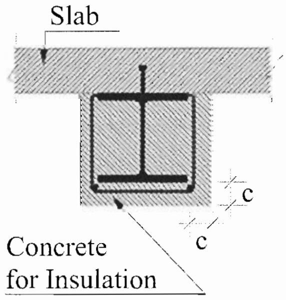
Profile Width b [mm]	Min. Axis Distance [mm]	Standard Fire Resistance R60	R90	R120	R180
170	u_1	100	120	-	-
	u_2	45	60	-	-
200	u_1	80	100	120	-
	u_2	40	55	60	-
250	u_1	60	75	90	120
	u_2	35	50	60	60
$\geq$ 300	u_1	40	50	70	90
	u_2	25*	45	60	60

NOTE: *) This value has to be checked according to 4.4.1.2 of EN 1992-1-1

(10) If the concrete encasing the steel beam has only an insulation function, the fire resistance R30 to R180 may be fulfilled for a concrete cover c of the steel section according to Table 4.3.

NOTE: For R30, concrete need only be placed between the flanges of the steel section.

Table 4.3: Minimum concrete cover for a steel section with concrete acting as fire protection



	Standard Fire Resistance				
	R30	R60	R90	R120	R180
Concrete cover c [mm]	0	25	30	40	50

(11) Where concrete encasing has only an insulation function, fabric reinforcement should be placed according to 5.1(6), except for R30.

4.2.3 Composite columns

4.2.3.1 General

(1) The design Tables 4.4, 4.6 and 4.7 are valid for braced frames.

(2) Load levels $\eta_{fi,t}$ in Tables 4.6 and 4.7 are defined by 4.1(7)P assuming pin-ended supports of the column for the calculation of R_d, provided that both column ends are rotationally restrained in the fire situation. This is generally the case in practice according to Figures 5.3 to 5.6 when assuming that only the level under consideration is submitted to fire conditions.

(3) When using Tables 4.6 and 4.7, R_d has to be based on twice the buckling length used in the fire design situation.

(4) Tables 4.4 to 4.7 are valid both for concentric axial or eccentric loads applied to columns. When determining R_d, the design resistance for normal temperature design, the eccentricity of the load should be considered.

(5) The tabulated data given in Tables 4.4 to 4.7 are valid for columns with a maximum length of 30 times the minimum external dimension of the cross-section chosen.

4.2.3.2 Composite columns made of totally encased steel sections

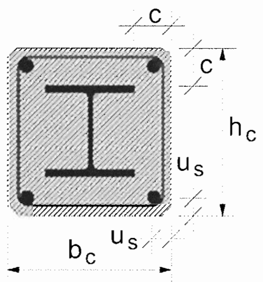
(1) Composite columns made of totally encased steel sections may be classified as a function of the depth b_c or h_c, the concrete cover c of the steel section and the minimum axis distance u_s of the reinforcing bars as given by the two alternative solutions in Table 4.4.

(2) All load levels $\eta_{fi,t}$ may be used when applying (10) of 4.1.

(3) The reinforcement should consist of a minimum of 4 bars with a diameter of 12 mm. In all cases the minimum percentage of longitudinal reinforcing bars should fulfil the requirements of EN 1994-1-1.

(4) The maximum percentage of longitudinal reinforcing bars should fulfil the requirements of EN 1994-1-1. For stirrups it should be referred to EN 1992-1-1.

Table 4.4: Minimum cross-sectional dimensions, minimum concrete cover of the steel section and minimum axis distance of the reinforcing bars, of composite columns made of totally encased steel sections.



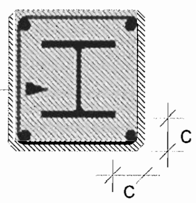
		Standard Fire Resistance					
		R30	R60	R90	R120	R180	R240
1.1	Minimum dimensions h_c and b_c [mm]	150	180	220	300	350	400
1.2	minimum concrete cover of steel section c [mm]	40	50	50	75	75	75
1.3	minimum axis distance of reinforcing bars u_s [mm]	20*	30	30	40	50	50
	or						
2.1	Minimum dimensions h_c and b_c [mm]	-	200	250	350	400	-
2.2	minimum concrete cover of steel section c [mm]	-	40	40	50	60	-
2.3	minimum axis distance of reinforcing bars u_s [mm]	-	20*	20*	30	40	-

NOTE: *) These values have to be checked according to 4.4.1.2 of EN 1992-1-1

(5) If the concrete encasing the steel section has only an insulation function, when designing the column for normal temperature design, the fire resistance R30 to R180 may be fulfilled for a concrete cover c of the steel section according to Table 4.5.

NOTE: For R30, concrete need only be placed between the flanges of the steel section.

Table 4.5: Minimum concrete cover for a steel section with concrete acting as fire protection



Concrete for Insulation	Standard Fire Resistance				
	R30	R60	R90	R120	R180
Concrete cover c [mm]	0	25	30	40	50

(6) Where concrete encasing has only an insulation function, fabric reinforcement should be placed according to 5.1(6), except for R30.

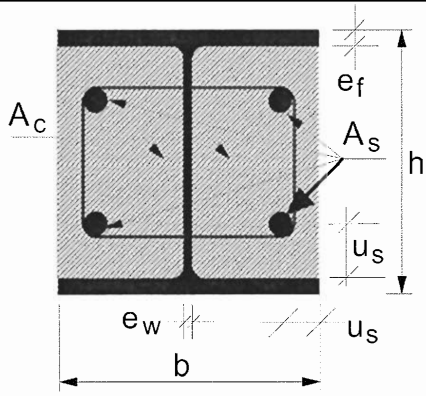
4.2.3.3 Composite columns made of partially encased steel sections

(1) Composite columns made of partially encased steel sections may be classified in function of the load level $\eta_{fi,t}$, the depth b or h, the minimum axis distance of the reinforcing bars u_s and the ratio between the web thickness e_w and the flange thickness e_f as given in Table 4.6.

(2) When determining R_d and $R_{fi,d,t} = \eta_{fi,t}\, R_d$, in connection with Table 4.6, reinforcement ratios $A_s / \left(A_c + A_s\right)$ higher than 6 % or lower than 1 %, should not be taken into account.

(3) Table 4.6 may be used for the structural steel grades S 235, S 275 and S 355.

Table 4.6: Minimum cross-sectional dimensions, minimum axis distance and minimum reinforcement ratios of composite columns made of partially encased steel sections.

		**Standard Fire Resistance**			
		R30	**R60**	**R90**	**R120**
	Minimum ratio of web to flange thickness e_w/e_f	0,5	0,5	0,5	0,5
1	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \le 0{,}28$				
1.1	minimum dimensions h and b [mm]	160	200	300	400
1.2	minimum axis distance of reinforcing bars u_s [mm]	-	50	50	70
1.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	-	4	3	4
2	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \le 0{,}47$				
2.1	minimum dimensions h and b [mm]	160	300	400	-
2.2	minimum axis distance of reinforcing bars u_s [mm]	-	50	70	-
2.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	-	4	4	-
3	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \le 0{,}66$				
3.1	minimum dimensions h and b [mm]	160	400	-	-
3.2	minimum axis distance of reinforcing bars u_s [mm]	40	70	-	-
3.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	1	4	-	-

NOTE: The values of the load level $\eta_{fi,t}$ have been adapted to the design rules for composite columns in EN 1994-1-1.

4.2.3.4 Composite columns made of concrete filled hollow sections

(1) Composite columns made of concrete filled hollow sections may be classified as a function of the load level $\eta_{fi,t}$, the cross-section size b, h or d, the ratio of reinforcement $A_S / (A_C + A_S)$ and the minimum axis distance of the reinforcing bars u_S according to Table 4.7.

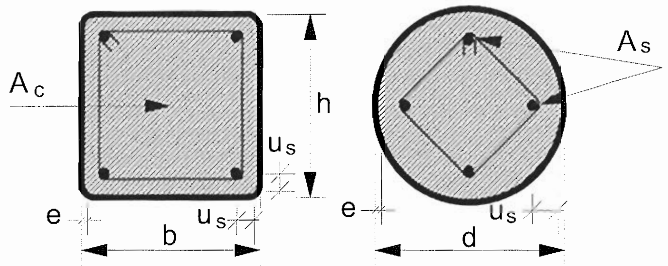
NOTE: Alternatively to this method, the design rules given in 5.3.2 or 5.3.3 of EN1992-1-2 may be used, when neglecting the steel tube.

(2) When calculating R_d and $R_{fi,d,t} = \eta_{fi,t}\ R_d$, in connection with Table 4.7, following rules apply:

- irrespective of the steel grade of the hollow sections, a nominal yield point of 235 N/mm² is taken into account;
- the wall thickness e of the hollow section is considered up to a maximum of 1/25 of b or d;
- reinforcement ratios $A_s / \left(A_c + A_s\right)$ higher than 3 % are not taken into account and
- the concrete strength is considered as for normal temperature design.

(3) The values given in Table 4.7 are valid for the steel grade S 500 used for the reinforcement A_s.

Table 4.7: Minimum cross-sectional dimensions, minimum reinforcement ratios and minimum axis distance of the reinforcing bars of composite columns made of concrete filled hollow sections

	 steel section: (b / e) ≥ 25 or (d / e) ≥ 25	Standard Fire Resistance				
		R30	**R60**	**R90**	**R120**	**R180**
1	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq$ 0,28					
1.1	Minimum dimensions h and b or minimum diameter d [mm]	160	200	220	260	400
1.2	Minimum ratio of reinforcement $A_S / (A_C + A_S)$ in (%)	0	1,5	3,0	6,0	6,0
1.3	Minimum axis distance of reinforcing bars u_S [mm]	-	30	40	50	60
2	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq$ 0,47					
2.1	Minimum dimensions h and b or minimum diameter d [mm]	260	260	400	450	500
2.2	Minimum ratio of reinforcement $A_S / (A_C + A_S)$ in (%)	0	3,0	6,0	6,0	6,0
2.3	Minimum axis distance of reinforcing bars u_S [mm]	-	30	40	50	60
3	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leq$ 0,66					
3.1	Minimum dimensions h and b or minimum diameter d [mm]	260	450	550	-	-
3.2	Minimum ratio of reinforcement $A_S / (A_C + A_S)$ in (%)	3,0	6,0	6,0	-	-
3.3	Minimum axis distance of reinforcing bars u_S [mm]	25	30	40	-	-

NOTE: The values of the load level $\eta_{fi,t}$ have been adapted to the design rules for composite columns in EN 1994-1-1.

4.3 Simple Calculation Models

4.3.1 General rules for composite slabs and composite beams

(1) The following rules refer to member analysis according to 2.4.2. They are only valid for the standard fire exposure.

(2) Rules that are common to composite slabs and composite beams are given hereafter. In addition, rules for slabs are given in 4.3.2 and 4.3.3 and for composite beams are given in 4.3.4.

(3)P For composite beams in which the effective section is Class 1 or Class 2 (see EN 1993-1-1), and for composite slabs, the design bending resistance shall be determined by plastic theory.

(4) The plastic neutral axis of a composite slab or composite beam may be determined from:

$$\sum_{i=1}^{n} A_i k_{y,\theta,i} \left(\frac{f_{y,i}}{\gamma_{M,fi,a}} \right) + \alpha_{slab} \sum_{j=1}^{m} A_j k_{c,\theta,j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right) = 0 \tag{4.2}$$

where:

α_{slab} is the coefficient taking into account the assumption of the rectangular stress block when designing slabs, α_{slab}= 0,85.

$f_{y,i}$ is the nominal yield strength f_y for the elemental steel area A_i, taken as positive on the compression side of the plastic neutral axis and negative on the tension side;

$f_{c,j}$ is the design strength for the elemental concrete area A_j at 20°C. For concrete parts tension is ignored;

$k_{y,\theta,i}$ or $k_{c,\theta,j}$ are as defined in Table 3.2 or Table 3.3.

(5) The design moment resistance $M_{fi,t,Rd}$ may be determined from:

$$M_{fi,t,Rd} = \sum_{i=1}^{n} A_i z_i k_{y,\theta,i} \left(\frac{f_{y,i}}{\gamma_{M,fi}} \right) + \alpha_{slab} \sum_{j=1}^{m} A_j z_j k_{c,\theta,j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right) \tag{4.3}$$

where:

z_i, z_j is the distance from the plastic neutral axis to the centroid of the elemental area A_i or A_j.

(6) For continuous composite slabs and beams, the rules of EN 1992-1-2 and EN 1994-1-1 apply in order to guarantee the required rotation capacity.

4.3.2 Unprotected composite slabs

(1) Typical examples of concrete slabs with profiled steel sheets with or without reinforcing bars are given in Figure 1.1.

(2) The following rules apply to the calculation of the standard fire resistance of both simply supported and continuous concrete slabs with profiled steel sheets and reinforcement, as described below when heated from below according to the standard temperature-time curve.

(3) This method is only applicable to directly heated steel sheets not protected by any insulation and to composite slabs with no insulation between the composite slab and the screed (see Figures 4.1 and 4.2).

NOTE: A method is given in D.4 of Annex D for the calculation of the effective thickness h_{eff}.

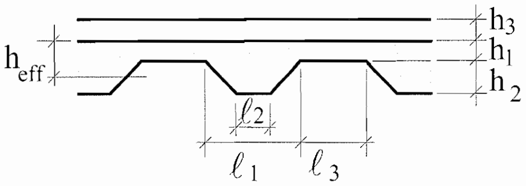


Figure 4.1: Symbols for trapezoidal sheeting

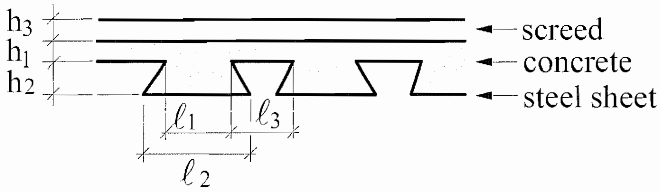


Figure 4.2: Symbols for re-entrant sheeting

(4) The possible effect on the fire resistance of axial restraint is not taken into account in the subsequent rules.

(5) For a design complying with EN 1994-1-1, the fire resistance of composite concrete slabs with profiled steel sheets, with or without additional reinforcement, is at least 30 minutes, when assessed under the load bearing criterion "R" according to (1)P of 2.1.2. For means to verify whether the thermal insulation criterion "I" is fulfilled, see hereafter.

(6) For composite slabs the integrity criterion "E" is assumed to be satisfied.

NOTE 1: In D.1 of Annex D a method is given for the calculation of the fire resistance with respect to the criterion of thermal insulation “I”.

NOTE 2: In D.2 and D.3 of Annex D a method is given for the calculation of the fire resistance with respect to the criterion of mechanical resistance “R” and in relation to the sagging and hogging moment resistances.

(7) Lightweight concrete defined in 3.3.3 and 3.4 may be used.

4.3.3 Protected composite slabs

(1) An improvement of the fire resistance of the composite slab may be obtained by using a protection system applied to the steel sheet in order to decrease the heat transfer to the composite slab.

(2) The performance of the protection system used for a composite slab should be assessed according to:

- ENV 13381-1 for suspended ceilings
- ENV 13381-5 for protection materials

(3) The thermal insulation criterion "I" is assessed by deducing from the effective thickness h_{eff} the equivalent concrete thickness of the protection system (see ENV 13381-5).

(4) The load bearing criterion "R" is fulfilled as long as the temperature of the steel sheet of the composite slab is lower or equal to 350°C, when heated from below by the standard fire.

NOTE: The fire resistance, with regard to the load bearing criterion “R”, of protected composite slabs is at least 30' (see 4.3.2(5)).

4.3.4 Composite beams

4.3.4.1 Structural Behaviour

4.3.4.1.1 General

(1)P Composite beams shall be checked for:

- resistance of critical cross-sections in accordance with 6.1.1(P) of EN 1994-1-1 to bending (4.3.4.1.2);
- vertical shear (4.3.4.1.3);
- resistance to longitudinal shear (4.3.4.1.5).

NOTE: Guidance on critical cross-sections is given in 6.1.1(4)P of EN1994-1-1.

(2) Where in the fire situation, test evidence (see EN 1365 Part 3) of composite action between the floor slab and the steel beam is available, beams which for normal conditions are assumed to be non-composite may be assumed to be composite in fire conditions.

(3) The temperature distribution over the cross-section may be determined from test, advanced calculation models (4.4.2) or for composite beams comprising steel beams with no concrete encasement, from the simple calculation model of 4.3.4.2.2.

4.3.4.1.2 Bending resistance of cross-sections of beams

(1) The design bending resistance may be determined by plastic theory for any class of cross sections except for class 4.

(2) For simply supported beams, the steel flange in compression may be treated, independent of its class, as class 1, provided it is connected to the concrete slab by shear connectors placed in accordance to 6.6.5.5 of EN1994-1-1.

(3) For class 4 steel cross-sections, refer to 4.2.3.6 of EN 1993-1-2.

4.3.4.1.3 Vertical shear resistance of cross-sections of beams

(1)P The resistance to vertical shear shall be taken as the resistance of the structural steel section (see 4.2.3.3(6) and 4.2.3.4(4) of EN 1993-1-2), unless the value of a contribution from the concrete part of the beam has been established by tests.

NOTE: For the calculation of the vertical shear resistance of the structural steel section, a method is given in E.4 of Annex E.

(2) For simply supported beams with webs encased in concrete no check is required provided for normal design the web was assumed to resist all vertical shear.

4.3.4.1.4 Combined bending and vertical shear

(1) For partially encased beams under hogging bending, the web may resist the vertical shear even if this web does not contribute to the moment resistance.

NOTE 1: For partially encased beams under hogging bending, a method is given in F.2(7) of Annex F.

NOTE 2: For composite beams comprising steel beams with no concrete encasement, a method is given in E.2 and E.4 of Annex E.

4.3.4.1.5 Longitudinal Shear Resistance

(1)P The total design longitudinal shear shall be determined in a manner consistent with the design bending resistance, taking account of the difference in the normal force in concrete and in structural steel over a critical length.

(2) In case of design by partial shear connection in the fire situation, the variation of longitudinal shear forces in function of the heating should be considered.

(3) The total design longitudinal shear over the critical length in the area of sagging bending is calculated from the compression force in the slab given by:

$$F_c = \alpha_{slab} \sum_{j=1}^{m} A_j k_{c,\theta,j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right) \tag{4.4}$$

or by the tension force in the steel profile given by:

$$F_a = \sum_{i=1}^{n} A_i k_{y,\theta,i} \left(\frac{f_{y,i}}{\gamma_{M,fi,a}} \right) \text{ whichever is smaller.} \tag{4.5}$$

NOTE: For the calculation of the longitudinal shear in the area of hogging bending, a method is given in E.2 of Annex E.

(4)P Adequate transverse reinforcement shall be provided to distribute the longitudinal shear according to 6.6.6.2 of EN 1994-1-1.

4.3.4.2 Composite beams comprising steel beams with no concrete encasement

4.3.4.2.1 General

(1) The following assessment of the fire resistance of a composite beam comprising a steel beam with no concrete encasement is applicable to simply supported elements and continuous beams (see Figure 1.2).

4.3.4.2.2 Heating of the cross-section

Steel beam

(1) When calculating the temperature distribution of the steel section, the cross section may be divided into various parts according to Figure 4.3.

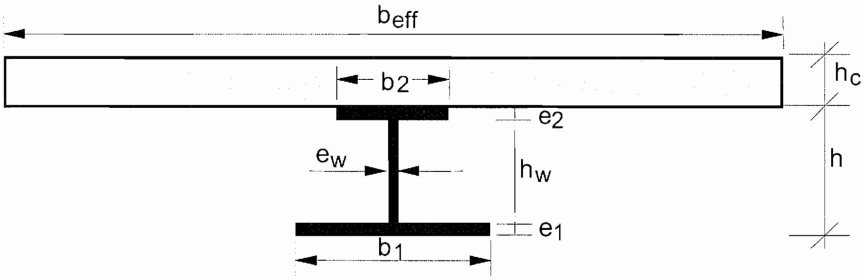


Figure 4.3: Elements of a cross-section

(2) It is assumed that no heat transfer takes place between these different parts nor between the upper flange and the concrete slab.

(3) The increase of temperature $\Delta\theta_{a,t}$ of the various parts of an **unprotected steel beam** during the time interval Δt may be determined from:

$$\Delta\theta_{a,t} = k_{shadow} \left(\frac{1}{c_a \rho_a} \right) \left(\frac{A_i}{V_i} \right) \dot{h}_{net} \, \Delta t \qquad [°C] \tag{4.6}$$

where

k_{shadow} is a correction factor for the shadow effect (see(4))

c_a is the specific heat of steel in accordance with (4) of 3.3.1 [J/kgK]

ρ_a is the density of steel in accordance with (1)P of 3.4 [kg/m^3]

A_i is the exposed surface area of the part i of the steel cross-section per unit length [m²/m]

A_i / V_i is the section factor [m^{-1}] of the part i of the steel cross-section

V_i is the volume of the part i of the steel cross section per unit length [m^3/m]

$\dot{h}_{net}$ is the design value of the net heat flux per unit area in accordance with 3.1 of EN 1991-1-2

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} \qquad [W/m^2]$$

$$\dot{h}_{net,c} = \alpha_c \left(\theta_t - \theta_{a,t} \right) \qquad [W/m^2]$$

$$\dot{h}_{net,r} = \varepsilon_m \, \varepsilon_f \left(5{,}67 \, . \, 10^{-8} \right) \left[\left(\theta_t + 273 \right)^4 - \left(\theta_{a,t} + 273 \right)^4 \right] \qquad [W/m^2]$$

ε_m as defined in 2.2 (2)

ε_f is the emissivity of the fire according to 3.1 (6) of EN 1991-1-2

θ_t is the ambient gas temperature at time t [°C]

$\theta_{a,t}$ is the steel temperature at time t [°C] supposed to be uniform in each part of the steel cross-section

Δt is the time interval [sec]

(4) The shadow effect may be determined from:

$$k_{shadow} = 0{,}9\left(\frac{e_1 + e_2 + 1/2 \cdot b_1 + \sqrt{h_w^{\ 2} + 1/4 \cdot (b_1 - b_2)^2}}{h_w + b_1 + 1/2 \cdot b_2 + e_1 + e_2 - e_w} \right) \tag{4.7}$$

with $e_1, b_1, e_w, h_w, e_2, b_2$ and cross sectional dimensions according to Figure 4.3.

