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Title: Draft BS EN 1994-1-2 Eurocode 4. Design of composite steel and concrete structures. Part 1-2: Structural fire design.

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Introduction

This draft standard is based on national and international discussions. Your comments on this draft are invited and will assist in the preparation of the consequent standard.

For international standards, comments will be reviewed by the relevant UK national committee before sending the consensus UK vote and comments to the international committee, which will then decide appropriate action. If the international standard is approved, it is usual for the text to be published as a British Standard.

For national standards, comments will be reviewed by the relevant UK national committee and the resulting standards published as a British Standard.

UK Vote

Please indicate whether you consider the UK should submit a negative (with supporting technical reasons) or positive vote on this draft. Please indicate if you are aware of any reason why this draft standard should not be published as a British Standard.

Submission of Comments

- **Annotated drafts are not acceptable and will be rejected.**
- All comments should be submitted online at <https://standardsdevelopment.bsigroup.com/>. You will need to register in order to comment.

Template for comments and secretariat observations

Date: xx/xx/20xx	Document: ISO/DIS xxxx
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1	2	(3)	4	5	(6)	7
MB	Clause No./ Subclause No./Annex (e.g. 3.1)	Paragraph/ Figure/Table/Note	Type of comment	Comment (justification for change) by the MB	Proposed change by the MB	Secretariat observations on each comment submitted
	3.1	Definition 1	ed	Definition is ambiguous and needs clarifying.	Amend to read '...so that the mains connector to which no connection...'	
	6.4	Paragraph 2	te	The use of the UV photometer as an alternative cannot be supported as serious problems have been encountered in its use in the UK.	Delete reference to UV photometer.	

English Version

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

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

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

This draft European Standard is submitted to CEN members for enquiry. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, 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.

This draft European Standard was established by CEN 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 CEN-CENELEC Management Centre has the same status as the official versions.

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

Recipients of this draft are invited to submit, with their comments, notification of any relevant patent rights of which they are aware and to provide supporting documentation.

Warning : This document is not a European Standard. It is distributed for review and comments. It is subject to change without notice and shall not be referred to as a European Standard.



EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

CEN-CENELEC Management Centre: Rue de la Science 23, B-1040 Brussels

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European foreword

This document (prEN 1994-1-2:2024) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1994-1-2:200 and its amendments and corrigenda.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognize the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

0 Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

- EN 1990, Eurocode — Basis of structural and geotechnical design
- EN 1991, Eurocode 1 — Actions on structures
- EN 1992, Eurocode 2 — Design of concrete structures
- EN 1993, Eurocode 3 — Design of steel structures
- EN 1994, Eurocode 4 — Design of composite steel and concrete structures
- EN 1995, Eurocode 5 — Design of timber structures
- EN 1996, Eurocode 6 — Design of masonry structures
- EN 1997, Eurocode 7 — Geotechnical design
- EN 1998, Eurocode 8 — Design of structures for earthquake resistance
- EN 1999, Eurocode 9 — Design of aluminium structures
- New parts are under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators software developers and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1994 (all parts)

EN 1994 applies to the design of steel and concrete composite structures in buildings and civil engineering works. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990, *Basis of structural and geotechnical design*.

EN 1994 is concerned only with the requirements for resistance, serviceability, durability and fire resistance of steel structures. Other requirements, e.g. concerning thermal or sound insulation, are not covered.

EN 1994 is subdivided in various parts:

EN 1994-1-1, *Eurocode 4 — Design of composite steel and concrete structures — Part 1-1: General rules and rules for buildings*;

EN 1994-1-2, *Eurocode 4 — Design of composite steel and concrete structures — Part 1-2: Structural fire design*;

EN 1994-2, *Eurocode 4 — Design of composite steel and concrete structures — Part 2: Bridges*.

0.3 Introduction to prEN 1994-1-2

prEN 1994-1-2 describes the principles, requirements and rules for the structural design of steel and concrete composite buildings exposed to fire.

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

The parts of the Structural Eurocodes relating to fire deal with specific aspects of passive fire protection in terms of designing structures and parts for adequate loadbearing 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 fire regulations or by referring to fire safety engineering for assessing passive and active measures, see EN 1991-1-2.

Supplementary requirements concerning, e.g.:

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

are not given in this standard because they are subject to specification by the competent authority.

0.4 Verbal forms used in the Eurocodes

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may” expresses a course of action permissible within the limits of the Eurocodes.