NOTE: The above equation giving the shadow effect (k_{shadow}), and its use in (3), is an approximation, based on the results of a large amount of systematic calculations; for more refined calculation models, the configuration factor concept as presented in 3.1 and Annex G of EN1991-1-2 should be applied.

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

(6) The increase of temperature $\Delta\theta_{a,t}$ of various parts of an **insulated steel beam** during the time interval Δt may be obtained from:

$$\Delta\theta_{a,t} = \left[\left(\frac{\lambda_p / d_p}{c_a \rho_a}\right)\left(\frac{A_{p,i}}{V_i}\right)\left(\frac{1}{1+w/3}\right)\left(\theta_t - \theta_{a,t}\right)\Delta t\right] - \left[\left(e^{w/10} - 1\right)\Delta\theta_t\right] \tag{4.8}$$

with $w = \left(\dfrac{c_p \, \rho_p}{c_a \, \rho_a}\right) d_p \left(\dfrac{A_{p,i}}{V_i}\right)$ and

where:

λ_p	is the thermal conductivity of the fire protection material as specified in (1)P of 3.3.4	[W/mK]
d_p	is the thickness of the fire protection material	[m]
$A_{p,i}$	is the area of the inner surface of the fire protection material per unit length of the part i of the steel member	[m²/m]
c_p	is the specific heat of the fire protection material as specified in (1)P of 3.3.4	[J/kgK]
ρ_p	is the density of the fire protection material	[kg/m³]
θ_t	is the ambient gas temperature at time t	[°C]
$\Delta\theta_t$	is the increase of the ambient gas temperature [°C] during the time interval Δt	

(7) Any negative temperature increase $\Delta\theta_{a,t}$ obtained by (6) should be replaced by zero.

(8) The value of Δt should not be taken as more than 30 seconds for (6).

(9) For non protected members and members with contour protection, the section factor A_i/V_i or $A_{p,i}/V_i$ should be calculated as follows:

for the lower flange:

$$A_i/V_i \text{ or } A_{p,i}/V_i = 2\,(b_1 + e_1)/b_1\,e_1 \tag{4.9a}$$

for the upper flange, when at least 85% of the upper flange of the steel profile is in contact with the concrete slab or, when any void formed between the upper flange and a profiled steel deck is filled with non-combustible material:

$$A_i/V_i \text{ or } A_{p,i}/V_i = (b_2 + 2\,e_2)/b_2\,e_2 \tag{4.9b}$$

for the upper flange when used with a composite floor when less than 85% of the upper flange of the steel profile is in contact with the profiled steel deck:

$$A_i/V_i \text{ or } A_{p,i}/V_i = 2\,(b_2 + e_2)/b_2\,e_2 \tag{4.9c}$$

(10) If the beam depth h does not exceed 500 mm, the temperature of the web may be taken as equal to that of the lower flange.

(11) For members with box-protection, a uniform temperature may be assumed over the height of the profile when using (6) together with A_p/V .

where:

A_p is the area of the inner surface of the box protection per unit length of the steel beam [m²/m]

V is the volume of the complete cross-section of the steel beam per unit length [m³/m]

(12) As an alternative to (6), temperatures in a steel section after a given time of fire duration may be obtained from design flow charts determined in conformity with EN 13381 Part 4 and Part 5.

(13) Protection of a steel beam bordered by a concrete floor on top, may be achieved by a horizontal screen below, and its temperature development may be calculated according to 4.2.5.3 of EN 1993-1-2.

Flat concrete or steel deck-concrete slab system

(14) The following rules (15) to (16) may be used for flat concrete slabs or for steel deck-concrete slab systems with re-entrant or trapezoidal steel sheets.

(15) A uniform temperature distribution may be assumed over the effective width b_{eff} of the concrete slab.

NOTE: In order to determine temperatures over the thickness of the concrete slab a method is given in the Table D.5 of Annex D.

(16) For the mechanical analysis it may be assumed, that for concrete temperatures below 250°C, no strength reduction of concrete is considered.

4.3.4.2.3 Structural behaviour - critical temperature model

(1) In using the following critical temperature model, the temperature of the steel section is assumed to be uniform.

(2)P The method is applicable to symmetric sections of a maximum depth h of 500 mm and to a slab depth h_c not less than 120 mm, used in connection with simply supported beams exclusively subject to sagging bending moments.

(3) The critical temperature θ_{cr} may be determined from the load level $\eta_{fi,t}$ applied to the composite section and from the strength of steel at elevated temperatures $f_{ay,\theta cr}$ according to the relationship:

for R30 $$0{,}9\,\eta_{fi,t} = f_{ay,\theta cr}/f_{ay} \tag{4.10a}$$

in any other case $$1{,}0\,\eta_{fi,t} = f_{ay,\theta cr}/f_{ay} \tag{4.10b}$$

where $\eta_{fi,t} = E_{fi,d,t}/R_d$ and $E_{fi,d,t} = \eta_{fi}\,E_d$ according to (7)P of 4.1 and (3) of 2.4.2.

(4) The temperature rise in the steel section may be determined from (3) or (6) of 4.3.4.2.2 using the section factor A_i/V_i or $A_{p\,i}/V_i$ of the lower flange of the steel section.

4.3.4.2.4 Structural behaviour - bending moment resistance model

(1) As an alternative to 4.3.4.2.3 the bending moment resistance may be calculated by the plastic theory, taking into account the variation of material properties with temperature (see 4.3.4.1.2).

(2) The sagging and hogging moment resistances may be calculated taking into account the degree of shear connection.

NOTE: For the calculation of sagging and hogging moment resistances, a method is given in Annex E.

4.3.4.2.5 Verification of shear resistance of stud connectors

(1) The design shear resistance in the fire situation of a welded headed stud should be determined both for solid and steel deck-concrete slab systems in accordance with EN 1994-1-1, except that the partial factor γ_v should be replaced by $\gamma_{M,fi,v}$ and the smaller of the following reduced values is to be used:

$$P_{fi,Rd} = 0{,}8 \,.\, k_{u,\theta} \,.\, P_{Rd}$$, with P_{Rd} as obtained from equation 6.18 of EN 1994-1-1 or (4.11a)

$$P_{fi,Rd} = k_{c,\theta} \,.\, P_{Rd}$$, with P_{Rd} as obtained from equation 6.19 of EN 1994-1-1 and (4.11b)

where values of $k_{u,\theta}$ and $k_{c,\theta}$ are taken from Tables 3.2 and 3.3 respectively.

(2) The temperature θ_v [°C] of the stud connectors and θ_c [°C] of the concrete may be taken as 80 % and 40 % respectively of the temperature of the upper flange of the beam.

4.3.4.3 Composite beams comprising steel beams with partial concrete encasement

4.3.4.3.1 General

(1) The bending moment resistance of a partially encased steel beam connected to a concrete slab may be calculated using 4.3.4.1.2 or alternatively using the method given hereafter.

(2) The following assessment of the fire resistance of a composite beam, comprising a steel beam with partial concrete encasement according to Figure 1.5, is applicable to simply supported or continuous beams including cantilever parts.

(3) The following rules apply to composite beams heated from below by the standard temperature-time curve.

(4)P The effect of temperatures on material characteristics is taken into account either by reducing the dimensions of the parts composing the cross section or by multiplying the characteristic mechanical properties of materials by a reduction factor.

NOTE: For the calculation of this reduction factor, a method is given in Annex F

(5)P It is assumed that there is no reduction of the shear resistance of the connectors welded to the upper flange, as long as these connectors are fixed directly to the effective width of that flange.

NOTE: For the evaluation of this effective width, a method is given in F.1 of Annex F

(6) This method may be used to classify composite beams in the standard fire classes R30, R60, R90, R120 or R180.

(7) This method may be used in connection with a slab with profiled steel sheets, if for trapezoidal profiles void fillers are used on top of the beams, if re-entrant profiles are chosen or if (16) of 4.1 is fulfilled.

(8) The slab thickness h_c (see Figure 4.4) should be greater than the minimum slab thickness given in Table 4.8. This table may be used for solid and steel deck-concrete slab systems.

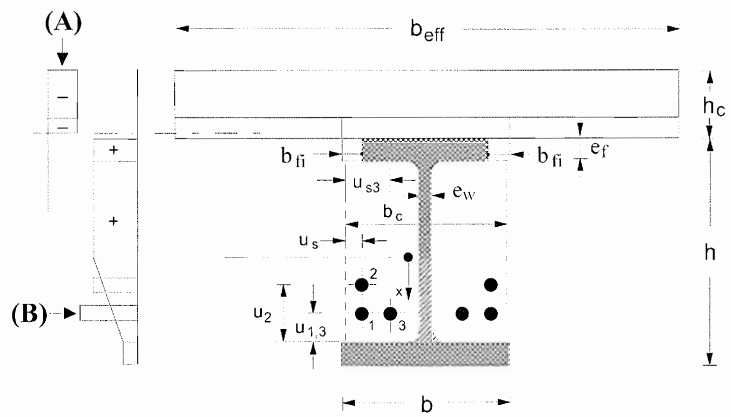
Table 4.8: Minimum slab thickness

Standard Fire Resistance	Minimum Slab Thickness h_c [mm]
R30	60
R60	80
R90	100
R120	120
R180	150

4.3.4.3.2 Structural behaviour

(1) For a simply supported beam, the maximum sagging bending moment produced by loads should be compared to the sagging moment resistance which is calculated according to 4.3.4.3.3.

(2) For the calculation of the sagging moment resistance M_{fi,Rd^+} Figure 4.4 may be considered.



NOTE to Figure 4.4: **(A)** Example of stress distribution in concrete;
(B) Example of stress distribution in steel

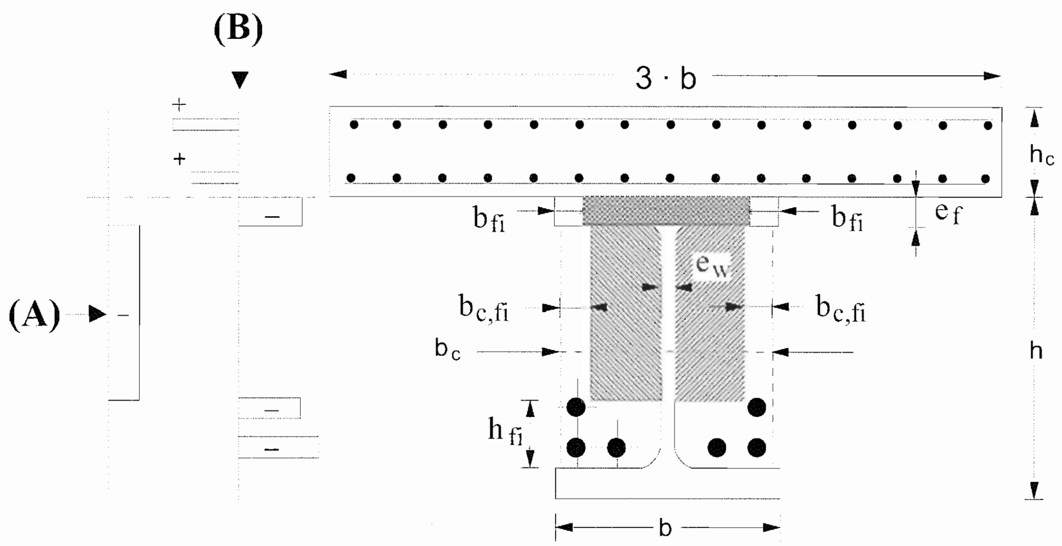
Figure 4.4: Elements of a cross-section for the calculation of the sagging moment resistance

(3)P For a span of a continuous beam, the sagging moment resistance in any critical cross- section and the hogging moment resistance on each support shall be calculated according to 4.3.4.3.3 and 4.3.4.3.4.

(4) For the calculation of the hogging moment resistance M_{fi,Rd^-} Figure 4.5 may be considered.

(5) For the calculation of the moment resistance corresponding to the different fire classes, the following mechanical characteristics may be adopted:

- for the profile, the yield point f_{ay} possibly reduced;
- for the reinforcing bars, the reduced yield point $k_r\ f_{ry}$ or $k_s\ f_{sy}$;
- for the concrete, the compressive cylinder strength f_c.



NOTE to Figure 4.5: (A) Example of stress distribution in concrete;
(B) Example of stress distribution in steel

Figure 4.5: Elements of a cross-section for the calculation of the hogging moment resistance

(6)P The design values of the mechanical characteristics given in (5) are obtained by applying the partial factors given in (1)P of 2.3.

(7) Beams, which are considered as simply supported for normal temperature design, may be considered as continuous in the fire situation if (5) of 5.4.1 is fulfilled.

4.3.4.3.3 Sagging moment resistance $M_{fi,Rd}{}^{+}$

(1) The width b_{eff} of the concrete slab should be equal to the effective width chosen according to 5.4.1.2 of EN 1994-1-1.

(2) In order to calculate the sagging moment resistance, the concrete of the slab in compression, the upper flange of the profile, the web of the profile, the lower flange of the profile and the reinforcing bars should be considered. For each of these parts of the cross section, a corresponding rule may define the effect of the temperature. The concrete in tension of the slab and the concrete between the flanges of the profile should be ignored (see Figure 4.4).

(3) On the basis of the essential equilibrium conditions and on the basis of the plastic theory, the neutral bending axis may be defined and the sagging moment resistance may be calculated.

4.3.4.3.4 Hogging moment resistance $M_{fi,Rd}{}^{-}$

(1) The effective width of the concrete slab is reduced to three times the width of the steel profile (see Figure 4.5). This effective width determines the reinforcing bars to be taken into account.

(2) In order to calculate the hogging moment resistance, the reinforcing bars in the concrete slab, the upper flange of the profile except when (4) is applicable, and the concrete in compression between the flanges of the profile should be considered. For each of these parts of the cross-section a corresponding rule may define the effect of the temperature. The concrete in tension of the slab, the web and the lower flange of the profile should be ignored.

NOTE: For the design of the web, regarding vertical shear, a method is given in F.2 of Annex F.

(3) The reinforcing bars situated between the flanges may participate in compression and be considered in the calculation of the hogging moment resistance, provided the corresponding stirrups fulfil the relevant requirements given in EN 1992-1-1, in order to restrain the reinforcing bars against local buckling, and provided either both the steel profile and the reinforcing bars are continuous at the support or (5) of 5.4.1 is applicable.

(4) In the case of a simply supported beam according to (5) of 5.4.1, the upper flange should not be taken into account if it is in tension.

(5) On the basis of the essential equilibrium conditions and on the basis of the plastic theory, the neutral bending axis may be defined and the hogging moment resistance may be calculated.

(6)P The principles of plastic global analysis apply for the combination of sagging and hogging moments if plastic hinges develop at supports.

(7) Composite beams comprising steel beams with partial concrete encasement may be assumed not to fail through lateral torsional buckling in the fire situation.

4.3.4.4 Steel beams with partial concrete encasement

(1) If the partially encased beam supports a concrete slab, without shear connection according to Figure 1.3, the rules given in 4.3.4.3 may be applied by assuming no mechanical resistance of the reinforced concrete slab.

4.3.5 Composite columns

4.3.5.1 Structural behaviour

(1)P The simple calculation models described hereafter shall only be used for columns in braced frames.

NOTE: EN1994-1-1, 6.7.3.1(1), in all cases limits the relative slenderness $\bar{\lambda}$ for normal design, to a maximum of 2.

(2) In simple calculation models the design value in the fire situation, of the resistance of composite columns in axial compression (buckling load) should be obtained from:

$$N_{fi,Rd} = \chi\, N_{fi,pl,Rd} \tag{4.12}$$

where:

χ is the reduction coefficient for buckling curve c of 6.3.1 of EN 1993-1-1 and depending on the relative slenderness $\bar{\lambda}_\theta$,

$N_{fi,pl,Rd}$ is the design value of the plastic resistance to axial compression in the fire situation.

(3) The cross section of a composite column may be divided into various parts. These are denoted "a" for the steel profile, "s" for the reinforcing bars and "c" for the concrete.

(4) The design value of the plastic resistance to axial compression in the fire situation is given by:

$$N_{fi,pl,Rd} = \sum_j \left(A_{a,\theta}\, f_{ay,\theta}\right)/\gamma_{M,fi,a} + \sum_k \left(A_{s,\theta}\, f_{sy,\theta}\right)/\gamma_{M,fi,s} + \sum_m \left(A_{c,\theta}\, f_{c,\theta}\right)/\gamma_{M,fi,c} \tag{4.13}$$

where:

AC1 $A_{i,\theta}$ is the area of each element of the cross-section ($i = a$ or c or s), which may be affected by the fire. AC1

(5) The effective flexural stiffness is calculated as

$$(EI)_{fi,eff} = \sum_j \left(\varphi_{a,\theta}\, E_{a,\theta}\, I_{a,\theta} \right) + \sum_k \left(\varphi_{s,\theta}\, E_{s,\theta}\, I_{s,\theta} \right) + \sum_m \left(\varphi_{c,\theta}\, E_{c,sec,\theta}\, I_{c,\theta} \right) \tag{4.14}$$

where:

$I_{i,\theta}$ is the second moment of area, of the partially reduced part i of the cross-section for bending around the weak or strong axis,

$\varphi_{i,\theta}$ is the reduction coefficient depending on the effect of thermal stresses.

$E_{c,sec,\theta}$ is the characteristic value for the secant modulus of concrete in the fire situation, given by $f_{c,\theta}$ divided by $\varepsilon_{cu,\theta}$ (see Figure 3.2).

NOTE: A method is given in G.6 of Annex G, for the evaluation of the reduction coefficient of partially encased steel sections.

(6) The Euler buckling load or elastic critical load in the fire situation is as follows

$$N_{fi,cr} = \pi^2 \,(EI)_{fi,eff} \;/\; \ell_\theta^2 \tag{4.15}$$

where:

ℓ_θ is the buckling length of the column in the fire situation.

(7) The relative slenderness is given by:

$$\bar{\lambda}_\theta = \sqrt{N_{fi,pl,R} / N_{fi,cr}} \tag{4.16}$$

where

$N_{fi,pl,R}$ is the value of $N_{fi,pl,Rd}$ according to (4) when the factors $\gamma_{M,fi,a}$, $\gamma_{M,fi,s}$ and $\gamma_{M,fi,c}$ are taken as 1,0.

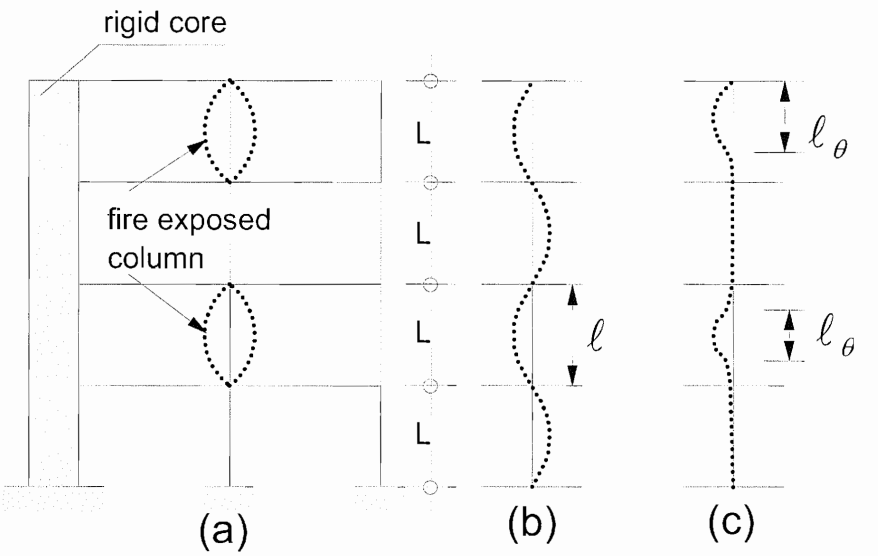
(8) For the determination of the buckling length ℓ_θ of columns, the rules of EN 1994-1-1 apply, with the exception given hereafter.

(9) A column at the level under consideration, fully connected to the column above and below, may be considered as effectively restrained at such connections, provided the resistance to fire of the building elements, which separate the levels under consideration, is at least equal to the fire resistance of the column.

(10) In the case of a composite frame, for which each of the storeys may be considered as a fire compartment with sufficient fire resistance, the buckling length ℓ_θ of a column on an intermediate storey subject to fire is given by L_{ei}. For a column on the top floor subject to fire the buckling length ℓ_θ in the fire situation is given by L_{et} (see Figure 4.6). For a column on the lowest floor subject to fire, the buckling length ℓ_θ may vary, depending on the rotation rigidity of the column base, from L_{ei} to L_{et}.

NOTE1: Values for L_{ei} and L_{et} may be defined in the National Annex. The recommended values are 0,5 and 0,7 times the system length L.

NOTE2: For the buckling length reference may be made to 5.3.2(2) and 5.3.3(3) of EN1992-1-2 and to 4.2.3.2(4) of EN1993-1-2.



a) section through the building
b) deformation mode at room temperature
c) deformation mode at elevated temperature

Figure 4.6: Structural behaviour of columns in braced frames

(11) The following rules apply for composite columns heated all around by the standard temperature-time curve.

4.3.5.2 Steel sections with partial concrete encasement

(1) The fire resistance of columns composed of steel sections with partial concrete encasement according to Figure 1.7 may be assessed by simple calculation models.

NOTE 1: For steel sections with partial concrete encasement, a method is given in Annex G.

NOTE 2: For eccentric loads a method is given in G.7 of Annex G.

(2) For constructional details refer to 5.1, 5.3.1 and 5.4.

4.3.5.3 Unprotected concrete filled hollow sections

(1) The fire resistance of columns composed of unprotected concrete filled square or circular hollow sections may be assessed by simple calculation models.

NOTE 1: For unprotected concrete filled hollow sections, a method is given in Annex H.