The verb “can” expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.5 National Annex for prEN 1994-1-2

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing prEN 1994-1-2 can have a National Annex containing all national choices that relate to the design of buildings and civil engineering works constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this standard, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by the relevant parties.

National choice is allowed in prEN 1994-1-2 through notes to the following clauses:

4.5(1) 4.7(2) G.2(1)

National choice is allowed in prEN 1994-1-2 on the application of the following informative annex:

Annex B

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

1 Scope

1.1 Scope of prEN 1994-1-2

(1) prEN 1994-1-2 gives rules for the design of steel-concrete composite structures for the accidental design situation of fire exposure. It only identifies differences from, or supplements to, rules for normal temperature design.

(2) prEN 1994-1-2 only applies to structures, or parts of structures, that are within the scope of EN 1994-1-1 and are designed accordingly.

1.2 Assumptions

(1) The assumptions of EN 1990 apply, along with the following:

- The choice of the relevant design fire scenario is made by appropriate qualified and experienced personnel or is given by the relevant national regulation;
- Any fire protection measure taken into account in the design will be adequately maintained.

2 Normative references

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

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

EN 1990, *Eurocode — Basis of structural and geotechnical design*

EN 1991 (all parts), *Eurocode 1 — Actions on structures*

FprEN 1991-1-2:2023, *Eurocode 1 — Actions on structures — Part 1-2: Actions on structures exposed to fire*

EN 1992-1-1, *Eurocode 2 — Design of concrete structures — Part 1-1: General rules and rules for buildings, bridges and civil engineering structures*

EN 1993-1-1:2022, *Eurocode 3 — Design of steel structures — Part 1-1: General rules and rules for buildings*

prEN 1994-1-1:2024, *Eurocode 4 — Design of composite steel and concrete structures — Part 1-1: General rules and rules for buildings*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990 and EN 1991-1-2 and the following apply.

3.1.1

axis distance

distance between the centre of the reinforcing bar and the nearest edge of concrete

3.1.2

critical temperature of reinforcement

temperature of reinforcement at which loadbearing failure of the element is expected to occur at a given stress

3.1.3

critical temperature of structural steel element

temperature for a given load level, at which failure is expected to occur in a structural steel element assuming a uniform temperature distribution

3.1.4

effective cross-section

cross-section of the member in structural fire design used in the effective cross-section method

Note 1 to entry: It is obtained by removing parts of the cross-section with assumed zero strength and stiffness.

3.1.5

failure time of fire protection system

duration of protection of member against direct fire exposure (e.g. when the fire protective sheathing or other protection fall off the composite member, or when a structural member initially protecting the member fails due to collapse, or when the protection from another structural member is no longer effective due to excessive deformation)

3.1.6

fire protection material

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

3.1.7

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

3.1.8

part of structure

isolated part of a structure with appropriate support and boundary conditions

3.1.9

protected members

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

3.1.10

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

3.2 Symbols and abbreviations

For the purposes of this document, the following symbols apply.

3.2.1 Symbols

3.2.1.1 Latin upper-case letters

Symbol	Definition
A	cross-sectional area
A_c	cross-sectional area of concrete part
A_f	cross-sectional area of a steel flange
A_i	area of part i of the cross-section or exposed surface area of part i of the steel cross-section per unit length
$A_{i,\theta}$	Reduced area of part i of the cross-section at temperature θ
A_i / V_i	section factor of part i of the steel cross-section (non-protected member)
A_p	area of the inner surface of the fire protection material per unit length of the steel beam
A_p / V	section factor of the steel cross-section with box protection
$A_{p,i}$	area of the inner surface of the fire protection material per unit length of part i of the steel member
$A_{p,i} / V_i$	section factor of part i of the steel cross-section
A_s	cross-sectional area of the reinforcing bars
E	integrity criterion
E_a	characteristic value for the modulus of elasticity of structural steel at 20 °C
$E_{a,\theta}$	characteristic value for the slope of the linear elastic range of the stress-strain relationship of structural steel at elevated temperatures
$E_{c,sec,\theta}$	characteristic value for the secant modulus of concrete in the fire situation, given by $f_{c,\theta}$ divided by $\varepsilon_{cu,\theta}$
E_d	design effect of actions for normal temperature design
$E_{d,fi}$	design effect of actions in the fire situation, assumed to be time independent
$(EI)_{eff,fi}$	effective flexural stiffness in the fire situation
E_k	characteristic value for the modulus of elasticity at 20 °C
E_s	characteristic value for the modulus of elasticity of modulus of elasticity of reinforcing bars at 20 °C