NOTE 2: For eccentric loads a method is given in H.4 of Annex H.

(2) For constructional details refer to 5.1, 5.3.2 and 5.4.

4.3.5.4 Protected concrete filled hollow sections

(1) An improvement of the fire resistance of concrete filled hollow sections may be obtained by using a protection system around the steel column in order to decrease the heat transfer.

(2) The performance of the protection system used for concrete filled hollow sections should be assessed according to:

- EN 13381-2 as far as vertical screens are concerned and

- EN 13381-6 as far as coating or sprayed materials are concerned.

(3) The load bearing criterion "R" may be assumed to be met provided the temperature of the hollow section is lower than 350°C.

4.4 Advanced calculation models

4.4.1 Basis of analysis

(1)P Advanced calculation models 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.

NOTE: Compared with tabulated data and simple calculation models, advanced calculation models give an improved approximation of the actual structural behaviour under fire conditions.

(2) Advanced calculation models may be used for individual members, for subassemblies or for entire structures.

(3) Advanced calculation models may be used with any type of cross-section.

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

- the development and distribution of the temperature within structural elements (thermal response model) and

- 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 model (including local buckling, insufficient rotation capacity, spalling and failure in shear), shall be eliminated by appropriate means which may be constructional detailing.

(6) Advanced calculation models may be used when information concerning stress and strain evolution, deformations and / or temperature fields are required.

(7) Advanced calculation models may be used in association with any time-temperature heating curve, provided that the material properties are known for the relevant temperature range.

4.4.2 Thermal response

(1)P Advanced calculation models 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 EN 1991-1-2 and
- the variation of the thermal properties of the materials according to 3.1 and 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 concrete and the fire protection material may conservatively be neglected.

4.4.3 Mechanical response

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

(2)P The mechanical response model shall also take account of:

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

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

(4) Provided that the stress-strain relationships given in 3.1 and 3.2 are used, the effect of high temperature creep need not be given explicit consideration.

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

4.4.4 Validation of advanced calculation models

(1)P The validity of any advanced calculation model shall be verified by applying the following rules (2)P and (4)P.

(2)P A verification of the calculation results shall be made on basis of relevant test results.

(3) Calculation results may refer to deformations, temperatures and fire resistance times.

(4)P The critical parameters shall be checked, by means of a sensitivity analysis, to ensure that the model complies with sound engineering principles.

(5) Critical parameters may refer to the buckling length, the size of the elements, the load level, etc.

Section 5 Constructional details

5.1 Introduction

(1)P Constructional detailing shall guarantee the required level of shear connection between steel and concrete for composite columns and composite beams, for normal temperature design and in the fire situation.

(2)P If this shear connection cannot be maintained under fire conditions, either the steel or the concrete part of the composite section shall fulfil the fire requirements independently.

(3) For concrete-filled hollow sections and partially encased sections, shear connectors should not be attached to the directly heated unprotected parts of the steel sections. However thick bearing blocks with shear studs are accepted (see Figures 5.5 and 5.6).

(4) If welded sections are used, the steel parts directly exposed to fire should be attached to the protected steel parts by sufficiently strong welds.

(5) For fire exposed concrete surfaces, the concrete cover of reinforcing bars defined in 4.4.1 of EN 1992-1-1, should, in all cases, be between 20mm and 50mm. This requirement is needed in order to reduce the danger of spalling under fire exposure.

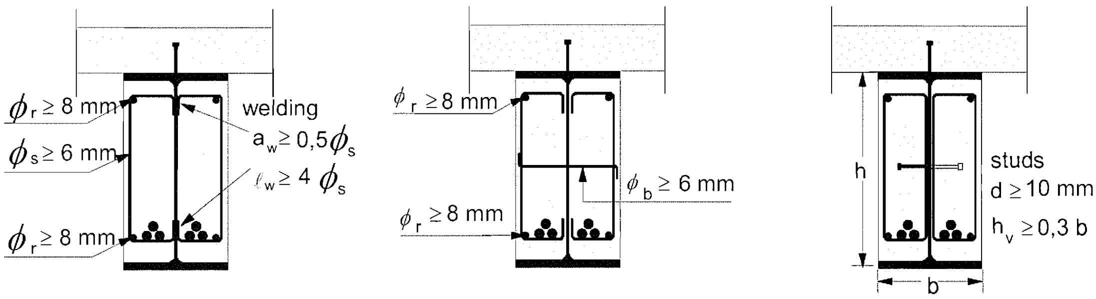
(6) In cases where concrete encasement provides only an insulation function, steel fabric reinforcement with a maximum spacing of 250 mm and a minimum diameter of 4 mm in both directions is to be placed around the section and should fulfil (5).

(7) When the concrete cover of reinforcing bars exceeds 50 mm, a mesh must be placed near the exposed surface to satisfy (5).

5.2 Composite beams

(1)P For composite beams comprising steel beams with partial concrete encasement, the concrete between the flanges shall be reinforced and fixed to the web of the beam.

(2) The partially encased concrete should be reinforced by stirrups of a minimum diameter $\varnothing_S$ of 6 mm or by a reinforcing fabric with a minimum diameter of 4 mm. The concrete cover of the stirrups should not exceed 35 mm. The distance between the stirrups should not exceed 250 mm. In the corners of the stirrups a longitudinal reinforcement of a minimum diameter $\varnothing_r$ of 8 mm should be placed (see Figure 5.1).



a) Welding of stirrups to the web

b) Bars through holes in the web, fixed to the stirrups

c) Welding of studs to the web

Figure 5.1: Measures providing connection between the steel profile and the encasing concrete

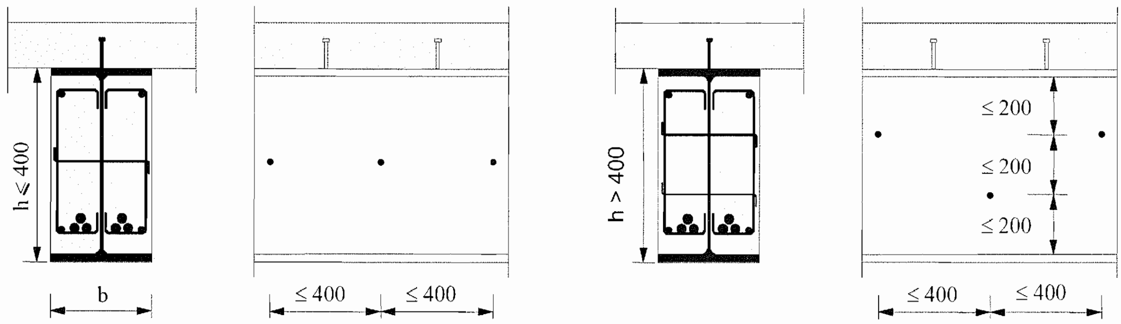
(3) The concrete between the flanges may be fixed to the web by welding the stirrups to the web by a fillet weld with a minimum throat thickness a_w of 0,5 $\varnothing_s$ and a minimum length ℓ_w of 4 $\varnothing_s$ (see Figure 5.1.a).

(4) The concrete between the flanges may be fixed to the web of the beam by means of bars, penetrating the web through holes, or studs welded to both sides of the web under following conditions:

- the bars have a minimum diameter $\varnothing_b$ of 6 mm (see Figures 5.1.b) and

- the studs have a minimum diameter d of 10 mm and a minimum length h_v of 0,3b. Their head should be covered by at least 20 mm of concrete (see Figures 5.1.c);

- the bars or studs are arranged as given in Figure 5.2.a for steel profiles with a maximum depth h of 400 mm or as given in Figure 5.2.b for steel profiles with a depth h larger than 400 mm. When the height is larger than 400 mm, the rows of connectors disposed in staggered way should be at a distance smaller or equal to 200 mm.



a) height of steel profile h ≤ 400 mm

b) height of steel profile h > 400 mm

Figure 5.2: Arrangement of bars or studs providing connection between the steel profile and the encased concrete

5.3 Composite columns

5.3.1 Composite columns with partially encased steel sections

(1)P The concrete between the flanges of the steel sections shall be fixed to the web either by means of stirrups or by studs (see Figure 5.1).

(2) The stirrups should be welded to the web or penetrate the web through holes. If studs are used, they should be welded to the web.

(3) The spacing of studs or stirrups along the column axis should not exceed 500 mm. At load introduction areas this spacing should be reduced according to EN 1994-1-1.

NOTE : For steel sections with a profile depth h greater than 400 mm, studs and stirrups may be chosen according to Figure G.2 of Annex G.

5.3.2 Composite columns with concrete filled hollow sections

(1)P There shall be no additional shear connection along the column, between the beam to column connections.

(2) The additional reinforcement should be held in place by means of stirrups and spacers.

(3) The spacing of stirrups along the column axis should not exceed 15 times the smallest diameter of the longitudinal reinforcing bars.

(4)P The hollow steel section shall contain holes with a diameter of not less than 20 mm located at least one at the top and one at the bottom of the column in every storey.

(5) The spacing of these holes should never exceed 5 m.

5.4 Connections between composite beams and columns

5.4.1 General

(1)P The beam to column connections shall be designed and constructed in such a way that they support the applied forces and moments for the same fire resistance time as that of the member transmitting the actions.

(2) For fire protected members one way of achieving the requirement of (1)P is to apply at least the same fire protection as that of the member transmitting the actions, and to ensure for the connection a load ratio which is less than or equal to that of the beam.

NOTE: For the design of fire protected connections, methods are given in 4.2.1 (6) and Annex D of EN 1993-1-2.

(3) Composite beams and columns may be connected using bearing blocks or shear flats welded to the steel section of the composite column. The beams are supported on the bearing blocks or their webs are bolted to the shear flats. If bearing blocks are used, appropriate constructional detailing should guarantee that the beam cannot slip from supports during the cooling phase.

(4) If connections are made in accordance with Figures 5.4 to 5.6, their fire resistance may be assumed to comply with the requirements of the adjacent structural members. Bearing blocks welded to composite columns may be used with protected steel beams.

(5) In the case of a beam simply supported for normal temperature design, a hogging moment may be developed at the support in the fire situation, provided the concrete slab is reinforced in such a way as to guarantee the continuity of the slab and provided there is an effective transmission of the compression force through the steel connection (see Figure 5.3).

(6) A hogging moment may always be developed according to (5) and Figure 5.3 in the fire situation if

- gap < 10 mm or

- 10 mm ≤ gap < 15 mm, for R30 up to R180 and a beam span larger that 5 m.

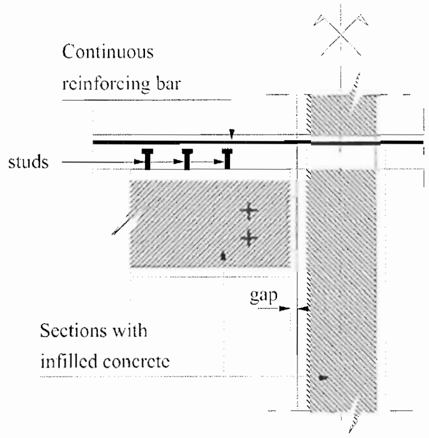


Figure 5.3: Hogging moment connection for fire conditions

5.4.2 Connections between composite beams and composite columns with steel sections encased in concrete

(1) Bearing blocks or shear flats according to Figure 5.4 may be directly welded to the flange of the steel profile of the composite column in order to support a composite beam.

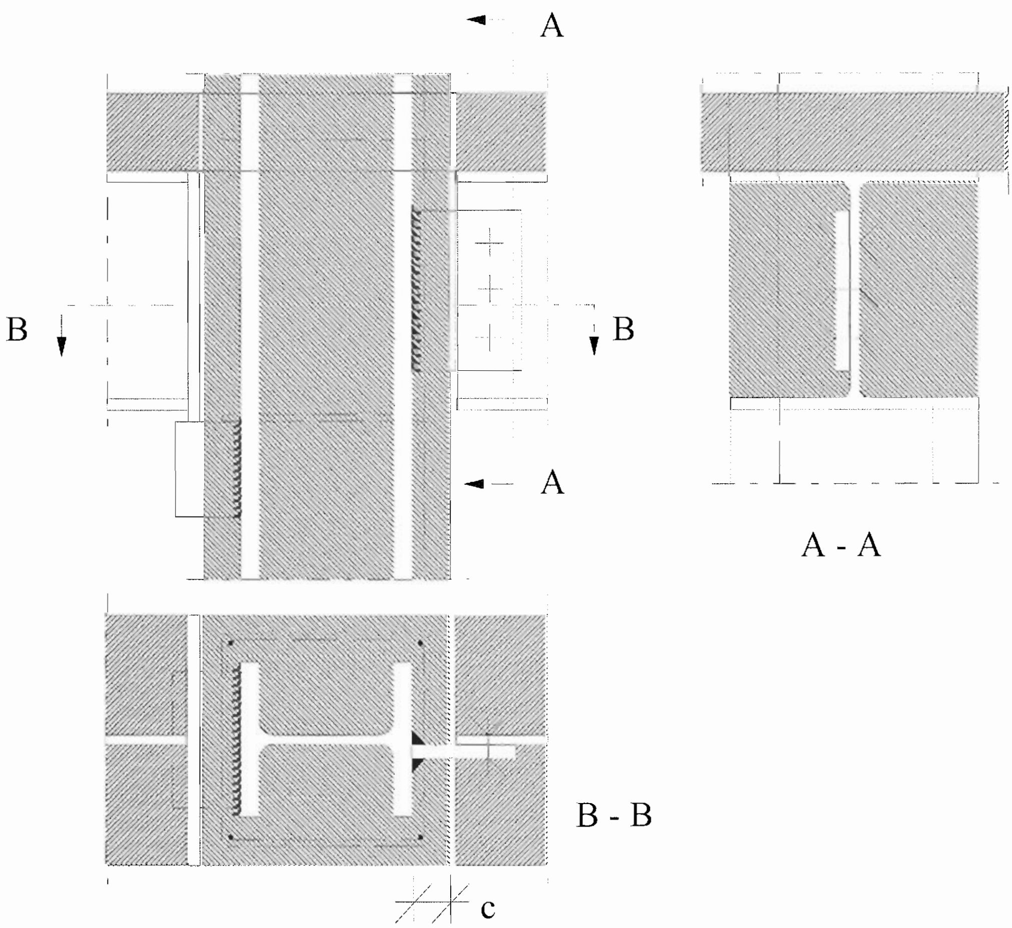


Figure 5.4: Examples of connections to a totally encased steel section of a column.

5.4.3 Connections between composite beams and composite columns with partially encased steel sections.

(1) Additional studs should be provided if unprotected bearing blocks are used (see Figure 5.5.a), because welds are exposed to fire. The shear resistance of studs should be checked according to 4.3.4.2.5 (1) with a stud temperature equal to the average temperature of the bearing block.

(2) For fire resistance classes up to R 120 the additional studs are not needed if the following conditions are fulfilled (see Figure 5.5.b):

- the unprotected bearing block has a minimum thickness of at least 80 mm;
- it is continuously welded on four sides to the column flange;
- the upper weld, protected against direct radiation, has a thickness of at least 1,5 times the thickness of the surrounding welds and should in normal temperature design support at least 40 % of the design shear load.

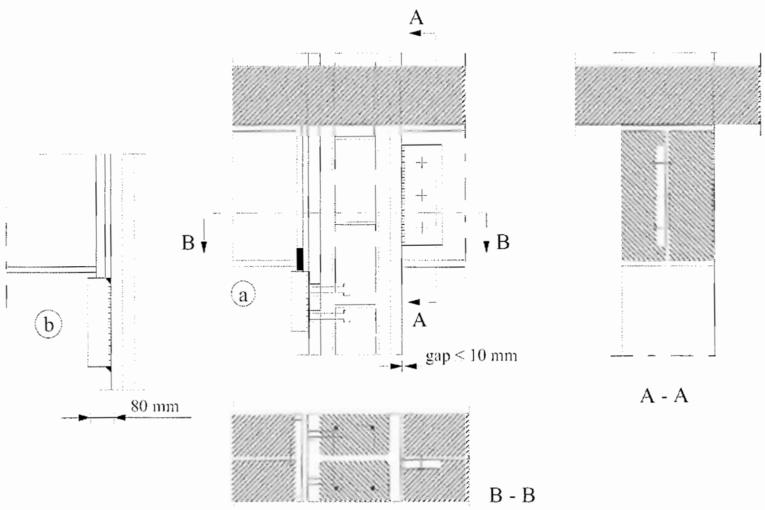


Figure 5.5: Examples of connections to a partially encased steel section

(3) If shear flats are used, the remaining gap between beam and column needs no additional protection if smaller than 10 mm (see Figure 5.5.a).

(4) For different types of connections, refer to (1)P of 5.4.1.

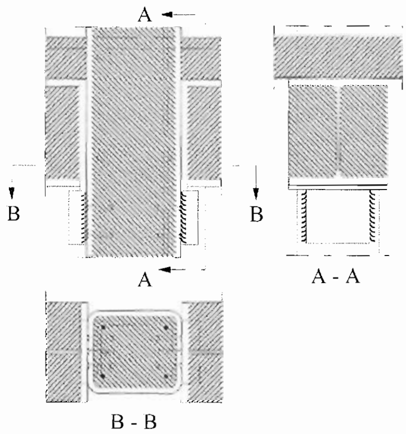
5.4.4 Connections between composite beams and composite columns with concrete filled hollow sections

(1) Composite beams may be connected to composite columns with concrete filled hollow sections using either bearing blocks or shear flats (see Figure 5.6).

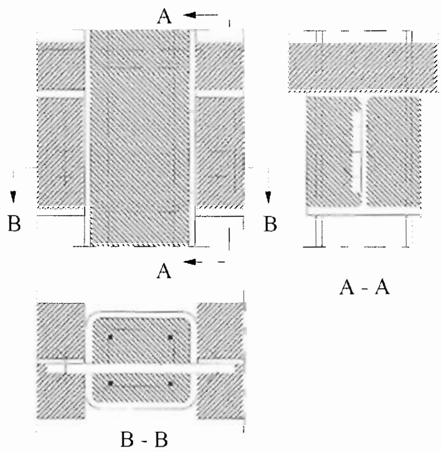
(2)P Shear and tension forces shall be transmitted by adequate means from the beam to the reinforced concrete core of this composite column type.

(3) If bearing blocks are used (see Figure 5.6.a) the shear load transfer in case of fire should be ensured by means of additional studs. The shear resistance of studs should be checked according to 4.3.4.2.5(1) with a stud temperature equal to the average temperature of the bearing block.

(4) If shear flats are used (see Figure 5.6.b), they should penetrate the column and they should be connected to both walls by welding.



a) Bearing blocks with additional studs



b) Penetrating shear flats

Figure 5.6: Examples of connections to a concrete filled hollow section

Annex A
[informative]

Stress-strain relationships at elevated temperatures for structural steels.

(1) A graphical display of the stress-strain relationships for the steel grade S235 is presented in Figure A.1 up to a maximum strain of $\varepsilon_{ay,\theta}$ = 2 %. This presentation corresponds to ranges I and II of Figure 3.1 and to the tabulated data of Table 3.2 without strain-hardening, as specified in 3.2.1.

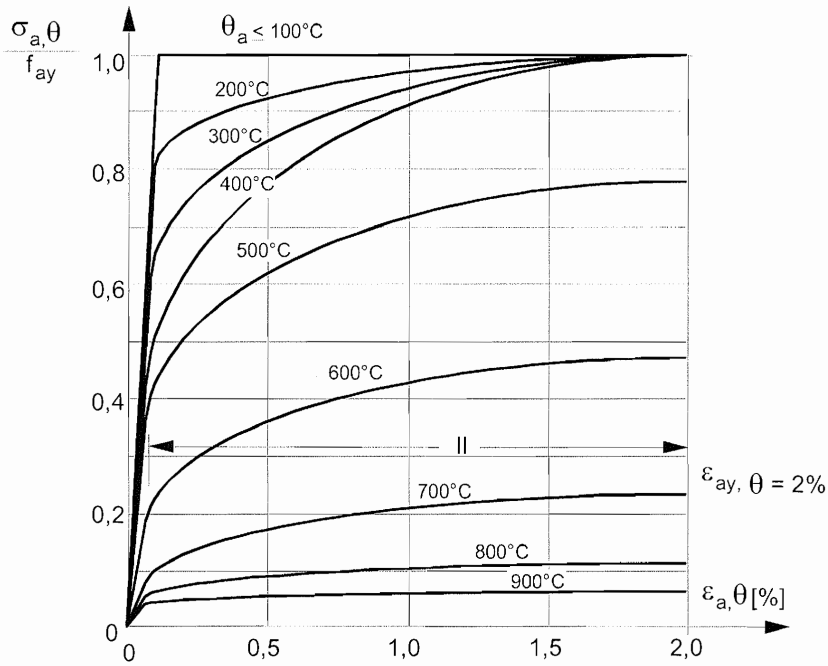


Figure A.1: Graphical presentation of the stress-strain relationships for the steel grade S235 up to a strain of 2 %.

(2) For steel grades S235, S275, S355, S420 and S460 the stress strain relationships may be evaluated up to a maximum strain of 2 % through the equations presented in Table 3.1.

(3) For temperatures below 400°C, the alternative strain-hardening option mentioned in (4) of 3.2.1. may be used as follows in (4), (5) and (6).

(4) A graphical display of the stress-strain relationships, strain-hardening included, is given in Figure A.2 where:

- for strains up to 2 %, Figure A.2 is in conformity with Figure A.1 (range I and II);
- for strains between 2 % and 4 %, a linear increasing branch is assumed (range IIIa);
- for strains between 4 % and 15 % (range IIIb) an horizontal plateau is considered with $\varepsilon_{au,\theta}$ = 15%;
- for strains between 15 % and 20 % a decreasing branch (range IV) is considered with $\varepsilon_{ae,\theta}$ = 20 %.

(5) The tensile strength at elevated temperature $f_{au,\theta}$ allowing for strain-hardening (see Figure A.3), may be determined as follows:

$$\theta_a \leq 300°C; \qquad f_{au,\theta} = 1{,}25\, f_{ay} \tag{A.1}$$

$$300 < \theta_a \leq 400°C; \qquad f_{au,\theta} = f_{ay}\,(2 - 0{,}0025\,\theta_a) \tag{A.2}$$

$$\theta_a \geq 400°C; \qquad f_{au,\theta} = f_{ay,\theta} \tag{A.3}$$

(6) For strains $\varepsilon_{a,\theta}$ higher than 2 % the stress-strain relationships allowing for strain-hardening may be determined as follows:

$$2\,\% < \varepsilon_{a,\theta} < 4\,\% \qquad \sigma_{a,\theta} = \left[\left(f_{au,\theta} - f_{ay,\theta}\right)/0{,}02\right]\ \varepsilon_{a,\theta} - f_{au,\theta} + 2\, f_{ay,\theta} \tag{A.4}$$

$$4\,\% \leq \varepsilon_{a,\theta} \leq 15\,\% \qquad \sigma_{a,\theta} = f_{au,\theta} \tag{A.5}$$

$$15\,\% < \varepsilon_{a,\theta} < 20\,\% \qquad \sigma_{a,\theta} = \left[1 - \left(\left(\varepsilon_{a,\theta} - 0{,}15\right)/0{,}05\right)\right] f_{au,\theta} \tag{A.6}$$

$$\varepsilon_{a,\theta} \geq 20\,\% \qquad \sigma_{a,\theta} = 0 \tag{A.7}$$

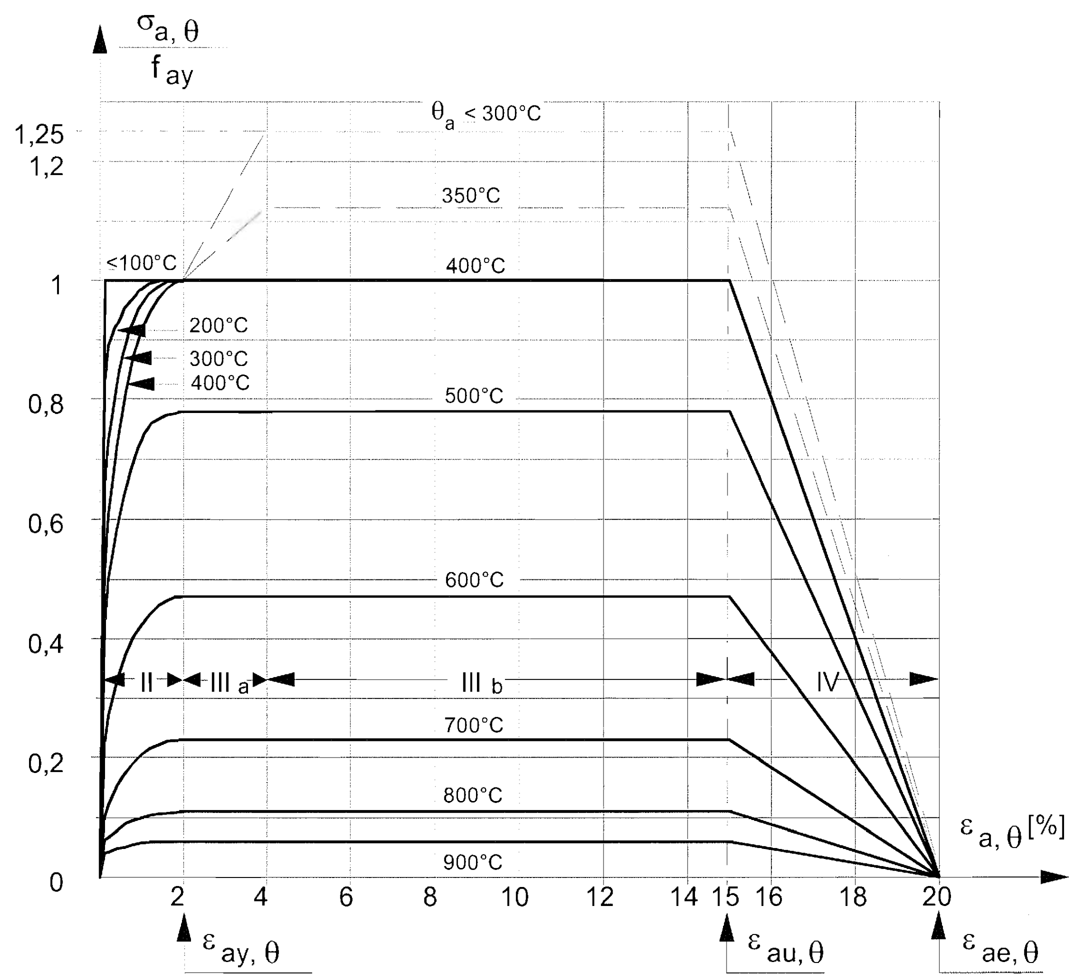


Figure A.2: Graphical presentation of the stress-strain relationships of structural steel at elevated temperatures, strain-hardening included.

(7) The main parameters $E_{a,\theta}$, $f_{ap,\theta}$, $f_{ay,\theta}$, and $f_{au,\theta}$ of the alternative strain-hardening option may be obtained from the reduction factors k_θ of Figure A.3.

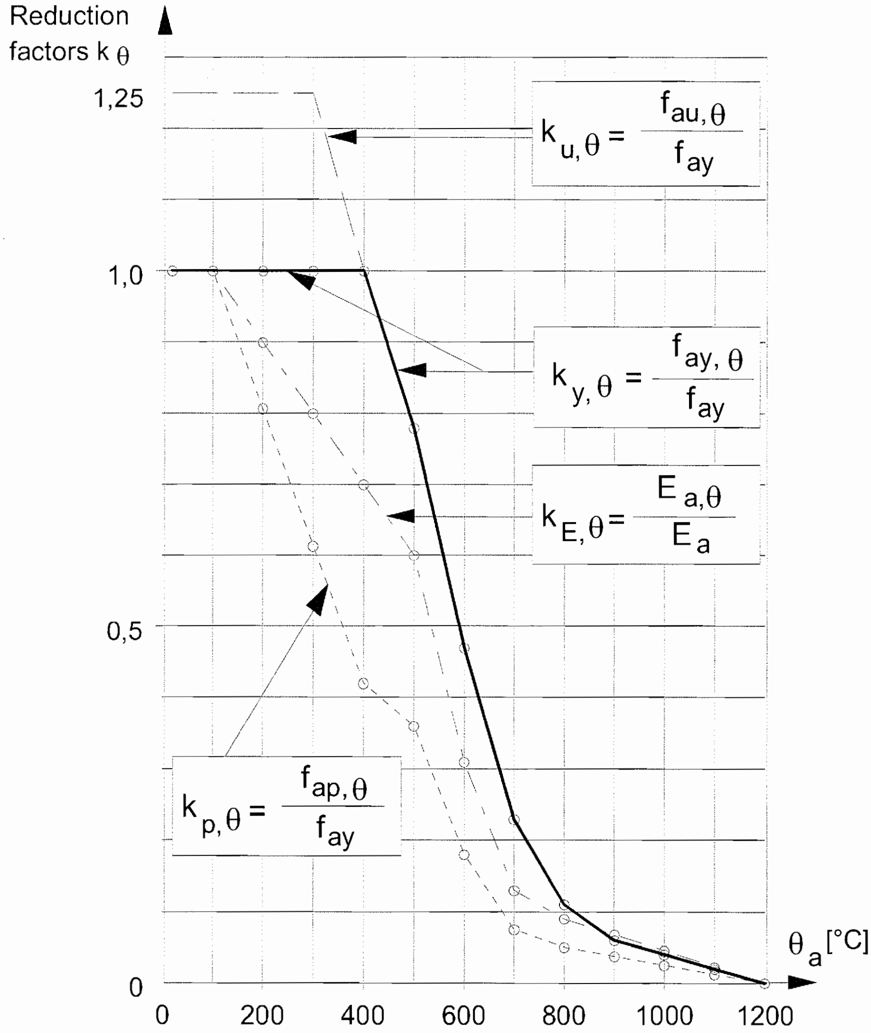


Figure A.3: Reduction factors k_θ for stress-strain relationships allowing for strain-hardening of structural steel at elevated temperatures (see also Table 3.2 of 3.2.1).

Annex B
[informative]

Stress-strain relationships at elevated temperatures for concrete with siliceous aggregates

(1) A graphical display of the stress-strain relationships for concrete with siliceous aggregates is presented in Figure B.1 up to a maximum strain of $\varepsilon_{ce,\theta}$ = 4,75 %. This presentation corresponds to the mathematical formulation of Figure 3.2 and to the tabulated data of Table 3.3 as specified in 3.2.2.

(2) The permitted range and the recommended values of $\varepsilon_{cu,\theta}$ strain corresponding to $f_{c,\theta}$ according to Figure 3.2, may be taken from Table B.1.

(3) The recommended values of $\varepsilon_{ce,\theta}$ may be taken from Table B.1.

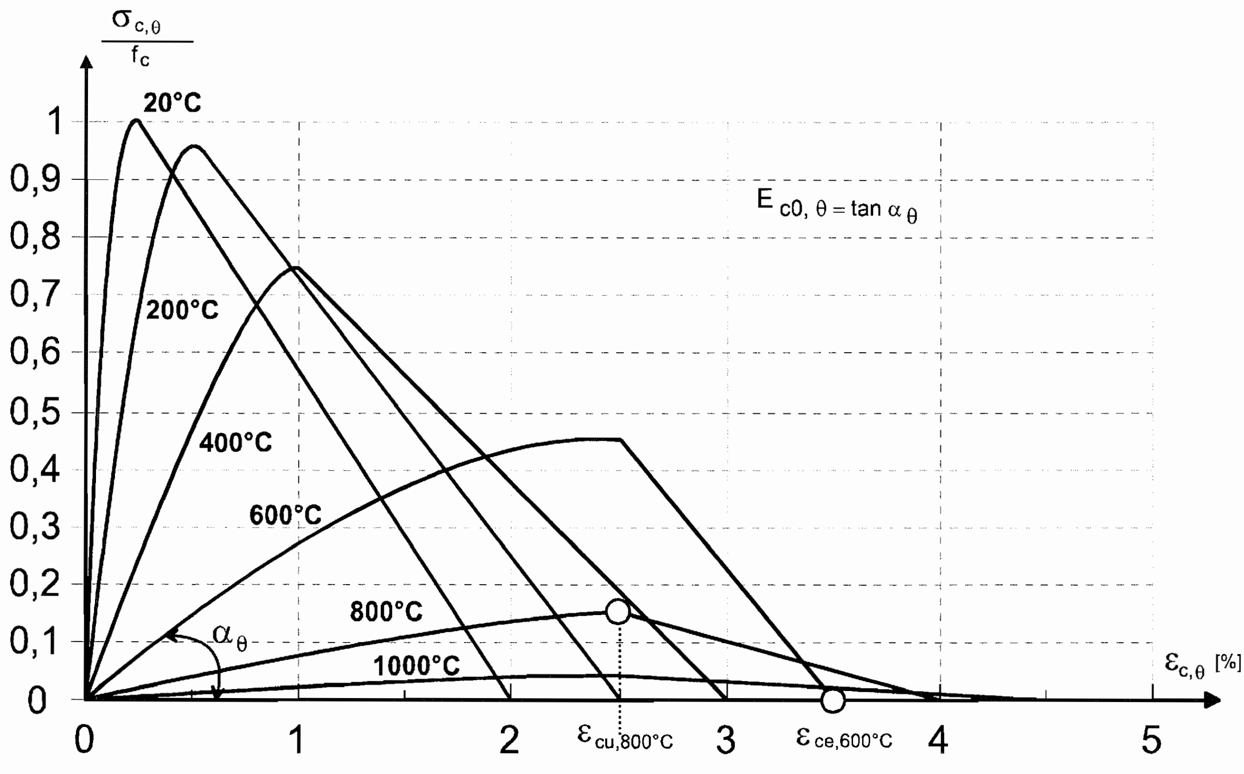


Figure B.1: Graphical presentation of the stress-strain relationships for concrete with siliceous aggregates with a linear descending branch, including the recommended values $\varepsilon_{cu,\theta}$ and $\varepsilon_{ce,\theta}$ of Table B.1.

Table. B.1: Parameters $\varepsilon_{cu,\theta}$ and $\varepsilon_{ce,\theta}$ defining the recommended range of the descending branch, for the stress-strain relationships of concrete at elevated temperatures.

Concrete temperature θ_c [°C]	$\varepsilon_{cu,\theta} \cdot 10^3$ recommended value	$\varepsilon_{ce,\theta} \cdot 10^3$ recommended value
20	2,5	20,0
100	4,0	22,5
200	5,5	25,0
300	7,0	27,5
400	10	30,0
500	15	32,5
600	25	35,0
700	25	37,5
800	25	40,0
900	25	42,5
1000	25	45,0
1100	25	47,5
1200	-	-

(4) The main parameters $f_{c,\theta}$ and $\varepsilon_{cu,\theta}$ of the stress-strain relationships at elevated temperatures, for normal concrete with siliceous aggregates and for lightweight concrete, may be illustrated by Figure B.2. The compressive strength $f_{c,\theta}$ and the corresponding strain $\varepsilon_{cu,\theta}$ define completely range I of the material model together with the equations of Figure 3.2 (see also Table 3.3 of 3.2.2).

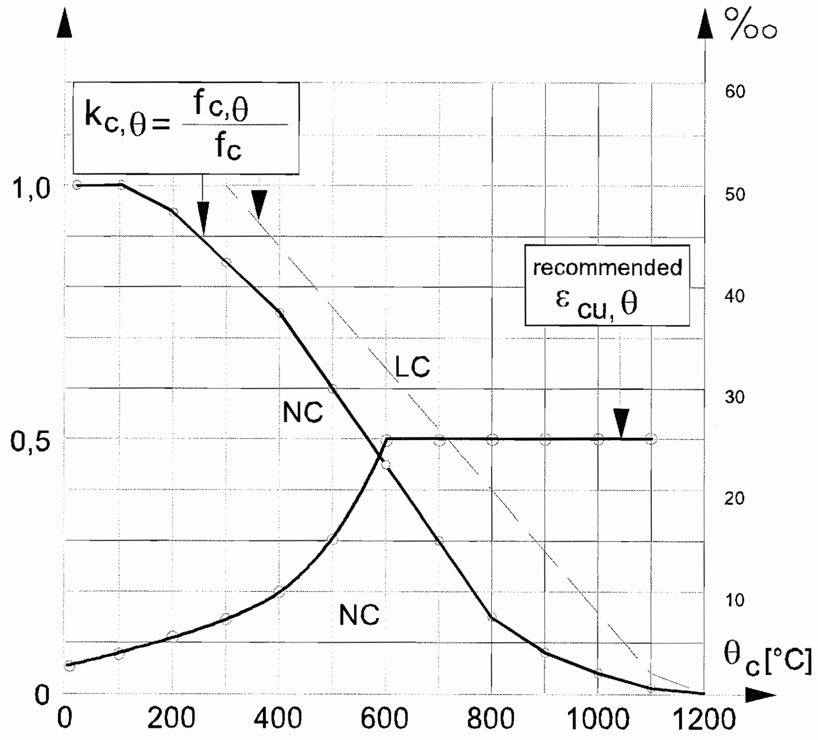


Figure B.2: Parameters for stress-strain relationships at elevated temperatures of normal concrete (NC) and lightweight concrete (LC).

Annex C
[informative]

Concrete stress-strain relationships adapted to natural fires with a decreasing heating branch for use in advanced calculation models.

(1) Following heating to a maximum temperature of θ_{max}, and subsequent cooling down to ambient temperature of 20°C, concrete does not recover its initial compressive strength f_c.

(2) When considering the descending branch of the concrete heating curve (see Figure C.1), the value of $\varepsilon_{cu,\theta}$ and the value of the slope of the descending branch of the stress-strain relationship may both be maintained equal to the corresponding values for θ_{max} (see Figure C.2).

(3) The residual compressive strength of concrete heated to a maximum temperature θ_{max} and having cooled down to the ambient temperature of 20°C, may be given as follows:

$f_{c,\theta,\,20°C} = \varphi\, f_c$ where for (C.1)

$$20^\circ C \le \theta_{max} < 100^\circ C; \qquad \varphi = k_{c,\theta\,max} \tag{C.2}$$

$$100^\circ C \le \theta_{max} < 300^\circ C; \qquad \text{AC1} \ \varphi = 1{,}0 - [0{,}235\,(\theta_{max} - 100)/200] \ \text{AC1} \tag{C.3}$$

$$\theta_{max} \ge 300^\circ C; \qquad \varphi = 0{,}9\, k_{c,\theta\,max} \tag{C.4}$$

Note: The reduction factor $k_{c,\theta\,max}$ is taken according to (4) of 3.2.2.

(4) During the cooling down of concrete with $\theta_{max} \ge \theta \ge 20°C$, the corresponding compressive cylinder strength $f_{c,\theta}$ may be interpolated in a linear way between $f_{c,\theta\,max}$ and $f_{c,\theta,20°C}$.

(5) The above rules may be illustrated in Figure C.2 for a concrete grade C40/50 as follows:

θ_1 = 200°C;	$f_{c,\theta 1}$ = 0,95 . 40 = 38	[N/mm²]	(C.5)
	$\varepsilon_{cu,\theta 1}$ = 0,55	[%]	(C.6)
	$\varepsilon_{ce,\theta 1}$ = 2,5	[%]	(C.7)
θ_2 = 400°C;	$f_{c,\theta 2}$ = 0,75 . 40 = 30	[N/mm²]	(C.8)
	$\varepsilon_{cu,\theta 2}$ = 1	[%]	(C.9)
	$\varepsilon_{ce,\theta 2}$ = 3,0	[%]	(C.10)

For a possible maximum concrete temperature of θ_{max} = 600°C:

$f_{c,\theta\,max}$ = 0,45 . 40 = 18	[N/mm²]	(C.11)
$\varepsilon_{cu,\theta\,max}$ = 2,5	[%]	(C.12)
$\varepsilon_{ce,\theta\,max}$ = 3,5	[%]	(C.13)

For any lower temperature obtained during the subsequent cooling down phase as for θ_3 = 400°C:

$$f_{c,\theta,\,20°C} = \left(0{,}9\,k_{c,\theta\,max}\right) f_c = 0{,}9\,.\,0{,}45\,.\,40 = 16{,}2 \qquad [\text{N/mm}^2] \qquad (C.14)$$

$$f_{c,\theta 3} = f_{c,\theta\,max} - \left[\left(f_{c,\theta\,max} - f_{c,\theta,20°C}\right)\left(\theta_{max} - \theta_3\right) / \left(\theta_{max} - 20\right)\right] = 17{,}4 \qquad [\text{N/mm}^2] \qquad (C.15)$$

$$\varepsilon_{cu,\theta 3} = \varepsilon_{cu,\theta\,max} = 2{,}5 \qquad [\%] \qquad (C.16)$$

$$\varepsilon_{ce,\theta 3} = \varepsilon_{cu,\theta 3} + \left[\left(\varepsilon_{ce,\theta\,max} - \varepsilon_{cu,\theta\,max}\right) f_{c,\theta 3} / f_{c,\theta\,max}\right] = 3{,}46 \qquad [\%] \qquad (C.17)$$

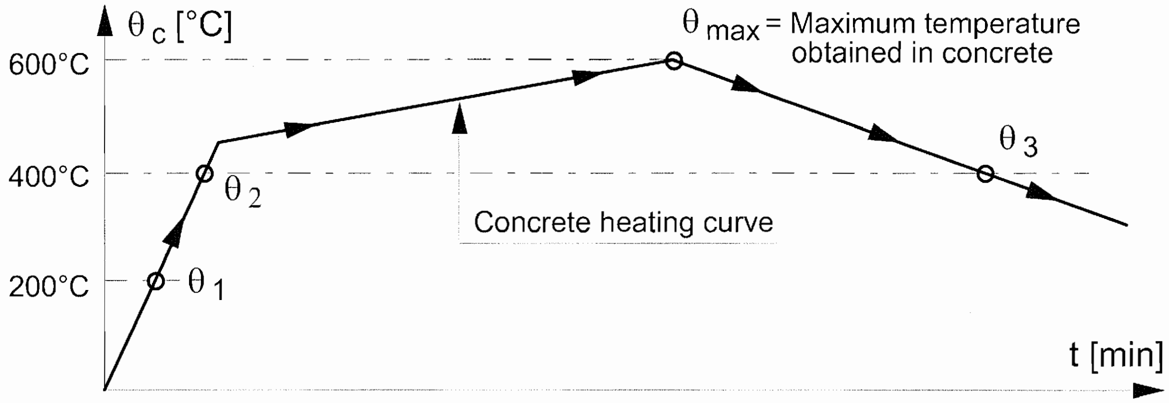


Figure C.1: Example of concrete heating and cooling

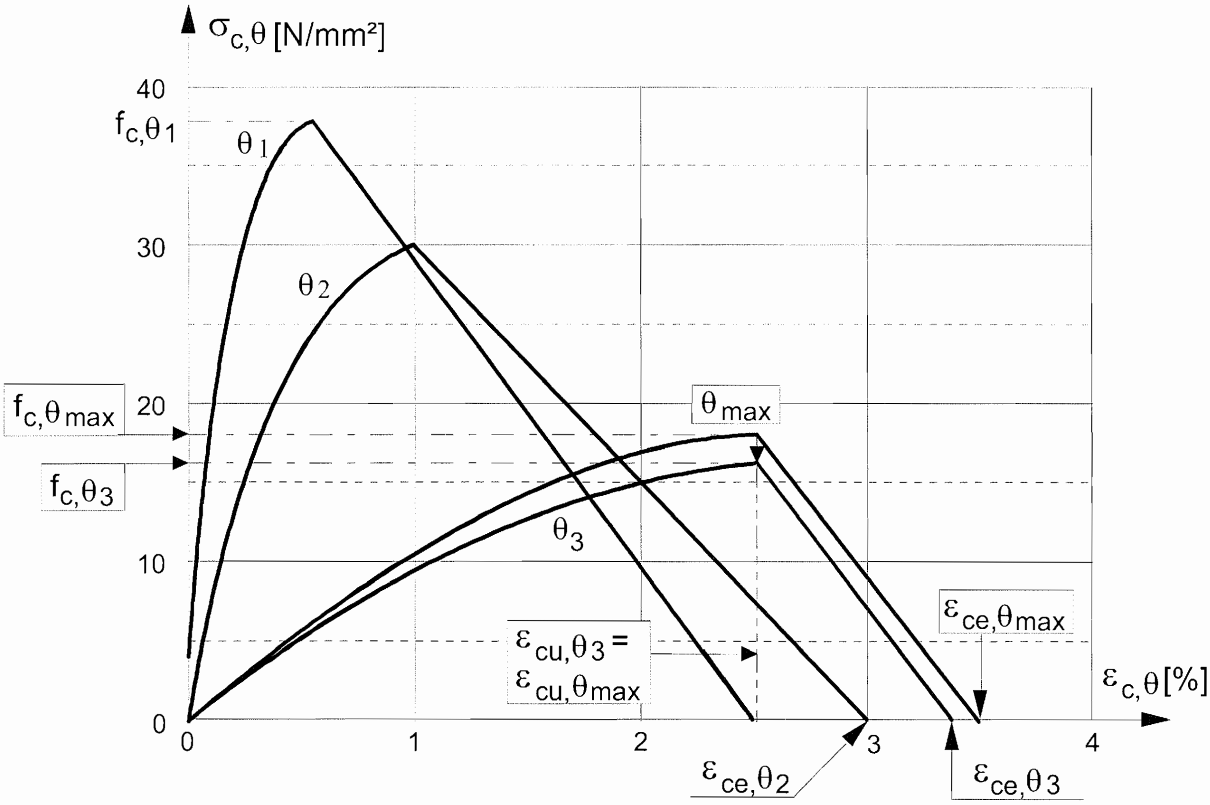


Figure C.2: Stress-strain relationships of the concrete strength class C40/50, heated up to θ_1 = 200°C , θ_2 = 400°C , θ_{max} = 600°C and cooled down to θ_3 = 400°C .