$E_{s,\theta}$	characteristic value for the slope of the linear elastic range of the stress-strain relationship of reinforcing steel at elevated temperatures
F_a	tensile force in the steel section
F_c	compressive force in the slab
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
L	system length of a column in the relevant storey
LC	lightweight concrete
$M_{Rd,fi+}; M_{Rd,fi-}$	design value of the sagging or hogging moment resistance in the fire situation
$M_{Rd,fi,t}$	design moment resistance in the fire situation at time t
NC	normal weight concrete
$N_{fi,cr}$	elastic critical load in the fire situation
$N_{pl,Rd,fi}$	design value of the plastic resistance to axial compression in the fire situation
$N_{pl,R,fi}$	value of $N_{pl,Rd,fi}$ when the factors $\gamma_{M,fi,a}$, $\gamma_{M,fi,s}$ and $\gamma_{M,fi,c}$ are taken as 1,0
$N_{Rd,fi}$	design value of the resistance of a member in axial compression in the fire situation
P_{Rd}	design shear resistance of a shear connector at normal temperature.
$P_{Rd,fi}$	design shear resistance of a shear connector in the fire situation
R	Load bearing criterion
R30, R60, R90, R120, R180	a member complying with the loadbearing criterion for 30, 60, 90, 120 or 180 minutes in standard fire exposure
R_d	design resistance for normal temperature
$R_{d,fi,t}$	design resistance in the fire situation, at time t
V_i	volume of part i of the steel cross-section per unit length
$X_{d,fi}$	design values of mechanical (strength and stiffness) material properties in the fire situation
X_k	characteristic value of a strength or stiffness property for normal temperature
$X_{k,\theta}$	characteristic value of a strength or stiffness property in the fire situation

3.2.2 Latin lower-case letters

a_w	throat thickness of weld
b	width
b_1	width of the bottom flange of the steel section
b_2	width of the top 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_{eff}	effective width of the concrete slab
c	concrete cover from edge of concrete to border of structural steel
c_a	specific heat of steel
c_c	specific heat of concrete
c_p	specific heat of the fire protection material
d	diameter of the composite column made of concrete filled circular hollow section or diameter of the studs welded to the web of the steel section
d_p	thickness of the fire protection material
e	thickness of hollow section
e_1	thickness of the bottom flange of the steel section
e_2	thickness of the top flange of the steel section
e_f	thickness of the flange of the steel section
e_p	thickness of the bottom plate of a shallow floor beam (type A)
e_w	thickness of the web of the steel section
ef	external fire exposure curve
$f_{ap,\theta}$	proportional limit of structural 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
$f_{ay,\theta}$	maximum stress level or effective yield strength of structural steel in the fire situation
$f_{ay,\theta,i}$	maximum stress level or effective yield strength of part i of the steel section in the fire situation
$f_{ay,\theta_{cr}}$	effective yield strength of structural steel at critical temperature θ_{cr}
f_{ay}	characteristic or nominal value for the yield strength of structural steel at 20 °C
f_{ck}	characteristic value of the compressive cylinder strength of concrete at 28 days and at 20 °C
$f_{ck,j}$	characteristic value of the compressive cylinder strength of part j of the concrete at 28 days and at 20 °C

$f_{c,\theta}$	characteristic value for the compressive cylinder strength of concrete at temperature θ
$f_{c,\theta,20\text{ °C}}$	characteristic value for the compressive cylinder strength of concrete heated to a maximum temperature $\theta_{c,\max}$ and having cooled down to 20 °C
$f_{c,\theta_{\max}}$	characteristic value for the compressive cylinder strength of concrete in the fire situation at maximum temperature $\theta_{\max}$
f_k	characteristic value of strength of the material
f_{sy}	characteristic or nominal value for the yield strength of a reinforcing bar at 20 °C
$f_{sp,\theta}$	proportional limit of reinforcing steel in the fire situation
$f_{sy,\theta}$	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 sheeting
h_2	height of the concrete part of a composite slab within the sheeting
h_3	thickness of the screed situated on top of the concrete
h_c	thickness of the concrete slab or depth of the composite column made of a totally encased section
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_v	height of the stud welded on the web of the steel section
h_w	height of the web of the steel section
$k_{c,\theta}$	reduction factor for the compressive strength of concrete giving the strength at elevated temperature $f_{c,\theta}$
$k_{c,\theta,j}$	reduction factor for the compressive strength of part j of concrete giving the strength at elevated temperature $f_{c,\theta}$
$k_{c,\theta,\max}$	reduction factor for the compressive strength of concrete giving the strength corresponding to the maximum temperature $\theta_{c,\max}$

$k_{E,\theta}$	reduction factor for the elastic modulus of structural steel or reinforcing bars giving the slope of the linear elastic range at elevated temperature $E_{a,\theta}$
$k_{E,\theta,i}$	reduction factor for the elastic modulus of part i of structural steel or reinforcing bars 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 or reinforcing bars giving the maximum stress level at elevated temperature $f_{a,\theta}$
$k_{y,\theta,i}$	reduction factor for the yield strength of structural steel or reinforcing bars for the elemental area A_i giving the maximum stress level at elevated temperature $f_{a,\theta}$ or $f_{s,\theta}$
$k_{p,\theta}$	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_{sh}	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 q
k_θ	temperature-dependent reduction factor for a strength or stiffness property
l	length at 20 °C of the member
l_1, l_2, l_3	specific dimensions of the re-entrant or trapezoidal steel sheeting
l_a	width of support of the concrete slab
l_{fi}	buckling length of the column in the fire situation
l_w	length of weld
$m_{Ed,fi}$	design transverse bending moment per unit length of the bottom plate or flange under fire conditions
$m_{pl,Rd,\theta}$	plastic bending resisting moment per unit length of the bottom plate or flange at temperature q
t	time, duration of fire exposure
u_s	minimum axis distance of the reinforcing bars
$u_1 ; u_2$	minimum axis distance or shortest distance from the centre of the reinforcing bar to any point on a web of the steel sheet
$z_i ; z_j$	distance from the plastic neutral axis to the centroid of the elemental area A_i or A_j

3.2.2.1 Greek upper-case letters

Δl	temperature induced expansion of the member
$\Delta l/l$	thermal expansion
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 ambient gas temperature [°C] during the time interval Δt
Φ	configuration factor

3.2.2.2 Greek lower-case letters

α_1	reduction of yield strength of a flange (or plate) due to transversal shear and bending
α_c	convective heat transfer coefficient
α_{slab}	coefficient to allow the use of the assumption of the rectangular stress block when designing concrete slabs and composite shallow floor beams
β	load level of a plate (or flange) under transversal shear
$\gamma_{M,fi}$	partial factor for a material property in the fire situation
$\gamma_{M,a,fi}$	partial factor for the strength of structural steel in the fire situation
$\gamma_{M,c,fi}$	partial factor for the strength of concrete in the fire situation
$\gamma_{M,s,fi}$	partial factor for the strength of reinforcing bars in the fire situation
$\gamma_{M,v,fi}$	partial factor for the shear resistance of stud connectors in the fire situation
γ_v	partial factor for the shear resistance of stud connectors at normal temperature
$\epsilon_{a,\theta}$	steel strain in the fire situation
$\epsilon_{ae,\theta}$	ultimate strain of steel in the fire situation
$\epsilon_{ay,\theta}$	yield strain of steel in the fire situation
$\epsilon_{ap,\theta}$	strain at the proportional limit of steel in the fire situation
$\epsilon_{au,\theta}$	limiting strain for yield strength of steel in the fire situation
$\epsilon_{c,\theta}$	concrete strain in the fire situation
$\epsilon_{c1,\theta}$	concrete strain corresponding to $f_{c,\theta}$
$\epsilon_{cu1,\theta}$	maximum concrete strain in the fire situation
ϵ_f	emissivity coefficient of the fire
ϵ_{fi}	coefficient for cross-sectional classification for the fire situation
ϵ_m	surface emissivity of the member
ϕ	reduction factor for the residual concrete strength after heating and cooling down in physical based models
ϕ_s	diameter of stirrup