Annex D
[Informative]

Model for the calculation of the fire resistance of unprotected composite slabs exposed to fire beneath the slab according to the standard temperature-time curve

D.1 Fire resistance according to thermal insulation

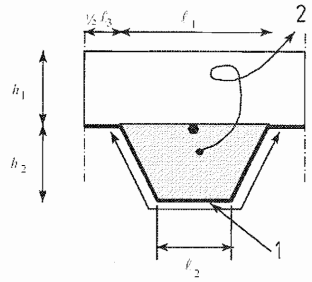
(1) The fire resistance with respect to both the average temperature rise (=140°C) and the maximum temperature rise (=180°C), criterion "I", may be determined according to the following equation:

$$t_i = a_0 + a_1 \cdot h_1 + a_2 \cdot \Phi + a_3 \cdot \frac{A}{L_r} + a_4 \cdot \frac{1}{\ell_3} + a_5 \cdot \frac{A}{L_r} \cdot \frac{1}{\ell_3} \qquad \text{(D.1)}$$

where:

t_i	the fire resistance with respect to thermal insulation	[min]
A	concrete volume of the rib per metre of rib length	[mm^3/m]
L_r	exposed area of the rib per metre of rib length	[mm^2/m]
A/L_r	the rib geometry factor	[mm]
Φ	the view factor of the upper flange	[-]
ℓ_3	the width of the upper flange (see Figure D.1)	[mm].

For the factors a_i, for different values of the concrete depth h_1, for both normal and lightweight concrete, refer to Figure D.1 and Table D.1. For intermediate values, linear interpolation is allowed.



$$\frac{A}{L_r} = \frac{h_2 \cdot \left(\dfrac{\ell_1 + \ell_2}{2} \right)}{\ell_2 + 2\sqrt{h_2^2 + \left(\dfrac{\ell_1 - \ell_2}{2} \right)^2}} \qquad \text{(D.2)}$$

Key
1 – Exposed surface: L_r
2 – Area: A

Figure D.1: Definition of the rib geometry factor A/L_r for ribs of composite slabs.

Table D.1: Coefficients for determination of the fire resistance with respect to thermal insulation

	a_0 [min]	a_1 [min/mm]	a_2 [min]	a_3 [min/mm]	a_4 [mm min]	a_5 [min]
Normal weight concrete	-28,8	1,55	-12,6	0,33	-735	48,0
Lightweight concrete	-79,2	2,18	-2,44	0,56	-542	52,3

(2) The configuration or view factor Φ of the upper flange may be determined as follows:

$$\Phi = \left(\sqrt{h_2^2 + \left(l_3 + \frac{l_1 - l_2}{2} \right)^2} - \sqrt{h_2^2 + \left(\frac{l_1 - l_2}{2} \right)^2} \right) \Big/ l_3 \qquad [-] \qquad (D.3)$$

D.2 Calculation of the sagging moment resistance $M_{fi,Rd}^{+}$

(1) The temperatures θ_a of the lower flange, web and upper flange of the steel decking may be given by:

$$\theta_a = b_0 + b_1 \cdot \frac{1}{\ell_3} + b_2 \cdot \frac{A}{L_r} + b_3 \cdot \Phi + b_4 \cdot \Phi^2 \qquad (D.4)$$

where:

θ_a is the temperature of the lower flange, web or upper flange [°C]

For factors b_i, for both normal and lightweight concrete, refer to Table D.2. For intermediate values, linear interpolation is allowed.

Table D.2: Coefficients for the determination of the temperatures of the parts of the steel decking

Concrete	Fire resistance [min]	Part of the steel sheet	b_0 [°C]	b_1 [°C]. mm	b_2 [°C]. mm	b_3 [°C]	b_4 [°C]
Normal weight concrete	60	Lower flange	951	-1197	-2,32	86,4	-150,7
		Web	661	-833	-2,96	537,7	-351,9
		Upper flange	340	-3269	-2,62	1148,4	-679,8
	90	Lower flange	1018	-839	-1,55	65,1	-108,1
		Web	816	-959	-2,21	464,9	-340,2
		Upper flange	618	-2786	-1,79	767,9	-472,0
	120	Lower flange	1063	-679	-1,13	46,7	-82,8
		Web	925	-949	-1,82	344,2	-267,4
		Upper flange	770	-2460	-1,67	592,6	-379,0
Light weight concrete	30	Lower flange	800	-1326	-2,65	114,5	-181,2
		Web	483	-286	-2,26	439,6	-244,0
		Upper flange	331	-2284	-1,54	488,8	-131,7
	60	Lower flange	955	-622	-1,32	47,7	-81,1
		Web	761	-558	-1,67	426,5	-303,0
		Upper flange	607	-2261	-1,02	664,5	-410,0
	90	Lower flange	1019	-478	-0,91	32,7	-60,8
		Web	906	-654	-1,36	287,8	-230,3
		Upper flange	789	-1847	-0,99	469,5	-313,0
	120	Lower flange	1062	-399	-0,65	19,8	-43,7
		Web	989	-629	-1,07	186,1	-152,6
		Upper flange	903	-1561	-0,92	305,2	-197,2

(2) The view factor Φ of the upper flange and the rib geometry factor A/L_r may be established according to D.1.

(3) The temperature θ_s of the reinforcement bars in the rib (see Figure D.2) is given by:

$$\theta_s = c_0 + \left(c_1 \cdot \frac{u_3}{h_2} \right) + \left(c_2 \cdot z \right) + \left(c_3 \cdot \frac{A}{L_r} \right) + \left(c_4 \cdot \alpha \right) + \left(c_5 \cdot \frac{1}{\ell_3} \right) \qquad \text{(D.5)}$$

where:

θ_s	the temperature of additional reinforcement in the rib	[°C]
u_3	distance to lower flange	[mm]
z	indication of the position in the rib (see (4))	[mm$^{-0.5}$]
α	angle of the web	[degrees]

For factors c_i for both normal and lightweight concrete, refer to Table D.3. For intermediate values, linear interpolation is allowed.

Table D.3: Coefficients for the determination of the temperatures of the reinforcement bars in the rib.

Concrete	Fire resistance [min]	c_0 [°C]	c_1 [°C]	c_2 [°C]. mm$^{0.5}$	c_3 [°C].mm	c_4 [°C/°]	c_5 [°C].mm
Normal weight concrete	60	1191	-250	-240	-5,01	1,04	-925
	90	1342	-256	-235	-5,30	1,39	-1267
	120	1387	-238	-227	-4,79	1,68	-1326
Light weight concrete	30	809	-135	-243	-0,70	0,48	-315
	60	1336	-242	-292	-6,11	1,63	-900
	90	1381	-240	-269	-5,46	2,24	-918
	120	1397	-230	-253	-4,44	2,47	-906

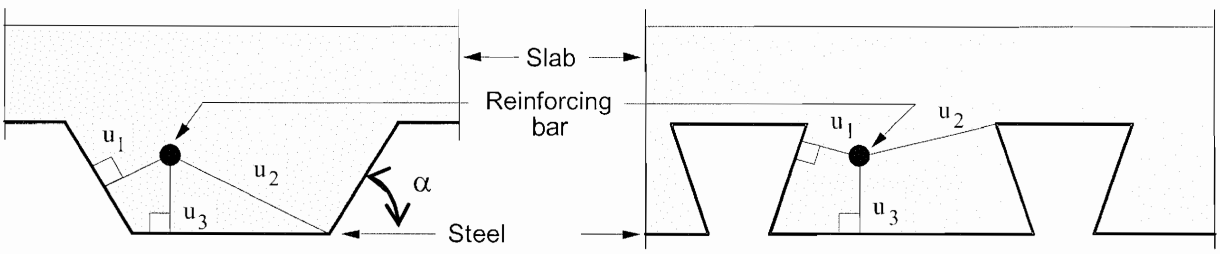


Figure D.2: Parameters for the position of the reinforcement bars

(4) The z-factor which indicates the position of the reinforcement bar is given by:

$$\frac{1}{z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}} \qquad \text{(D.6)}$$

(5) The distances u_1, u_2 and u_3 are expressed in mm and are defined as follows:

u_1, u_2: shortest distance of the centre of the reinforcement bar to any point of the webs of the steel sheet;

u_3: distance of the centre of the reinforcement bar to the lower flange of the steel sheet.

(6) Based on the temperatures given by (1) to (5), the ultimate stresses of the parts of the composite slab and the sagging moment resistance are calculated according to 4.3.1.

D.3 Calculation of the hogging moment resistance $M_{fi,Rd}^{-}$:

(1) As a conservative approximation, the contribution of the steel decking to the hogging moment capacity may be ignored.

(2) The hogging moment resistance of the slab is calculated by considering a reduced cross section. The parts of the cross section, with temperatures beyond a certain limiting temperature θ_{lim}, are neglected. The remaining cross section is considered as under room temperature conditions.

(3) The remaining cross section is established, on the basis of the isotherm for the limiting temperature (see Figures D.3). The isotherm for the limiting temperature, is schematised by means of 4 characteristic points, as follows:

point I: is situated at the central line of the rib, at a distance from the lower flange of the steel sheet and calculated as a function of the limiting temperature according to equation D.7 and D.9 of (4) and (5);

point IV: is situated at the central line between two ribs, at a distance from the upper flange of the steel sheet, calculated as a function of the limiting temperature according to equations D.7 and D.14 of (4) and (5);

point II: is situated on a line through point I, parallel to the lower flange of the steel sheet, at a distance from the web of the steel sheet, equal to that from the lower flange;

point III: is situated on a line through the upper flange of the steel sheet, at a distance from the web of the steel sheet, equal to the distance of point IV to the upper flange.

The isotherm is obtained by linear interpolation between the points I, II, III and IV.

Note: The limiting temperature is derived from equilibrium over the cross section and therefore has no relation with temperature penetration

A) Temperature distribution in a cross section

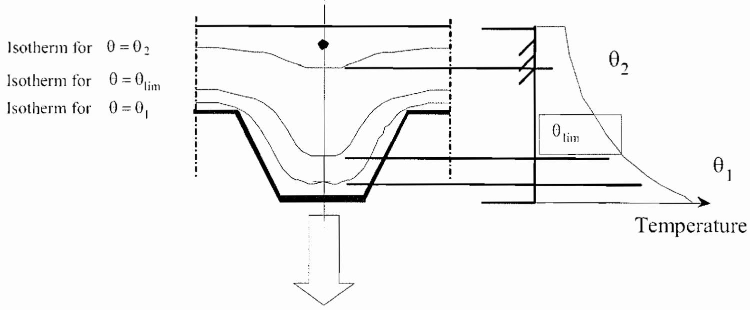


Figure D.3.a : Schematisation isotherm

B) Schematisation specific isotherm $\theta = \theta_{lim}$

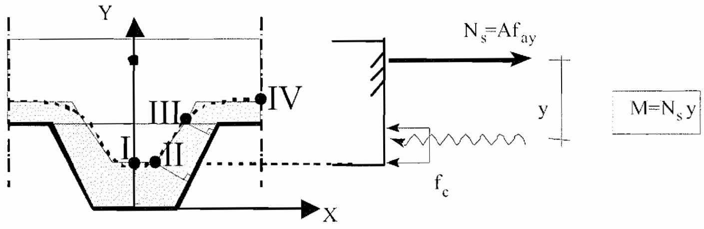


Figure D.3.b: Establishment of isotherms

(4) The limiting temperature, θ_{lim} is given by:

$$\theta_{lim} = d_0 + d_1 \cdot N_s + d_2 \cdot \frac{A}{L_r} + d_3 \cdot \Phi + d_4 \cdot \frac{1}{\ell_3} \tag{D.7}$$

where:

N_s is the normal force in the hogging reinforcement [N]

For factors d_i, for both normal and lightweight concrete, refer to Table D.4 For intermediate values, linear interpolation is allowed.

(5) The coordinates of the four points I to IV are given by:

$$X_I = 0 \tag{D.8}$$

$$Y_I = Y_{II} = \frac{1}{\left(\frac{1}{z} - \frac{4}{\sqrt{\ell_1 + \ell_3}}\right)^2} \tag{D.9}$$

$$X_{II} = \frac{1}{2}\ell_2 + \frac{Y_I}{\sin\alpha} \cdot (\cos\alpha - 1) \tag{D.10}$$

with: $\alpha = \arctan\left(\dfrac{2h_2}{\ell_1 - \ell_2}\right)$

$$X_{III} = \frac{1}{2}\ell_1 - \frac{b}{\sin\alpha} \tag{D.11}$$

with: $a = \left(\dfrac{1}{z} - \dfrac{1}{\sqrt{h_2}}\right)^2 \ell_1 \sin\alpha$

$$Y_{III} = h_2 \tag{D.12}$$

with:

$$b = \frac{1}{2}\ell_1 \sin\alpha\left(1 - \frac{\sqrt{a^2 - 4a + c}}{a}\right)$$

$$X_{IV} = \frac{1}{2}\ell_1 + \frac{1}{2}\ell_3 \tag{D.13}$$

with: $c = -8\left(1 + \sqrt{1 + a}\right);\ a \geq 8$

$$Y_{IV} = h_2 + b \tag{D.14}$$

with: $c = +8\left(1 + \sqrt{1 + a}\right);\ a < 8$

Table D.4 : Coefficients for the determination of the limiting temperature.

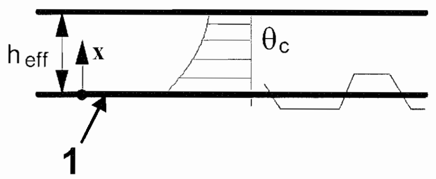
Concrete	Fire resistance [min]	d_0 [°C]	d_1 [°C] . N	d_2 [°C] . mm	d_3 [°C]	d_4 [°C] . mm
Normal weight concrete	60	867	$-1{,}9 \cdot 10^{-4}$	-8,75	-123	-1378
	90	1055	$-2{,}2 \cdot 10^{-4}$	-9,91	-154	-1990
	120	1144	$-2{,}2 \cdot 10^{-4}$	-9,71	-166	-2155
Light weight concrete	30	524	$-1{,}6 \cdot 10^{-4}$	-3,43	-80	-392
	60	1030	$-2{,}6 \cdot 10^{-4}$	-10,95	-181	-1834
	90	1159	$-2{,}5 \cdot 10^{-4}$	-10,88	-208	-2233
	120	1213	$-2{,}5 \cdot 10^{-4}$	-10,09	-214	-2320

(6) The parameter z given in (5) may be solved from the equation for the determination of the rebar temperature (i.e. equ. D.5), assuming u_3/h_2 = 0,75 and using $\theta_s = \theta_{lim}$.

(7) In the case of $Y_l > h_2$, the ribs of the slab may be neglected. Table D.5 may be used to obtain the location of the isotherm as a conservative approximation.

Table D.5: Temperature distribution in a solid slab of 100 mm thickness composed of normal weight concrete and not insulated.

Depth x mm	Temperature θ_c[°C] after a fire duration in min. of 30'	60'	90'	120'	180'	240'
5	535	705				
10	470	642	738			
15	415	581	681	754		
20	350	525	627	697		
25	300	469	571	642	738	
30	250	421	519	591	689	740
35	210	374	473	542	635	700
40	180	327	428	493	590	670
45	160	289	387	454	549	645
50	140	250	345	415	508	550
55	125	200	294	369	469	520
60	110	175	271	342	430	495
80	80	140	220	270	330	395
100	60	100	160	210	260	305



1 – Heated lower side of slab

(8) The hogging moment resistance is calculated by using the remaining cross section determined by (1) to (7) and by referring to 4.3.1

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

D.4 Effective thickness of a composite slab

(1) The effective h_{eff} is given by the formula:

$$h_{eff} = h_1 + 0{,}5\, h_2 \left(\frac{\ell_1 + \ell_2}{\ell_1 + \ell_3} \right) \qquad \text{for } h_2/h_1 \leq 1{,}5 \text{ and } h_1 > 40 \text{ mm} \qquad \text{(D.15a)}$$

$$h_{eff} = h_1 \left[1 + 0{,}75 \left(\frac{\ell_1 + \ell_2}{\ell_1 + \ell_3} \right) \right] \qquad \text{for } h_2/h_1 > 1{,}5 \text{ and } h_1 > 40 \text{ mm} \qquad \text{(D.15b)}$$

The cross sectional dimensions of the slab h_1, h_2, ℓ_1, ℓ_2 and ℓ_3 are given in Figures 4.1 and 4.2.

(2) If $\ell_3 > 2\,\ell_1$, the effective thickness may be taken equal to h_1 .

(3) The relation between the fire resistance with respect to the thermal insulation criterion and the minimum effective slab thickness h_{eff} is given in Table D.6 for common levels of fire resistance, where h_3 is the thickness of the screed layer if any on top of the concrete slab.

[AC1) Table D.6 - Minimum effective thickness as a function of the standard fire resistance

Standard Fire Resistance	Minimum effective thickness h_{eff} [mm]
I 30	60 - h_3
I 60	80 - h_3
I 90	100 - h_3
I 120	120 - h_3
I 180	150 - h_3
I 240	175 - h_3

(AC1]

D.5 Field of application

(1) The field of application for unprotected composite slabs is given in Table D.7 for both normal weight concrete (NC) and lightweight concrete (LC). For notations see Figures 4.1 and 4.2.

Table D.7: Field of application

for re-entrant steel sheet profiles	for trapezoidal steel sheet profiles
$77{,}0 \leq \ell_1 \leq 135{,}0$ mm	$80{,}0 \leq \ell_1 \leq 155{,}0$ mm
$110{,}0 \leq \ell_2 \leq 150{,}0$ mm	$32{,}0 \leq \ell_2 \leq 132{,}0$ mm
$38{,}5 \leq \ell_3 \leq 97{,}5$ mm	$40{,}0 \leq \ell_3 \leq 115{,}0$ mm
$50{,}0 \leq h_1 \leq 130{,}0$ mm	$50{,}0 \leq h_1 \leq 125{,}0$ mm
$30{,}0 \leq h_2 \leq 60{,}0$ mm	$50{,}0 \leq h_2 \leq 100{,}0$ mm

Annex E
[informative]

Model for the calculation of the sagging and hogging moment resistances of a steel beam connected to a concrete slab and exposed to fire beneath the concrete slab.

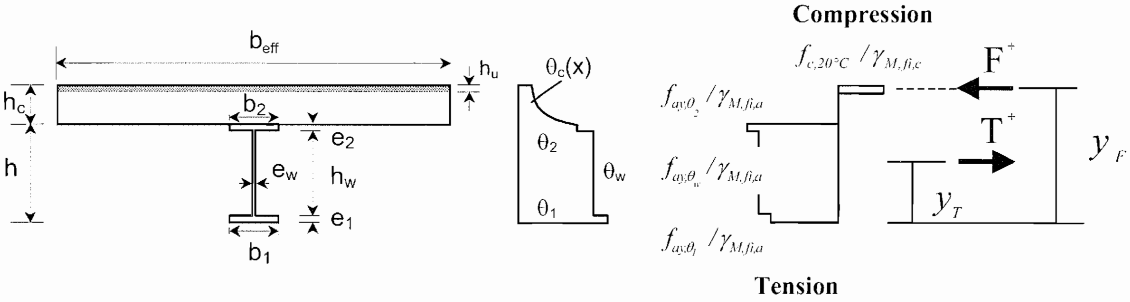


Figure E.1: Calculation of the sagging moment resistance

E.1 Calculation of the sagging moment resistance $M_{fi,Rd}^+$

(1) According to Figure E.1 the tensile force T^+ and its location y_T may be obtained from:

$$T^+ = \left[f_{ay,\theta 1}(b_1\, e_1) + f_{ay,\theta w}(h_w\, e_w) + f_{ay,\theta 2}(b_2\, e_2) \right] \,/\, \gamma_{M,fi,a} \tag{E.1}$$

$$y_T = \left[f_{ay,\theta 1}(b_1)(e_1^2/2) + f_{ay,\theta w}(h_w\, e_w)(e_1 + h_w/2) + f_{ay,\theta 2}(b_2\, e_2)(h - e_2/2) \right] / \left(T^+\, \gamma_{M,fi,a} \right) \tag{E.2}$$

with $f_{ay,\theta}$ the maximum stress level according to 3.2.1 at temperature θ defined following 4.3.4.2.2.