ϕ_r	diameter of longitudinal reinforcement
ϕ_b	diameter of bar through the web
$\varphi_{i,q}$	the reduction coefficient depending on the effect of thermal stresses of part i or flexural stiffness correction factor
η	load level of a plate (or flange) under transversal bending
η_{fi}	reduction factor applied to E_d in order to obtain $E_{fi,d}$
$\eta_{fi,t}$	load level for fire design
θ_a	steel temperature
$\theta_{a,t}$	steel temperature at time t
θ_c	concrete temperature
$\theta_{c,max}$	maximum temperature of concrete
θ_{cr}	critical temperature of a structural member
θ_i	temperature in the elemental area A_i
θ_s	reinforcing steel temperature
θ_t	ambient gas temperature at time t
θ_v	temperature of stud connectors
λ_a	thermal conductivity of steel
λ_c	thermal conductivity of concrete
λ_p	thermal conductivity of fire protection material
$\bar{\lambda}_\theta$	relative slenderness in the fire situation
ρ_a	density of steel
ρ_c	density of concrete
$\rho_{c,LC}$	density of lightweight concrete
ρ_p	density of fire protection material
σ	Stephan Boltzmann constant ($= 5,67 \cdot 10^{-8} [W/(m^2K^4)]$)
$\sigma_{c,\theta}$	stress of concrete under compression in the fire situation
τ_{Ed}	design transversal shear per unit length [kN/m] of the bottom plate at normal temperature
$\tau_{pl,Rd}$	plastic shear resistance per unit length [kN/m] of the bottom plate at normal temperature
χ	reduction coefficient

3.2.3 Additional symbols used in Annex A of prEN 1994-1-2

3.2.3.1 Greek lower-case symbols

$\sigma_{a, \theta}$ stress of the steel section in the fire situation

3.2.4 Additional symbols used in Annex B of prEN 1994-1-2

3.2.4.1 Latin upper-case letters

A concrete volume per metre of member length

A/L_r rib geometry factor

L_r exposed area of the rib per metre of rib length

N_s normal force in the hogging reinforcement

Y Y (major) axis

3.2.4.2 Latin lower-case letters

a_i coefficients for determination of the fire resistance with respect to thermal insulation

b_i coefficients for the determination of the temperatures of the parts of the steel sheeting

b_r width of the upper flange of the steel sheeting

c_i coefficients for the determination of the temperature of the reinforcing bars in the rib

d_{ef} height of the top re-entrant stiffener

d_i coefficients for the determination of the limiting temperature

$h_{1,mod}$ modified height of the concrete part of a composite slab above the sheeting in case of profiled steel sheeting with re-entrant stiffener

$h_{2,mod}$ modified height of the concrete part of a composite slab within the sheeting in case of profiled steel sheeting with re-entrant stiffener

h_p overall height of the profiled steel sheeting, excluding the height of the top re-entrant stiffener

t_i fire resistance time with respect to thermal insulation

u_3 distance from the centre of the reinforcing bar to the lower flange of the steel sheet

w_s width of the top re-entrant stiffener

z value which defines the position of the bar in the rib

3.2.4.3 Greek upper-case letters

Φ view factor of the upper flange

3.2.4.4 Greek lower-case letters

α angle of the web

θ_{lim} limiting temperature

3.2.5 Additional symbols used in Annex C of prEN 1994-1-2

3.2.5.1 Latin upper-case letters

A_r / V_r section factor of stiffeners

F^+, F^- total compressive force in the composite section in case of sagging or hogging bending moments

N number of shear connectors related to the governing critical length

$R_{y,Rd,fi}$ design yield resistance of the web with the stiffeners

T^+, T^- tensile force

3.2.5.2 Latin lower-case letters

b_{eff} effective width of the concrete slab in a region of hogging moment

h_{cr} depth corresponding to a concrete temperature below 250 °C

h_u thickness of the compressive zone

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

n total number of concrete layers in compression

r root radius for a rolled section

s_s length of the rigid support

3.2.5.3 Greek lower-case letters

θ_r temperature of stiffener

3.2.6 Additional symbols used in Annex D of prEN 1994-1-2

3.2.6.1 Latin upper case letters

$V_{pl,Rd,fi}$ design value of the shear plastic resistance in the fire situation

$V_{Ed,fi}$ design value of the shear force in the fire situation

3.2.6.2 Latin lower-case letters

b_{fi}	width reduction of top flange in the fire situation
$h_{c,fi}$	thickness reduction of the concrete slab
h_h	top part of depth of the web
h_l	bottom part of depth of the web
k_a	reduction factor of the yield point of the lower flange
u_i	the distance from the axis of the reinforcing bar to the inner side of the flange
u_{si}	the distance from the axis of the reinforcing bar to the concrete surface