(2) In a simply supported beam, the value of the tensile force T^+ obtained from (1) is limited by:

$$T^+ \leq N\, P_{fi,Rd} \tag{E.3}$$

where:

N is the smaller number of shear connectors related to any critical length of the beam and $P_{fi,Rd}$ is the design shear resistance in the fire situation of a shear connector according to 4.3.4.2.5.

NOTE: The critical lengths are defined by the end supports and the cross-section of maximum bending moment.

(3) The thickness of the compressive zone h_u is determined from:

$$h_u = T^+ \,/\, \left(b_{eff}\, f_c / \gamma_{M,fi,c} \right) \tag{E.4}$$

where b_{eff} is the effective width according to 5.4.1.2 of EN 1994-1-1, and f_c the compressive strength of concrete at room temperature.

(4) Two situations may occur:

$(h_c - h_u) \geq h_{cr}$ with h_{cr} is the depth **x** according to Table D.5 corresponding to a concrete temperature below 250°C. In that situation the value of h_u according to equation (E.4) applies.

or $(h_c - h_u) < h_{cr}$; some layers of the compressive zone of concrete are at a temperature higher than 250°C. In this respect, a decrease of the compressive strength of concrete may be considered according to 3.2.2. The h_u value may be determined by iteration varying the index "n" and assuming on the basis of Table D.5 an average temperature for every slice of 10 mm thickness, such as:

$$T^+ = F = \left[(h_c - h_{cr})(b_{eff}) f_c + \sum_{i=2}^{n-1} (10 b_{eff}) f_{c,\theta i} + (h_{u,n}\ b_{eff}) f_{c,\theta n} \right] \Big/ \gamma_{M,fi,c} \tag{E.5}$$

where:

$$h_u = (h_c - h_{cr}) + 10\,(n-2) + h_{u,n} \qquad \text{[mm]}$$

n is the total number of concrete layers in compression, including the top concrete layer $(h_c - h_{cr})$ with a temperature below 250°C.

(5) The point of application of this compression force is obtained from

$$y_F \approx h + h_c - (h_u/2) \tag{E.6}$$

and the sagging moment resistance is

$$M_{fi,Rd^+} = T^+\,(y_F - y_T) \tag{E.7}$$

with T^+, the tensile force given by the value of (E.5) while taking account of (E.3).

(6) This calculation model may be used for a composite slab with a profiled steel sheet, provided in (3) and (4), h_c is replaced by h_{eff} as defined in (1) of D.4 and h_u is limited by h_l as defined in Figures 4.1 and 4.2.

(7) This calculation model established in connection to 4.3.4.2.4, may be used for the critical temperature model of 4.3.4.2.3 by assuming that $\theta_1 = \theta_w = \theta_2 = \theta_{cr}$.

(8) A similar approach may be used if the neutral axis is not inside the concrete slab but in the steel beam.

E.2 Calculation of the hogging moment resistance M_{fi,Rd^-} at an intermediate support (or at a restraining support)

(1) The effective width of the slab at an intermediate support (or at the restraining support) b^-_{eff} may be determined so that the plastic neutral axis does not lie in the concrete slab, i.e. the slab is assumed to be cracked over its whole thickness. This effective width may not be larger than that determined at normal temperature, according to 5.4.1.2 of EN 1994-1-1.

(2) The longitudinal tensile reinforcing bars may be assumed at the plastic yield $f_{sy,\theta s}$ where θ_s is the temperature in the slab, at the level where the reinforcing bars are located.

(3) The following clauses assume that the plastic neutral axis is located just at the interface between the slab and the steel section. A similar approach may be used if the plastic neutral axis is within the steel cross section, by changing the formulae accordingly.

(4) The hogging plastic moment resistance of the composite section may be determined by considering the stress diagram of Figure E.2, with temperatures $\theta_1, \theta_2, \theta_w$ calculated according to 4.3.4.2.2.

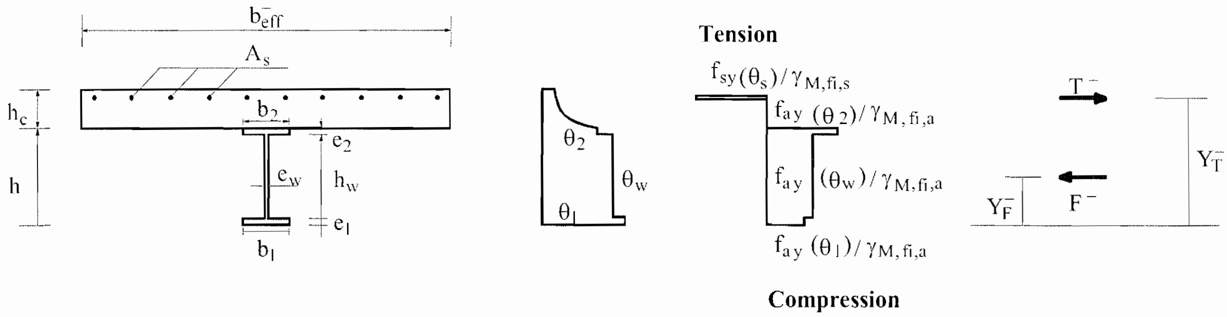


Figure E.2: Calculation of the hogging moment resistance

(5) The hogging moment resistance is given by : $M_{fi,Rd^-} = T^-(y^-_T - y^-_F)$

where :

T^- is the total tensile force of the reinforcing bars, equal to the compressive force F^- in the steel section.

AC1 (6) The value of the compressive force F^+ in the slab, at the critical cross section within the span, see (2) of E.1, may be such as :

$$F^+ \leq N \times P_{fi,Rd} - T^- \qquad \text{(E.8)}$$ AC1

where:

N is the number of shear connectors between the critical cross-section and the intermediate support (or the restraining support) and where $P_{fi,Rd}$ is the shear resistance of a shear connector in case of fire, as mentioned in clause 4.3.4.2.5.

(7) The previous clauses may be used for cross sections of class 1 or 2 defined in the fire situation; for sections of class 3 or 4 the following clauses (8) to (9) apply.

NOTE: Classification may be done according to 4.2.2 of EN1993-1-2.

(8) When the steel web or the lower steel flange of the composite section is of class 3 in the fire situation, its width may be reduced to an effective value adapted from EN 1993-1-5, where f_y and E are respectively replaced by $f_{ay,\theta}$ and $E_{a,\theta}$.

(9) When the steel web or the bottom steel flange of the composite section is of class 4 in the fire situation, its resistance may be neglected.

E.3 Local resistance at supports

(1) The local resistance of the steel section shall be checked against the reaction force at the support (or at the restraining support).

(2) The temperature of stiffener θ_r is calculated by considering its own section factor, A_r/V_r, according to 4.3.4.2.2.

(3) The local resistance of the steel section at the support (or at the restraining support) is taken equal to the lower value of the buckling or the crushing resistance.

(4) For the calculation of the buckling resistance a maximum width of the web of $15\varepsilon e_w$ on each side of the stiffener (see Figure E.3) may be added to the effective cross section of the stiffener. The relative slenderness $\overline{\lambda}_\theta$ used to calculate buckling resistance is given by :

$$\overline{\lambda}_0 = \overline{\lambda} \cdot \max\{(k_{y,\theta w} / k_{E,\theta w})^{0,5} ; (k_{y,\theta r} / k_{E,\theta r})^{0,5}\} \qquad \text{(E.9)}$$

where:

$k_{E,\theta}$ and $k_{y,\theta}$ are given in Table 3.2 ,

$\overline{\lambda}$ is the relative slenderness at room temperature for the stiffener associated with part of web as shown in Figure E.3 and

ε is calculated according to 4.2.2 of EN1993-1-2.

(5) For the calculation of the crushing resistance, the design crushing resistance, $R_{fi,y,Rd}$, of the web with the stiffeners is given by :

$$R_{fi,y,Rd} = \left[s_s + 5\left(e_f + r\right)\right] e_w \, f_{ay,\theta w} / \gamma_{M,fi,a} + A_r \, f_{ay,\theta r} / \gamma_{M,fi,a} \qquad \text{(E.10)}$$

where:

$f_{ay,\theta w}$ and $f_{ay,\theta r}$ are respectively the maximum stresses in steel at the temperature of web θ_w and of stiffener θ_r ;

r is equal to the root radius for a hot rolled section, or to $a\sqrt{2}$ with a the throat of fillet weld for a welded cross-section.

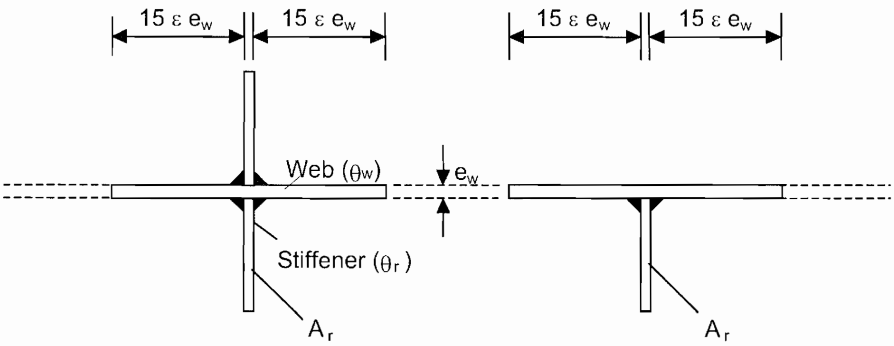


Figure E.3 : Stiffener on an intermediate support

E.4 Vertical shear resistance

(1) Clauses in 6.2.2 of EN 1994-1-1 may be used to check the vertical shear resistance of composite beams in fire situation by replacing E_a , f_{ay} and γ_a by $E_{a,\theta}$, $f_{ay,\theta}$ and $\gamma_{M,fi,a}$ respectively as defined in Table 3.2 and clause 2.3(1)P.

Annex F
[informative]

Model for the calculation of the sagging and hogging moment resistances of a partially encased steel beam connected to a concrete slab and exposed to fire beneath the concrete slab according to the standard temperature-time curve .

F.1 Reduced cross-section for sagging moment resistance M_{fi,Rd^+}

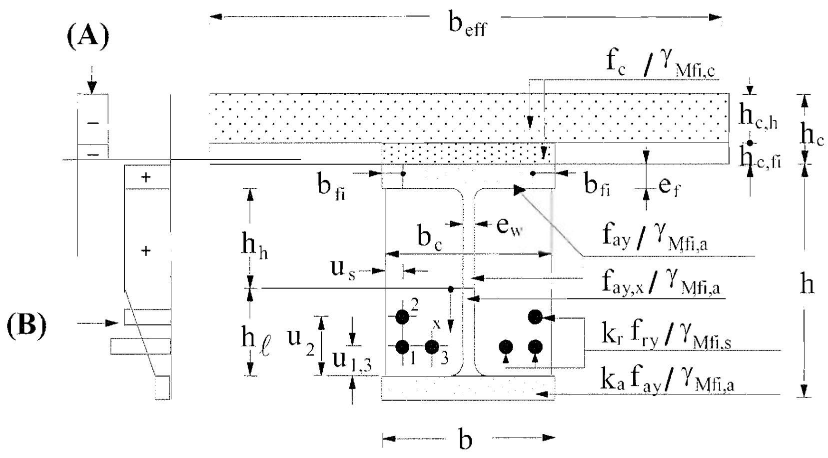


Figure F.1:Calculation scheme for the sagging moment resistance.

Note to Figure F.1: **(A)** Example of stress distribution in concrete;
(B) Example of stress distribution in steel

(1) The section of the concrete slab is reduced as shown in Figure F.1, but the design value of the compressive concrete strength $f_c / \gamma_{M,fi,c}$ is not varying in function of the fire classes. The values of the thickness reduction $h_{c,fi}$ of a flat concrete slab are given in Table F.1 for the different fire classes.

Table F.1: Thickness reduction $h_{c,fi}$ of the concrete slab.

Standard Fire Resistance	Slab Reduction $h_{c,fi}$ [mm]
R 30	10
R 60	20
R 90	30
R 120	40
R 180	55

(2) For other concrete slab systems the following rules apply:

- for trapezoidal steel sheets (see Figure 1.1) disposed transversally on the beam, the thickness reduction $h_{c,fi}$ of Table F.1 may be applied on the upper face of the steel deck (Figure F.2.a);

- for re-entrant profiles (see Figure 1.1) disposed transversally on the beam, the thickness reduction $h_{c,fi}$ of Table F.1 may be applied on the lower face of the steel deck. However, the value of $h_{c,fi}$ may not be smaller than the height of the deck profile (Figure F.2.b);
- when prefabricated concrete planks are used, the thickness reduction $h_{c,fi}$ of Table F.1 may be applied on the lower face of the concrete plank, but may not be smaller than the height of the joint, between precast elements, unable to transmit a compression stress (Figure F.2.c);
- for re-entrant profiles parallel to the beam, the thickness reduction $h_{c,fi}$ of Table F.1 applies on the lower face of the steel deck;
- for trapezoidal steel sheets parallel to the beam, the thickness reduction $h_{c,fi}$ of Table F.1 may be applied on the effective height of the slab h_{eff} (see Figure F.2.d), where the effective thickness of the slab h_{eff} is given in Figures 4.1 and in D.4 of Annex D.

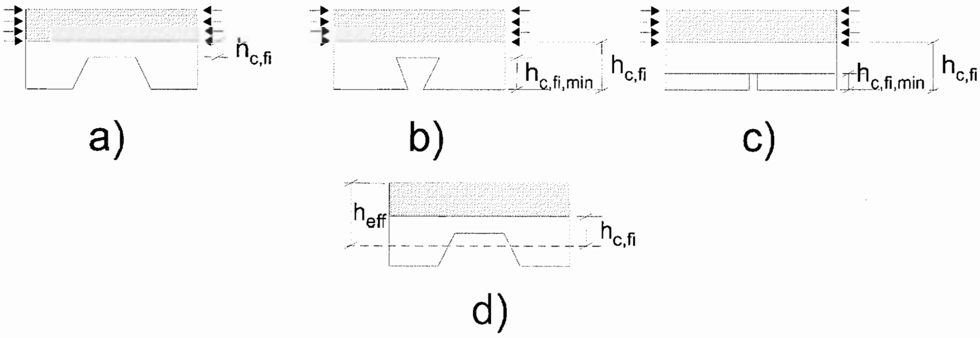


Figure F.2: Thickness reduction $h_{c,fi}$ for various types of concrete slabs

(3) The temperature θ_c of the concrete layer $h_{c,fi}$ situated directly on top of the upper flange, may be assumed to be 20°C.

(4) The effective width of the upper flange of the profile (b-2b_{fi}) varies as a function of the fire classes, but the design value of the yield point of the steel is taken equal to $f_{ay}/\gamma_{M,fi,a}$. The values of the flange width reduction b_{fi} are given in Table F.2 for the different fire classes.

Table F.2: Width reduction b_{fi} of the upper flange

Standard Fire Resistance	Width Reduction b_{fi} of the Upper Flange [mm]
R 30	$(e_f / 2) + (b - b_c) / 2$
R 60	$(e_f / 2) + 10 + (b - b_c) / 2$
R 90	$(e_f / 2) + 30 + (b - b_c) / 2$
R 120	$(e_f / 2) + 40 + (b - b_c) / 2$
R 180	$(e_f / 2) + 60 + (b - b_c) / 2$

(5) The web is divided into two parts, the top part h_h and the bottom part h_ℓ. The values of h_ℓ are given for the different fire classes by the formula $h_\ell = a_1 / b_c + a_2 e_w / (b_c h)$. Parameters a_1 and a_2 are given in Table F.3 for $h / b_c \leq 1$ or $h / b_c \geq 2$.

The bottom part h_ℓ is given directly in Table F.3 for $1 < h / b_c < 2$.

Table F.3: Bottom part of the web h_ℓ [mm] and $h_{\ell,min}$ [mm], with $h_{\ell,max}$ equal to $(h - 2e_f)$.

	Standard Fire Resistance	a_1 [mm²]	a_2 [mm²]	$h_{\ell,min}$ [mm]
	R 30	3 600	0	20
	R 60	9 500	20 000	30
$h / b_c \leq 1$	R 90	14 000	160 000	40
	R 120	23 000	180 000	45
	R 180	35 000	400 000	55
	R 30	3 600	0	20
	R 60	9 500	0	30
$h / b_c \geq 2$	R 90	14 000	75 000	40
	R 120	23 000	110 000	45
	R 180	35 000	250 000	55
	R 30	$h_\ell = 3\,600 / b_c$		20
	R 60	$h_\ell = 9\,500 / b_c + 20\,000\,(e_w / b_c h)\,(2 - h / b_c)$		30
$1 < h / b_c < 2$	R 90	$h_\ell = 14\,000 / b_c + 75\,000\,(e_w / b_c h)$ $+ 85\,000\,(e_w / b_c h)\,(2 - h / b_c)$		40
	R 120	$h_\ell = 23\,000 / b_c + 110\,000\,(e_w / b_c h)$ $+ 70\,000\,(e_w / b_c h)\,(2 - h / b_c)$		45
	R 180	$h_\ell = 35\,000 / b_c + 250\,000\,(e_w / b_c h)$ $+ 150\,000\,(e_w / b_c h)\,(2 - h / b_c)$		55

(6) The bottom part h_ℓ of the web may always be larger or equal than $h_{\ell,min}$ given in Table F.3.

(7) For the top part h_h of the web, the design value of the yield point of the steel is taken equal to $f_{ay} / \gamma_{M,fi,a}$. For the bottom part h_ℓ, the design value of the yield point depends on the distance x measured from the end of the top part of the web (see Figure F.1). The reduced yield point in h_ℓ may be obtained from:

$$f_{ay,x} = f_{ay}\left[1 - x\left(1 - k_a\right) / h_\ell\right] \tag{F.1}$$

where:

k_a is the reduction factor of the yield point of the lower flange given in (8). This leads to a trapezoidal form of the stress distribution in h_ℓ.

(8) The area of the lower flange of the steel profile is not modified. Its yield point is reduced by the factor k_a given in Table F.4. The reduction factor k_a is limited by the minimum and maximum values given in this table.

Table F.4: Reduction factor k_a of the yield point of the lower flange, with a_0 = (0,018 e_f + 0,7).

Standard Fire Resistance	Reduction Factor k_a	$k_{a,min}$	$k_{a,max}$
R 30	$[(1,12) - (84 / b_c) + (h / 22b_c)]a_0$	0,5	0,8
R 60	$[(0,21) - (26 / b_c) + (h / 24b_c)]a_0$	0,12	0,4
R 90	$[(0,12) - (17 / b_c) + (h / 38b_c)]a_0$	0,06	0,12
R 120	$[(0,1) - (15 / b_c) + (h / 40b_c)]a_0$	0,05	0,10
R 180	$[(0,03) - (3 / b_c) + (h / 50b_c)]a_0$	0,03	0,06

(9) The yield point of the reinforcing bars decreases with their temperature. Its reduction factor k_r is given in Table F.5 and depends on the fire class and on the position of the reinforcing bar. The reduction factor k_r is limited by the minimum and maximum values given in this table.

Table F.5: Reduction factor k_r of the yield point of a reinforcing bar with

$k_r = (ua_3 + a_4)a_5 / \sqrt{(A_m/V)}$				$k_{r,min}$	$k_{r,max}$
Standard Fire Resistance	a_3	a_4	a_5		
R 30	0,062	0,16	0,126	0,1	1
R 60	0,034	- 0,04	0,101		
R 90	0,026	- 0,154	0,090		
R 120	0,026	- 0,284	0,082		
R 180	0,024	- 0,562	0,076		

where:

A_m = $2h + b_c$ [mm]

V = $h\, b_c$ [mm²]

$$u = 1 / [(1/u_i) + (1/u_{si}) + 1/(b_c - e_w - u_{si})] \quad \text{(F.2)}$$

where:

u_i is the axis distance [mm] from the reinforcing bar to the inner side of the flange and

u_{si} is the axis distance [mm] from the reinforcing bar to the outside border of the concrete (see Figure F.1).

(10) The concrete cover of reinforcing bars should comply with 5.1.

(11) The shear resistance of the steel web may be verified using the distribution of the design values of yield strength according to (7). If $V_{fi,Sd} \geq 0,5\, V_{fi,pl,Rd}$ the resistance of the reinforced concrete may be considered.

F.2 Reduced cross-section for hogging moment resistance M_{fi,Rd^-}

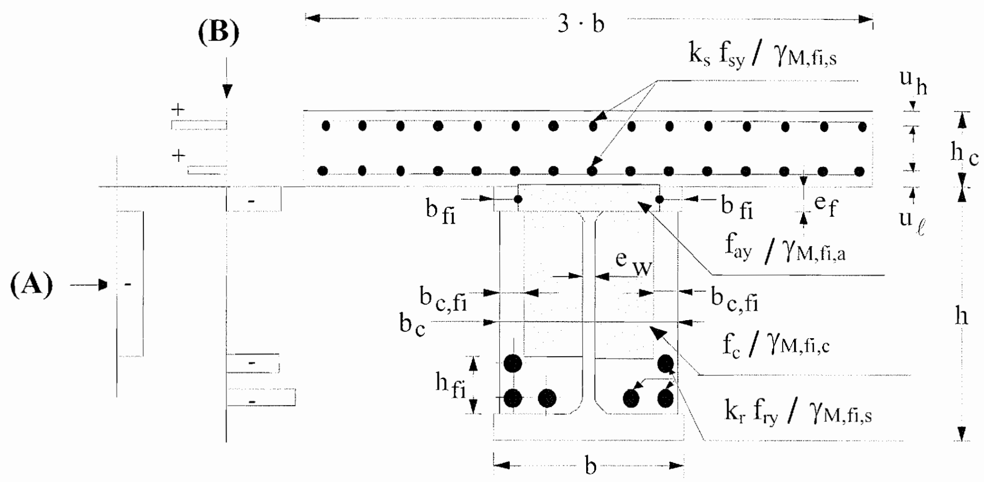


Figure F.3: Calculation scheme for the hogging moment resistance.