3.2.7 Additional symbols used in Annex E of prEN 1994-1-2

3.2.7.1 Latin upper-case letters

A_m	directly heated surface area of member per unit length
A_m / V	section factor of structural member
$E_{a,f}$	characteristic value for the modulus of elasticity of a steel flange at 20 °C
$E_{a,f,t}$	characteristic value for the modulus of elasticity corresponding to the average flange temperature
$(EI)_{c,z,fi}$	flexural stiffness of concrete in the fire situation (related to the central axis Z of the composite cross-section)
$(EI)_{f,z,fi}$	flexural stiffness of the two flanges of the steel section in the fire situation (related to the central axis Z of the composite cross-section)
$(EI)_{s,z,fi}$	flexural stiffness of the reinforcing bars in the fire situation (related to the central axis Z of the composite cross-section)
$(EI)_{eff,z,fi}$	effective flexural stiffness (related to the central axis Z of the composite cross-section) in the fire situation
$(EI)_{w,z,fi}$	flexural stiffness of the web of the steel section in the fire situation (related to the central axis Z of the composite cross-section)
$I_{s,z}$	the second moment of area of the reinforcing steel about the central axis Z
$N_{cr,z,fi}$	elastic critical load around axis Z in the fire situation
$N_{pl,Rd,f,fi}$	design value of the plastic resistance to axial compression of the two flanges
$N_{pl,Rd,w,fi}$	design values of the plastic resistance to axial compression of the web
$N_{pl,Rd,c,fi}$	design values of the plastic resistance to axial compression of the concrete in the fire situation
$N_{pl,Rd,s,fi}$	design values of the plastic resistance to axial compression of the reinforcing bars in the fire situation
N_{Rd}	design value of the resistance of a member in axial compression at 20 °C

$N_{Rd,\delta}$	design axial buckling load in case of an eccentric load at normal temperature
$N_{Rd,\delta,fi}$	design buckling load for a column subjected to a load with an eccentricity
$N_{Rd,z,fi}$	design value of the resistance of a member in axial compression, for buckling around the axis Z, in the fire situation
V	volume of the member per unit length
Z	Z (weak) central axis of the composite cross-section

3.2.7.2 Latin lower-case letters

$b_{c,fi}$	thickness reduction of the concrete area in the fire situation
$h_{w,fi}$	height reduction of the web in the fire situation
$k_{E,t}$	reduction factor of the modulus of elasticity of the reinforcing bars
$k_{y,t}$	reduction factor of the yield strength of the reinforcing bars

3.2.7.3 Greek lower-case letters

δ	eccentricity
$\theta_{c,t}$	average temperature in the concrete
$\theta_{f,t}$	average flange temperature
χ_z	reduction coefficient for bending around axis Z

3.2.8 Additional symbols used in Annex F of prEN 1994-1-2

3.2.8.1 Latin upper-case letters

B	shorter outer dimension of a rectangular or elliptical cross-section
D	outer diameter of a circular cross-section
$(EI)_{eff,II,fi}$	effective flexural stiffness for second-order analysis in the fire situation
$E_{i,\theta}$	the modulus of elasticity of part i at temperature θ_i
H	larger outer dimension of a rectangular or elliptical cross-section
I_i	second moment of area of part i of the cross-section
$K_{e,II}$	correction factor to be used in the design of composite columns
K_o	calibration factor to be used in the design of composite columns
K_θ	correction factor to be used in the design of composite columns at elevated temperature

3.2.8.2 Greek lower-case letters

α_M	coefficient related to bending of a composite column
β_i	coefficient for calculation of equivalent reinforcing bars temperature
$\theta_{c,eq}$	equivalent temperature of the concrete core
$\theta_{s,eq}$	equivalent temperature of the reinforcing bars
$\theta_{a,eq}$	equivalent temperature of the steel section
ρ_s	reinforcement ratio of the concrete core

3.2.9 Additional symbols used in Annex G of prEN 1994-1-2

3.2.9.1 Latin upper-case letters

K	ratio of tensile force per unit width of the reinforcing mesh in the shorter side cross-section to tensile force per unit width of the reinforcing mesh in the longer side cross-section
L	longer span of the floor design zone
L_1, L_2	spans of composite slab
$M_{0,fi}$	sagging moment resistance per unit width of the composite slab cross-section along the longer side of the floor design zone
$M_{Ed,b,1,fi}$	design value of bending moment in the fire situation in the two perimeter secondary beams
$M_{Ed,b,2,fi}$	design value of bending moment in the fire situation in the primary perimeter beams
$M_{Rd,j,fi}$	design value of the sagging moment resistance of the internal secondary composite beam j
$V_{Ed,b,1,fi}$	design value of shear force in the fire situation in the two perimeter secondary beams
$V_{Ed,b,2,fi}$	design value of bending moment in the fire situation in the primary perimeter beams