Note to Figure F.3: **(A)** Example of stress distribution in concrete;
(B) Example of stress distribution in steel

(1) The yield point of the reinforcing bars in the slab is multiplied by a reduction factor k_s given in Table F.6 and depends on the fire class and on the position of the reinforcing bars. The reduction factor k_s is limited by the minimum and maximum values given in this table.

Table F.6: Reduction factor k_s of the yield point of the reinforcing bars in the concrete slab with u, distance [mm] from the centre of the reinforcement to the lower slab edge, equal to u_ℓ or (h_c - u_h) (see Figure F.3).

Standard Fire Resistance	Reduction Factor k_s	$k_{s,min}$	$k_{s,max}$
R 30	1	0	1
R 60	(0,022 u) + 0,34		
R 90	(0,0275 u) - 0,1		
R 120	(0,022 u) - 0,2		
R 180	(0,018 u) - 0,26		

(2) For the upper flange of the profile, (4) of F.1 applies.

(3) The cross-section of the concrete between the flanges is reduced as shown in Figure F.3 but the design value of the compressive concrete strength $f_c/\gamma_{M,fi,c}$ does not vary as a function of the fire classes. The values of the width reduction $b_{c,fi}$ and of the height reduction h_{fi} of the encased concrete are given in Table F.7. The width and height reductions are limited by the minimum values given in this table.

Table F.7: Reduction of the cross-section of the concrete encased between the flanges.

Standard Fire Resistance	h_{fi} [mm]	$h_{fi,min}$ [mm]	$b_{c,fi}$ [mm]	$b_{c,fi,min}$ [mm]
R 30	25	25	25	25
R 60	$165 - (0{,}4b_c) - 8\ (h / b_c)$	30	$60 - (0{,}15b_c)$	30
R 90	$220 - (0{,}5b_c) - 8\ (h / b_c)$	45	$70 - (0{,}1b_c)$	35
R 120	$290 - (0{,}6b_c) - 10\ (h / b_c)$	55	$75 - (0{,}1b_c)$	45
R 180	$360 - (0{,}7b_c) - 10\ (h / b_c)$	65	$85 - (0{,}1b_c)$	55

(4) For the reinforcing bars situated in the concrete of the partially encased profile, (9) of F.1 applies.

(5) The concrete cover of reinforcing bars should comply with 5.1.

(6) In the areas with hogging bending moments, the shear force is assumed to be transmitted by the steel web, which is neglected when calculating the hogging bending moment resistance.

(7) The shear resistance of the steel web may be verified using the distribution of the design values of yield strength according to (7) of F.1.

F.3 Field of application

(1) The height h of the profile, b_c and the area h b_c should be at least equal to the minimum values given in Table F.8.

NOTE: The symbol b_c is the minimum value of either the width b of the lower flange or the width of the concrete part between the flanges, web thickness e_w included (see Figure F.1).

Table F.8: Minimum cross-section dimensions

Standard Fire Resistance	Minimum Profile Height h and Minimum Width b_c [mm]	Minimum Area h b_c [mm²]
R30	120	17500
R60	150	24000
R90	170	35000
R120	200	50000
R180	250	80000

(2) The flange thickness e_f should be smaller than the height h of the profile divided by 8.

Annex G
[informative]

Balanced summation model for the calculation of the fire resistance of composite columns with partially encased steel sections, for bending around the weak axis, exposed to fire all around the column according to the standard temperature-time curve .

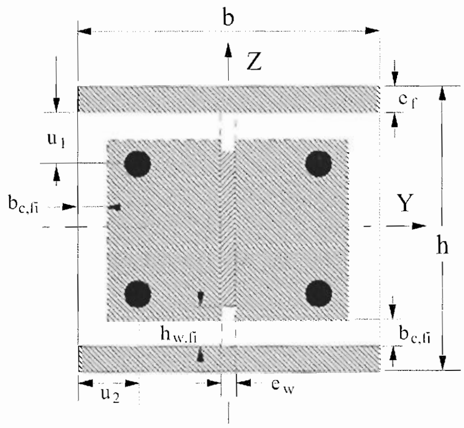


Figure G.1: Reduced cross-section for structural fire design

G.1 Introduction

(1) This calculation model is based on the principles and rules given in 4.3.5.1, but has been developed only for bending around the axis Z such as:

$$N_{fi,Rd,z} = \chi_z \, N_{fi,pl,Rd} \tag{G.1}$$

(2) For the calculation of the design value of the plastic resistance to axial compression $N_{fi,pl,Rd}$ and of the effective flexural stiffness $(EI)_{fi,eff,z}$ in the fire situation, the cross-section is divided into four components:

- the flanges of the steel profile;
- the web of the steel profile;
- the concrete contained by the steel profile and
- the reinforcing bars.

(3) Each component may be evaluated on the basis of a reduced characteristic strength, a reduced modulus of elasticity and a reduced cross-section in function of the standard fire resistance R30, R60, R90 or R120.

(4) The design value of the plastic resistance to axial compression and the effective flexural stiffness of the cross-section may be obtained, according to (4) and (5) of 4.3.5.1, by a balanced summation of the corresponding values of the four components.

(5) Strength and deformation properties of steel and concrete at elevated temperatures complies with the corresponding principles and rules of 3.1 and 3.2.

G.2 Flanges of the steel profile

(1) The average flange temperature may be determined from:

$$\theta_{f,t} = \theta_{o,t} + k_t \left(A_m / V\right) \tag{G.2}$$

where:

t is the duration in minutes of the fire exposure

A_m/V is the section factor in m^{-1}, with A_m = 2 (h + b) in [m] and V = h b in [m²]

$\theta_{o,t}$ is a temperature in °C given in Table G.1

k_t is an empirical coefficient given in Table G.1.

Table G.1: Parameters for the flange temperature

Standard Fire Resistance	$\theta_{o,t}$ [°C]	k_t [m°C]
R30	550	9,65
R60	680	9,55
R90	805	6,15
R120	900	4,65

(2) For the temperature $\theta = \theta_{f,t}$ the corresponding maximum stress level and the modulus of elasticity are determined from:

$$f_{ay,f,t} = f_{ay,f}\ k_{y,\theta} \quad \text{and} \tag{G.3}$$

$$E_{a,f,t} = E_{a,f}\ k_{E,\theta} \tag{G.4}$$

with $k_{y,\theta}$ and $k_{E,\theta}$ following Table 3.2 of 3.2.1

(3) The design value of the plastic resistance to axial compression and the flexural stiffness of the two flanges of the steel profile in the fire situation are determined from:

$$N_{fi,pl,Rd,f} = 2\left(b\, e_f\, f_{ay,f,t}\right) / \gamma_{M,fi,a} \quad \text{and} \tag{G.5}$$

$$(EI)_{fi,f,z} = E_{a,f,t}\left(e_f\, b^3\right) / 6 \tag{G.6}$$

G.3 Web of the steel profile

(1) The part of the web with the height $h_{w,fi}$ and starting at the inner edge of the flange may be neglected (see Figure G.1). This part is determined from:

$$h_{w,fi} = 0{,}5\left(h - 2e_f\right)\left(1 - \sqrt{1 - 0{,}16\left(H_t / h\right)}\right) \tag{G.7}$$

where H_t is given in Table G.2.

Table G.2: Parameter for height reduction of the web

Standard Fire Resistance	H_t [mm]
R 30	350
R 60	770
R 90	1100
R 120	1250

(2) The maximum stress level is obtained from:

$$f_{ay,w,t} = f_{ay,w}\sqrt{1-(0,16H_t/h)} \tag{G.8}$$

(3) The design value of the plastic resistance to axial compression and the flexural stiffness of the web of the steel profile in the fire situation are determined from:

$$N_{fi,pl,Rd,w} = \left[e_w\left(h-2e_f-2h_{w,fi}\right)f_{ay,w,t}\right]/\gamma_{M,fi,a} \tag{G.9}$$

$$(EI)_{fi,w,z} = \left[E_{a,w}\left(h-2e_f-2h_{w,fi}\right)e_w^3\right]/12 \tag{G.10}$$

G.4 Concrete

(1) An exterior layer of concrete with a thickness $b_{c,fi}$ may be neglected in the calculation (see Figure G.1). The thickness $b_{c,fi}$ is given in Table G.3, with A_m/V, the section factor in m^{-1} of the entire composite cross-section.

Table G.3: Thickness reduction of the concrete area

Standard Fire Resistance	$b_{c,fi}$ [mm]
R 30	4,0
R 60	15,0
R 90	0,5 (A_m/V) + 22,5
R 120	2,0 (A_m/V) + 24,0

(2) The average temperature in concrete $\theta_{c,t}$ is given in Table G.4 in function of the section factor A_m/V of the entire composite cross-section and for the standard fire resistance classes.

Table G.4: Average concrete temperarure

R30		R60		R90		R120	
A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]	A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]	A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]	A_m/V [m^{-1}]	$\theta_{c,t}$ [°C]
4	136	4	214	4	256	4	265
23	300	9	300	6	300	5	300
46	400	21	400	13	400	9	400
-	-	50	600	33	600	23	600
-	-	-	-	54	800	38	800
-	-	-	-	-	-	41	900
-	-	-	-	-	-	43	1000

(3) For the temperature $\theta = \theta_{c,t}$ the secant modulus of concrete is obtained from:

$$E_{c,sec,\theta} = f_{c,\theta} / \varepsilon_{cu,\theta} = f_c \, k_{c,\theta} / \varepsilon_{cu,\theta}$$ with $k_{c,\theta}$ and $\varepsilon_{cu,\theta}$ following Table 3.3 of 3.2.2 (G.11)

(4) The design value of the plastic resistance to axial compression and the flexural stiffness of the concrete in the fire situation are determined from:

$$N_{fi,pl,Rd,c} = 0{,}86 \left\{ \left(\left(h - 2e_f - 2b_{c,fi}\right)\left(b - e_w - 2b_{c,fi}\right) \right) - A_s \right\} \; f_{c,\theta} / \gamma_{M,fi,c} \qquad \text{(G.12)}$$

where A_s is the cross-section of the reinforcing bars, and 0,86 is a calibration factor.

$$(EI)_{fi,c,z} = E_{c,sec,\theta} \left[\left\{ \left(h - 2e_f - 2b_{c,fi}\right)\left(\left(b - 2b_{c,fi}\right)^3 - e_w^3\right) / 12 \right\} - I_{s,z} \right] \qquad \text{(G.13)}$$

where $I_{s,z}$ is the second moment of area of the reinforcing bars related to the central axis Z of the composite cross-section.

G.5 Reinforcing bars

(1) The reduction factor $k_{y,t}$ of the yield point and the reduction factor $k_{E,t}$ of the modulus of elasticity of the reinforcing bars, are defined in function of the standard fire resistance and the geometrical average u of the axis distances of the reinforcement to the outer borders of the concrete (see Tables G.5 and G.6).

Table G.5: Reduction factor $k_{y,t}$ for the yield point f_{sy} of the reinforcing bars

u[mm] Standard Fire Resistance	40	45	50	55	60
R30	1	1	1	1	1
R60	0,789	0,883	0,976	1	1
R90	0,314	0,434	0,572	0,696	0,822
R120	0,170	0,223	0,288	0,367	0,436

Table G.6: Reduction factor $k_{E,t}$ for the modulus of elasticity E_s of the reinforcing bars

u[mm] Standard Fire Resistance	40	45	50	55	60
R30	0,830	0,865	0,888	0,914	0,935
R60	0,604	0,647	0,689	0,729	0,763
R90	0,193	0,283	0,406	0,522	0,619
R120	0,110	0,128	0,173	0,233	0,285

(2) The geometrical average u of the axis distances u_1 and u_2 is obtained from:

$$u = \sqrt{u_1 \cdot u_2} \qquad \text{(G.14)}$$

where:

u_1 is the axis distance from the outer reinforcing bar to the inner flange edge [mm]

u_2 is the axis distance from the outer reinforcing bar to the concrete surface [mm]

Note: If $(u_1 - u_2) > 10$ mm, then $u = \sqrt{u_2 (u_2 + 10)}$,

or $(u_2 - u_1) > 10$ mm, then $u = \sqrt{u_1 (u_1 + 10)}$.

(3) The design value of the plastic resistance to axial compression and the flexural stiffness of the reinforcing bars in the fire situation are obtained from:

$$N_{fi,pl,Rd,s} = A_s \, k_{y,t} \, f_{sy} / \gamma_{M,fi,s} \tag{G.15}$$

$$(EI)_{fi,s,z} = k_{E,t} \, E_s \, I_{s,z} \tag{G.16}$$

G.6 Calculation of the axial buckling load at elevated temperatures

(1) According to (4) of G.1, the design value of the plastic resistance to axial compression and the effective flexural stiffness of the cross-section in the fire situation are determined from:

$$N_{fi,pl,Rd} = N_{fi,pl,Rd,f} + N_{fi,pl,Rd,w} + N_{fi,pl,Rd,c} + N_{fi,pl,Rd,s} \tag{G.17}$$

$$(EI)_{fi,eff,z} = \varphi_{f,\theta} \, (EI)_{fi,f,z} + \varphi_{w,\theta} \, (EI)_{fi,w,z} + \varphi_{c,\theta} \, (EI)_{fi,c,z} + \varphi_{s,\theta} \, (EI)_{fi,s,z} \tag{G.18}$$

where $\varphi_{i,\theta}$ is a reduction coefficient depending on the effect of thermal stresses. The values of $\varphi_{i,\theta}$ are given in Table G.7.

Table G.7: Reduction coefficients for bending stiffness

Standard Fire Resistance	$\varphi_{f,\theta}$	$\varphi_{w,\theta}$	$\varphi_{c,\theta}$	$\varphi_{s,\theta}$
R30	1,0	1,0	0,8	1,0
R60	0,9	1,0	0,8	0,9
R90	0,8	1,0	0,8	0,8
R120	1,0	1,0	0,8	1,0

(2) The Euler buckling load or elastic critical load follows by:

$$N_{fi,cr,z} = \pi^2 \, (EI)_{fi,eff,z} \, / \, \ell_\theta^2 \tag{G.19}$$

where:

ℓ_θ is the buckling length of the column in the fire situation.

(3) The non-dimensional slenderness ratio is obtained from:

$$\bar{\lambda}_\theta = \sqrt{N_{fi,pl,R} / N_{fi,cr,z}} \tag{G.20}$$

where:

$N_{fi,pl,R}$ is the value of $N_{fi,pl,Rd}$ according to (1) when the factors $\gamma_{M,fi,a}$, $\gamma_{M,fi,c}$ and $\gamma_{M,fi,s}$ are taken as 1,0.

(4) Using $\bar{\lambda}_\theta$ and the buckling curve c of EN 1993-1-1, the reduction coefficient χ_z may be calculated and the design axial buckling load in the fire situation is obtained from:

$$N_{fi,Rd,z} = \chi_z \, N_{fi,pl,Rd} \tag{G.21}$$

(5) The design values of the resistance of members in axial compression or the design axial buckling loads $N_{fi,Rd,z}$ are shown in Figures G.2 and G.3 as a function of the buckling length ℓ_θ for the profile series HEA and the material grades S355 of the steel profile, C40/50 of the concrete, S500 of the reinforcing bars and for the standard fire resistance classes R60, R90 and R120.

These design graphs are based on the partial material safety factors $\gamma_{M,fi,a} = \gamma_{M,fi,s} = \gamma_{M,fi,c} = 1{,}0$.

G.7 Eccentricity of loading

(1) For a column submitted to a load with an eccentricity δ, the design buckling load $N_{fi,Rd,\delta}$ may be obtained from:

$$N_{fi,Rd,\delta} = N_{fi,Rd}\left(N_{Rd,\delta}/N_{Rd}\right) \tag{G.22}$$

where:

N_{Rd} and $N_{Rd,\delta}$ represent the axial buckling load and the buckling load in case of an eccentric load calculated according to EN 1994-1-1, for normal temperature design.

(2) The application point of the eccentric load remains inside the composite cross-section of the column.

G.8 Field of application

(1) This calculation model may only be applied in the following conditions:

		buckling length ℓ_θ	≤	13,5b
230 mm	≤	height of cross section h	≤	1100 mm
230 mm	≤	width of cross section b	≤	500 mm
1 %	≤	percentage of reinforcing steel	≤	6 %
		standard fire resistance	≤	120 min

(2) In addition to (1), the minimum cross-section size b and h should be limited to 300 mm for the fire classes R90 and R120.

(3) For this calculation model the maximum buckling length ℓ_θ should be limited to 10b in the following situations:

- for R60, if 230 mm ≤ b < 300 mm or if h/b > 3 and
- for R90 and R120, if h/b > 3.

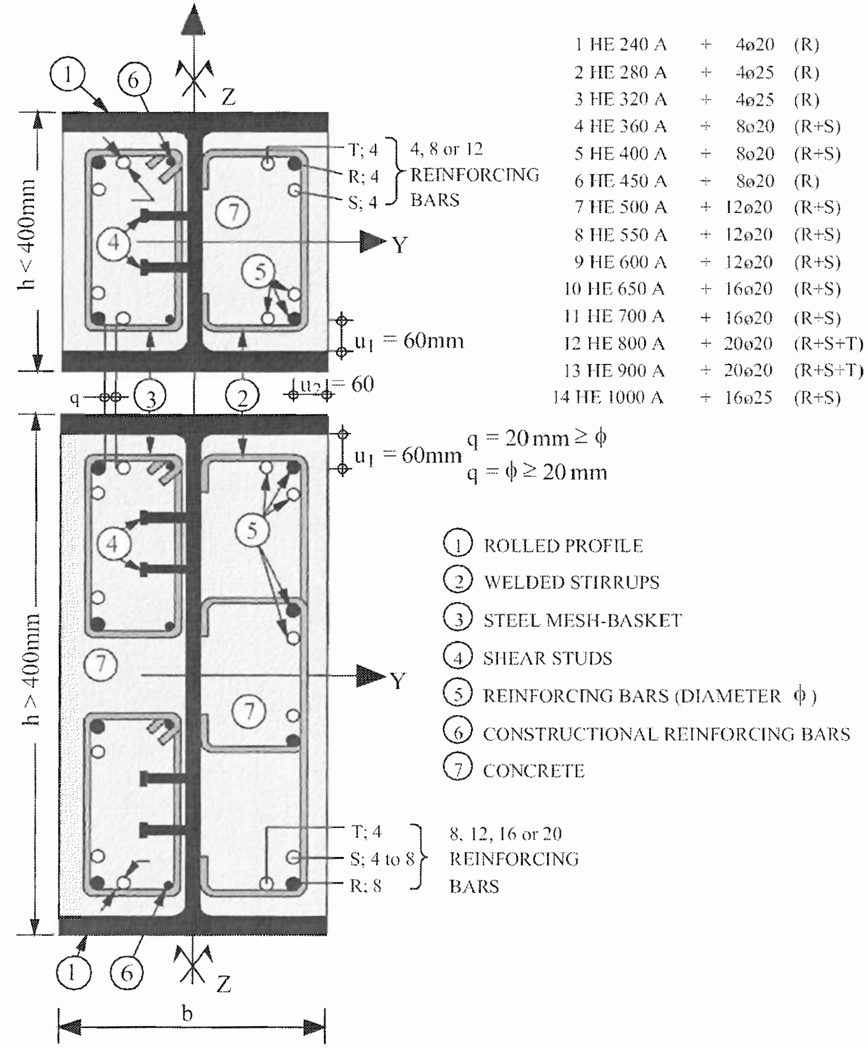


Figure G.2: Parameters for buckling resistance of partially encased steel sections