3.2.9.2 Latin lower-case letters

a	aspect ratio of the floor design zone
b_j	width of the composite slab supported by the internal composite beam j
c_M	factor in respect to primary or secondary beams taking into account tensile membrane action
d	distance between the centre of the reinforcing mesh (median plane) and the top side of the composite slab
e	overall enhancement factor under tensile membrane action
$(g_0)_1, (g_0)_2$	parameters defining the compressive stress block of concrete for sagging moment resistance of the composite slab per unit width in the shorter and the longer side cross-sections respectively
n_{ub}	total number of internal secondary composite beams
$q_{Ed,fi}$	uniformly distributed load to be supported by the floor design zone in the fire

$q_{Rd,fi}$	reference loadbearing capacity of the composite slab based on a yield line solution
$q_{Rd,slab,fi}$	loadbearing capacity of the composite slab under tensile membrane action
w	deflection limit of a composite floor in the fire situation under tensile membrane action

3.2.9.3 Greek lower-case letters

α_c	coefficient of thermal expansion of concrete
θ_1	temperature of the unexposed side of the composite slab
θ_2	temperature of the exposed side of the composite slab
μ	coefficient defining the ratio of sagging moment resistances of the composite slab in orthogonal directions

3.2.10 Additional symbols used in Annex H of prEN 1994-1-2

3.2.10.1 Latin upper-case letters

A_i, A_w, A_r	parameters for determination of the bending resistance of a shallow floor beam
B_i, B_w, B_r	parameters for determination of the bending resistance of a shallow floor beam
C_i, C_w, C_r	parameters for determination of the bending resistance of a shallow floor beam
$N_{c,Rd,fi}$	maximum design value of the concrete compressive force
$N_{t,Rd,fi}$	tensile resistance of the steel section
$V_{w,Rd,fi}$	shear resistance of the steel web

3.2.10.2 Latin lower-case letters

b_f	flange width
b_p	width of bottom plate of a shallow floor beam (type A)
$b_{p,eff,fi}$	reduced width of the bottom plate acting as support of the concrete slab welded to the structural steel section (type A) in the fire situation
$b_{fb,eff,fi}$	reduced width of the bottom flange acting as support of the concrete slab welded to the structural steel flange (type B) in the fire situation
b_{fb}	width of the bottom flange of a shallow floor beam (type B)
b_{ft}	width of the top flange of a shallow floor beam (type B)
b_w	width of in situ concrete between two slabs of a shallow floor beam (types A and B)
c_z	concrete cover above the top flange of the steel section
e_{fb}	thickness of the bottom flange of a shallow floor beam (type B)
e_{ft}	thickness of the top flange of a shallow floor beam (type B)

$h_{c,1}$	height of in situ concrete between slab and top flange of a shallow floor beam (types A and B)
h_s	concrete height defining the position of longitudinal reinforcement in relation to the web of the steel profile for shallow floor beams (types A and B)
k_c	coefficient depending on slab covering the surface of the bottom plate
k_h	coefficient for the calculation of the maximum design value of the concrete compressive force
k_t	coefficient for the calculation of the bottom flange temperature of a shallow floor beam
$k_{wy,0}$	Reduction factor for shear resistance of the steel web
l_a	width of support of the concrete slab (same as for l_p in 7.4.2.4)
u_c	horizontal concrete cover of the longitudinal reinforcement of a shallow floor beam (types A and B)
u_r	vertical concrete cover of the longitudinal reinforcement of a shallow floor beam (types A and B)
u_w	concrete cover between the longitudinal reinforcement and the steel web of a shallow floor beam (types A and B)

3.2.10.3 Greek lower-case letters

θ_{fb}	temperature of the bottom flange
θ_p	temperature of the welded plate
θ_w	temperature in the web

3.2.11 Additional symbols used in Annex I of prEN 1994-1-2

3.2.11.1 Latin upper-case letters

$N_{oc,Rd}$	design value of compression resistance of concrete slab at the centre of an opening
$V_{oc,Rd}$	design shear resistance of the concrete slab at the opening position

3.2.11.2 Latin lower-case letters

$b_{eff,o}$	effective widths for local shear and bending
$k_{a,fi}$	reduction factor to the Vierendeel bending resistance due to composite action

4 Basis of design

4.1 General

(1) The structural fire design shall be in accordance with the general rules given in EN 1990 and EN 1991 (all parts) and the specific design provisions for steel and concrete composite structures given in EN 1994-1-1.

(2) 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 loadbearing function during the relevant fire exposure.

(3) 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 to ensure that both integrity failure and insulation failure do not occur.