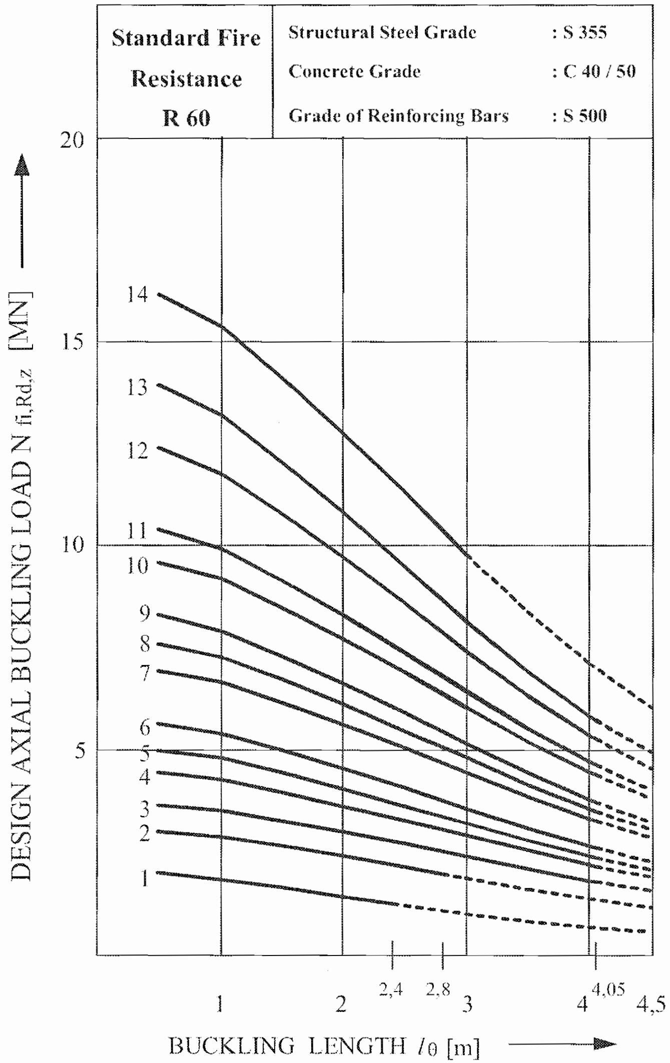


Figure G.3.a: Buckling loads of partially encased steel sections for R60

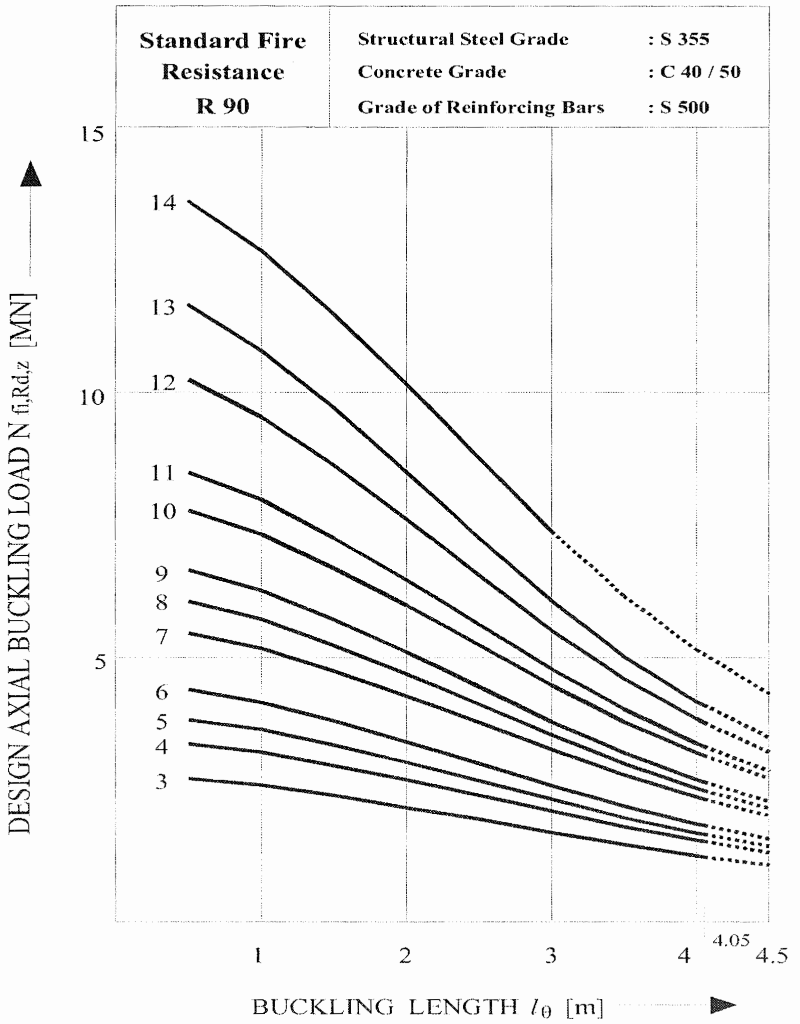


Figure G.3.b: Buckling loads of partially encased steel sections for R90

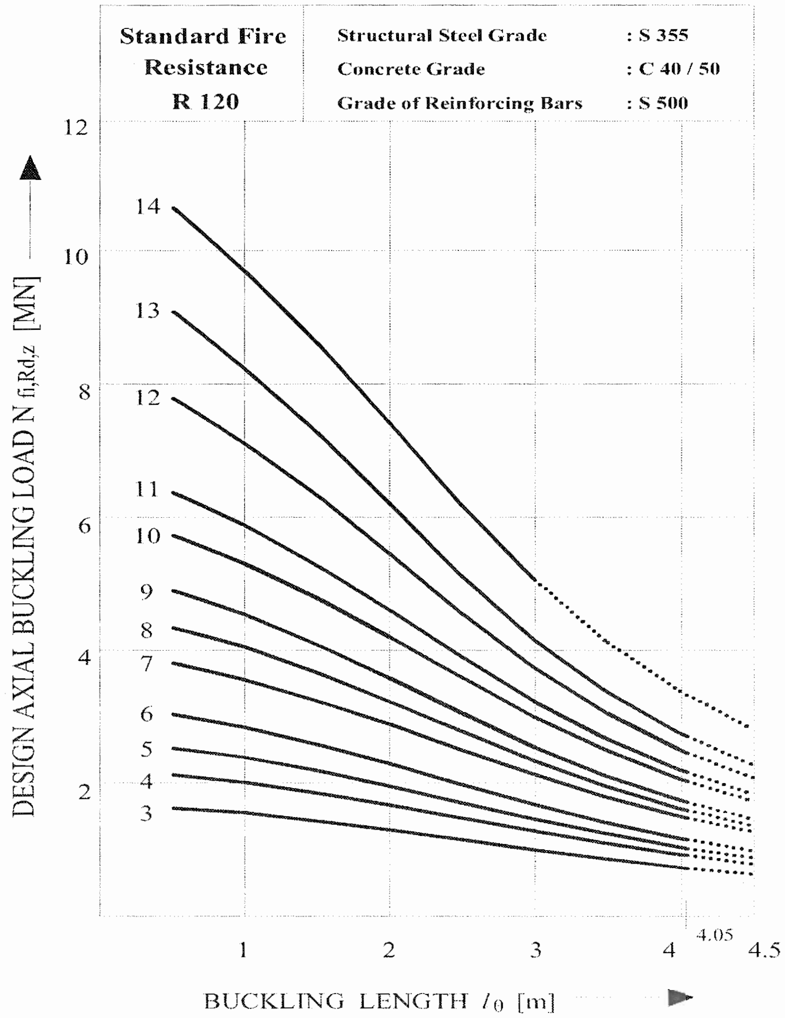


Figure G.3.c: Buckling loads of partially encased steel sections for R120

Annex H
[informative]

Simple calculation model for concrete filled hollow sections exposed to fire all around the column according to the standard temperature-time curve.

H.1 Introduction

(1) The calculation model to determine the design value of the resistance of a concrete filled hollow section column in axial compression and in the fire situation, is divided in two independent steps:

- calculation of the field of temperature in the composite cross-section after a given duration of fire exposure and
- calculation of the design axial buckling load $N_{fi,Rd}$ for the field of temperature previously obtained.

H.2 Temperature distribution

(1) The temperature distribution shall be calculated in accordance with 4.4.2

(2) In calculating the temperature distribution, the thermal resistance between the steel wall and the concrete may be neglected.

H.3 Design axial buckling load at elevated temperature

(1) For concrete filled hollow sections, the design axial buckling load $N_{fi,Rd}$ may be obtained from:

$$N_{fi,Rd} = N_{fi,cr} = N_{fi,pl,Rd} \tag{H.1}$$

where:

$$N_{fi,cr} = \pi^2 \left[E_{a,\theta,\sigma}\, I_a + E_{c,\theta,\sigma}\, I_c + E_{s,\theta,\sigma}\, I_s\right] / \ell_\theta^2 \quad \text{and} \tag{H.2}$$

$$N_{fi,pl,Rd} = A_a\, \sigma_{a,\theta} / \gamma_{M,fi,a} + A_c\, \sigma_{c,\theta} / \gamma_{M,fi,c} + A_s\, \sigma_{s,\theta} / \gamma_{M,fi,s} \quad \text{and where} \tag{H.3}$$

$N_{fi,cr}$ is the elastic critical or Euler buckling load,

$N_{fi,pl,Rd}$ is the design value of the plastic resistance to axial compression of the total cross-section,

ℓ_θ is the buckling length in the fire situation,

$E_{i,\theta,\sigma}$ is the tangent modulus of the stress-strain relationship for the material *i* at temperature θ and for a stress $\sigma_{i,\theta}$, (see Table 3.1 and Figure 3.2)

I_i is the second moment of area of the material *i*, related to the central axis y or z of the composite cross-section,

A_i is the cross-section area of material *i*,

$\sigma_{i,\theta}$ is the stress in material *i*, at the temperature θ.

(2) $E_{i,\theta,\sigma} \cdot I_i$ and $A_i \cdot \sigma_{i,\theta}$ are calculated as a summation of all elementary elements dy dz having the temperature θ after a fire duration t.

(3) The values of $E_{i,\theta,\sigma}$ and $\sigma_{i,\theta}$ to be used comply with:

$$\varepsilon_a = \varepsilon_c = \varepsilon_s = \varepsilon \tag{H.4}$$

where:

ε is the axial strain of the column and

ε_i is the axial strain of the material *i* of the cross-section.

(4) The design axial buckling loads $N_{fi,Rd}$ may be given in design graphs, like those of Figures H.3 and H.4, in function of the relevant physical parameters.

NOTE: The normal procedure is to increase the strain in steps. As the strain increases, $E_{i,\theta,\sigma}$ and $N_{fi,cr}$ decrease and $\sigma_{i,\theta}$ and $N_{fi,pl,Rd}$ increase. The level of strain is found where $N_{fi,cr}$ and $N_{fi,pl,Rd}$ are equal and the condition in (1) is satisfied.

H.4 Eccentricity of loading

(1) The following rules are applicable provided that, in the fire situation, the ratio between bending moment and axial force, $M/N = \delta$, does not exceed 0,5 times the size b or d of the cross-section.

(2) For a load eccentricity δ, the equivalent axial load N_{equ} to be used in connection with the axial load design graphs in the fire situation may be obtained from:

$$N_{equ} = N_{fi,Sd} / \left(\varphi_s \cdot \varphi_\delta\right) \tag{H.5}$$

where:

φ_s is given by Figure H.1 and φ_δ by Figure H.2.

b is the size of a square section,

d is the diameter of a circular section,

δ is the eccentricity of the load.

H.5 Field of application

(1) This calculation model may only be applied for square or circular sections in the following conditions:

	buckling length ℓ_θ	$\leq$ 4,5 m
140 mm $\leq$	depth b or diameter d of cross-section	$\leq$ 400 mm
C20/25 $\leq$	concrete grades	$\leq$ C40/50
0 % $\leq$	percentage of reinforcing steel	$\leq$ 5 %
	Standard fire resistance	$\leq$ 120 min.

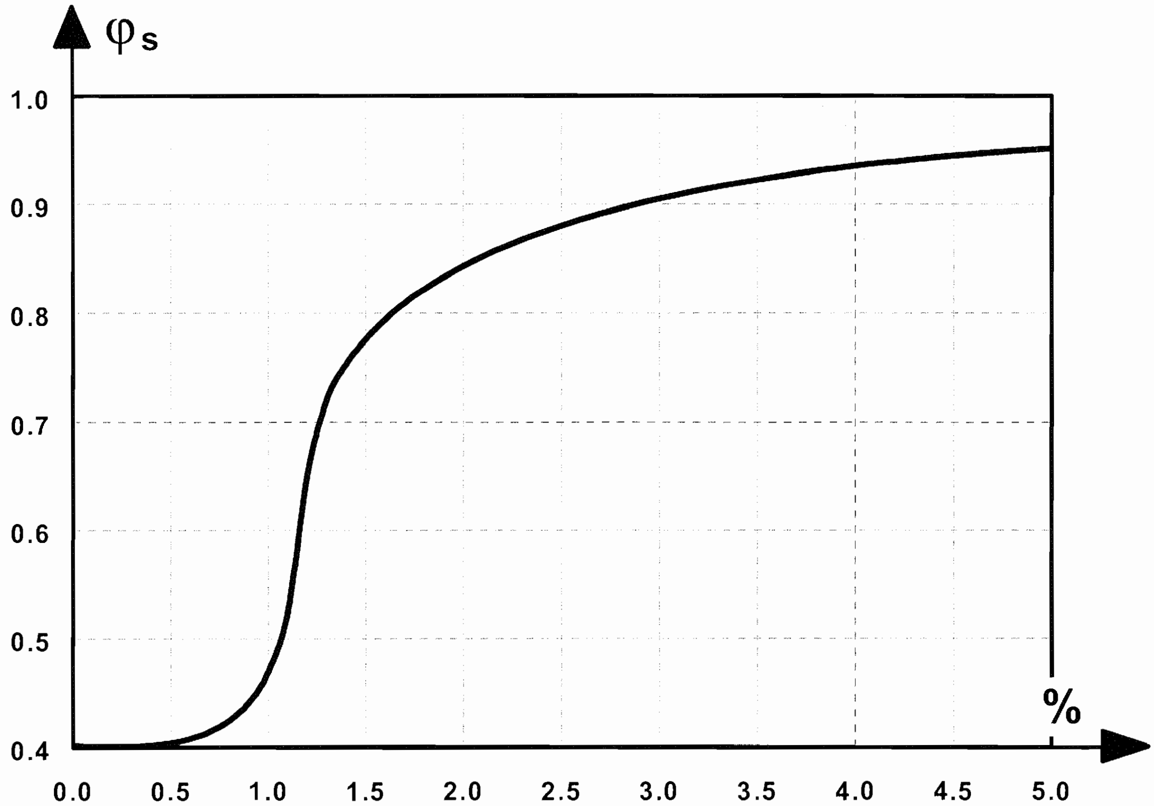


Figure H.1: Correction coefficient φ_s as a function of the percentage of reinforcement

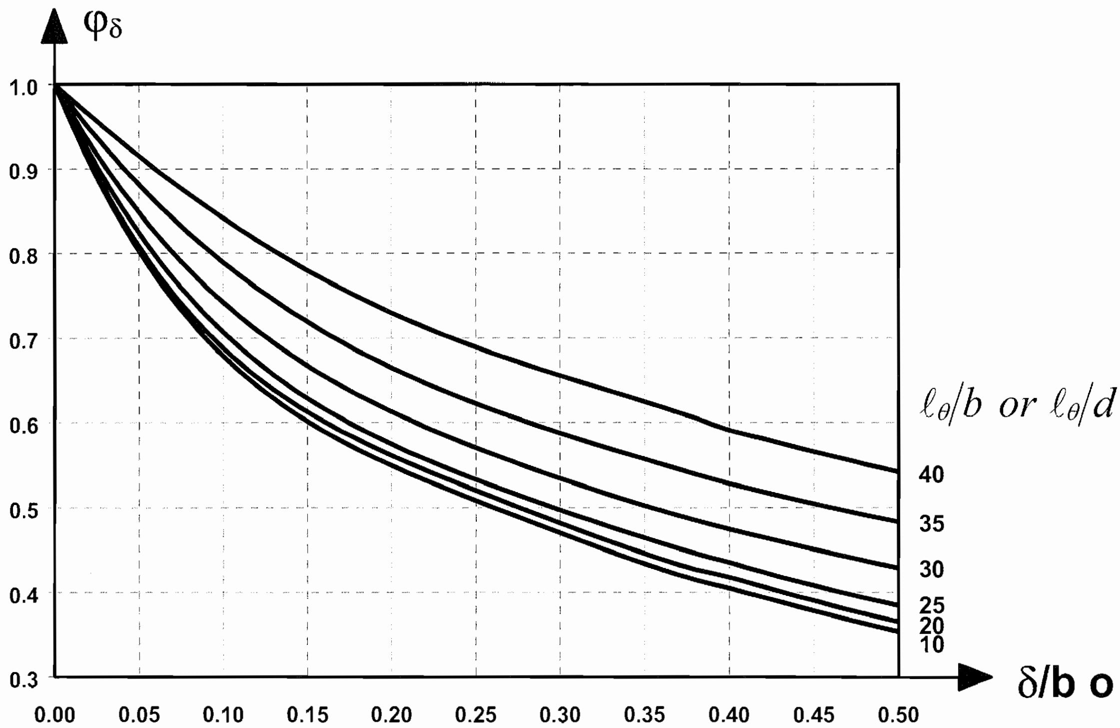
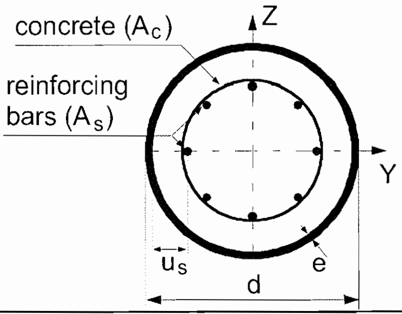
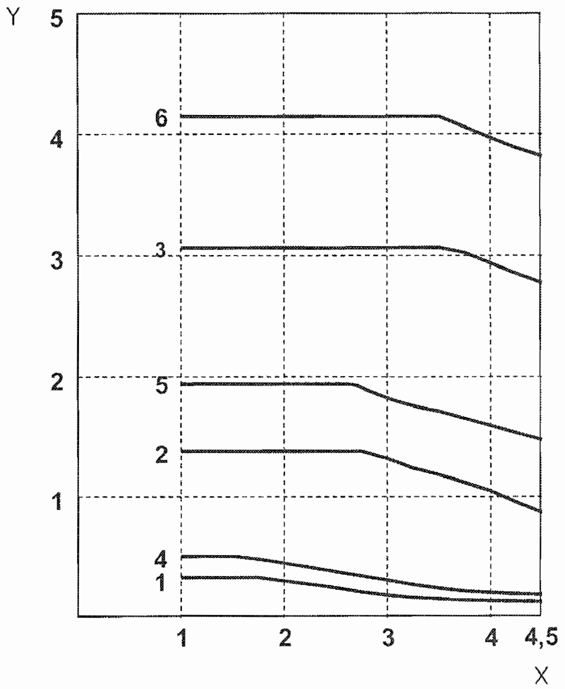


Figure H.2: Correction coefficient φ_δ as a function of the eccentricity δ

Standard Fire resistance R60	Structural Steel Grade :	S 355
	Concrete Grade :	C30/35
	Reinforcing Bars :	S 500
	Reinforcement axis distance u_s:	40 mm

Design curve	Circular section	$A_s / (A_c + A_s)$ %
1	291,1 x 4,5	1,0
2	329,9 x 5,6	1,0
3	406,4 x 6,3	1,0
4	291,1 x 4,5	4,0
5	329,9 x 5,6	4,0
6	406,4 x 6,3	4,0



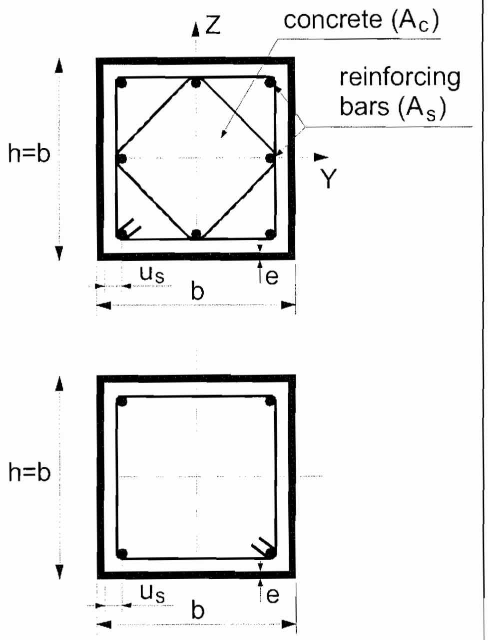


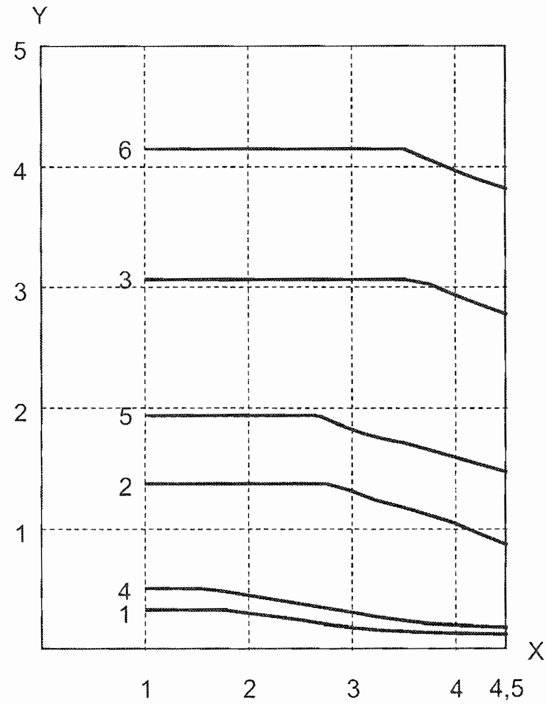
X - Buckling length (m)
Y - Axial buckling load (MN)

Figure H.3 : Example of design graph for CIRCULAR HOLLOW SECTIONS (R60)

Design curve	Square section	$A_s/(A_c + A_s)$ %
1	200 x 6,3	1,0
2	300 x 7,1	1,0
3	400 x 10	1,0
4	200 x 6,3	4,0
5	300 x 7,1	4,0
6	400 x 10	4,0

Standard Fire resistance		
R90	Structural Steel Grade :	S 355
	Concrete Grade :	C30/35
	Reinforcing Bars :	S 500
	Reinforcement axis distance u_s:	40 mm





X - Buckling length (m)
Y - Axial buckling load (MN)

Figure H.4 : Example of design graph for SQUARE HOLLOW SECTIONS (R90)

Annex I
[informative]

Planning and evaluation of experimental models

I.1 Introduction

(1) Test results may be used to assess the fire behaviour of structural members, sub-assemblies or entire structures if they come from tests adequately performed.

(2) Tests may consider one of the possible thermal actions of section 3, of EN 1991-1-2.

(3) Test results may lead to a global assessment of the fire resistance of a structure or a part of it.

(4) Tests may take into account the heating conditions occuring in a fire and the adequate mechanical actions. The result is the time during which the structure maintains its resistance to the combined action of fire and static loads.

(5) Test results may lead to more accurate partial information concerning one or several stages of the aforementioned calculation models.

(6) Partial information may concern the thermal insulation of a slab, the field of temperature in a section, or the kind of failure of a structural element.

(7) Tests may only be carried out after a minimum of 5 months following concreting.

I.2 Test for global assessment

(1) The design of the tested specimen and the mechanical actions applied may reflect the conditions of use.

(2) Tests carried out on the basis of the conventional fire according to CEN standards may be considered to fulfil the aforementioned rule.

(3) The results obtained may only be used for the specific conditions of the test and, if any, for the field of application agreed by CEN standards.

I.3 Test for partial information

(1) The tested specimen may be designed according to the kind of partial information expected.

(2) Testing conditions may differ from the conditions of use of the structural member, if this has no influence on the partial information to be obtained.

(3) The use of the partial information obtained by testing is limited to the same relevant parameters as those studied during the test.

(4) Regarding heat transfer, results are valid for the same size of the element cross section and the same heating conditions.

(5) Regarding failure mechanism, results are valid for the same design of the structure, or part of it, the same boundary conditions and the same levels of loading.

(6) Test results obtained according to the aforementioned rules may be used to replace the appropriate information given by the calculation models of 4.2, 4.3 and 4.4.