NOTE For non-loadbearing elements, the loadbearing function under self-weight is expected to be maintained in order to fulfil the separating function.

(4) Deformation criteria shall be applied where the means of fire protection require consideration of the deformation of the loadbearing structure.

(5) Consideration of the deformation of the loadbearing structure may be omitted when the efficiency of the means of protection has been evaluated according to 5.2.3.

(6) Deformation criteria shall be applied where the design criteria for separating elements require taking into account deformation of the loadbearing structure.

(7) Consideration of the deformation of the loadbearing structure may be omitted when the separating elements fulfil requirements of a nominal fire exposure.

(8) Composite steel and concrete structures designed in accordance with EN 1993-1-1 should be considered to have sufficient minimum robustness for the accidental fire design case. Where additional requirements are necessary the procedures of prEN 1991-1-7:2023, Annex A, apply.

4.2 Nominal fire exposure

(1) For standard fire exposure, elements shall comply with one of the following functions or combinations of function defined in EN 1991-1-2 during the required time of fire exposure:

- loadbearing function only: loadbearing capacity (R);
- separating function only: integrity (E) and, when requested, insulation (I);
- separating and loadbearing functions: criteria R, E and, when requested, I.

NOTE 1 The loadbearing function is assumed to be satisfied when loadbearing capacity is maintained.

NOTE 2 The separating function is assumed to be satisfied when integrity and, when requested, insulation are maintained.

NOTE 3 Integrity is assumed to be maintained when a separating element of building construction, exposed to fire on one side, prevents the passage through it of flames and hot gases and the occurrence of flames on the unexposed side.

NOTE 4 Insulation is assumed to be maintained when the average temperature rise over the whole of the unexposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K.

(2) With the external fire exposure curve the same functions (R, E, I) apply, however the reference to this specific curve shall be identified by the letters "ef".

(3) With the hydrocarbon fire exposure curve the same functions (R, E, I) apply, however the reference to this specific curve shall be identified by the letters "HC".

4.3 Physically based fire exposure

(1) The loadbearing function shall be maintained during the complete duration of the fire, including the cooling phase, or during a required period of time according to FprEN 1991-1-2:2023, 4.4(4).

(2) For the verification of the separating function the following applies, assuming that the normal temperature is 20 °C:

- the average temperature rise of the unexposed side of the construction should be limited to 140 K and the maximum temperature rise of the unexposed side should not exceed 180 K during the heating phase until the maximum temperature in the fire compartment is reached;
- the average temperature rise of the unexposed side of the construction should be limited to 200 K and the maximum temperature rise of the unexposed side should not exceed 240 K during the cooling phase.

4.4 Actions

(1) Thermal and mechanical actions shall be taken from EN 1991-1-2.

4.5 Design values of material properties

(1) Design values of mechanical (strength and stiffness) material properties for the fire situation $X_{d,fi}$ are defined as follows:

$$X_{d,fi} = k_{\theta} X_k / \gamma_{M,fi} \quad (4.1)$$

where

- X_k is the characteristic value of a strength or stiffness property (generally f_k or E_k) for normal temperature design according to EN 1994-1-1;
- k_{θ} is the temperature-dependent reduction factor ($X_{k,\theta} / X_k$) for a strength or stiffness property, see Section 5.3;
- $\gamma_{M,fi}$ is the partial factor for the relevant mechanical material property for the fire situation, as defined according to 4.5(2).

NOTE 1 For mechanical properties of steel and concrete, the recommended values of the partial factor for the fire situation are $\gamma_{M,a,fi} = 1,0$; $\gamma_{M,s,fi} = 1,0$; $\gamma_{M,c,fi} = 1,0$ unless the National Annexes of EN 1992-1-2 and EN 1993-1-2 give a different value.

NOTE 2 The recommended value of the partial factor for the shear resistance of stud connector in the fire situation is $\gamma_{M,v,fi} = 1,0$ unless the National Annex gives a different value.

(2) The design values of thermal material properties for the fire situation should be taken equal to the characteristic values.

NOTE The characteristic values of thermal properties correspond to the mean values.

4.6 Verification methods

(1) The model of the structural system adopted for design shall reflect the performance of the structure in the fire situation.

(2) Mechanical resistance shall be verified for the required duration of fire exposure t according to Formula (4.2).

$$E_{d,fi} \leq R_{d,fi,t} \quad (4.2)$$

where

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

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

(3) The structural analysis for the fire situation should be carried out according to EN 1990:2023, 7.1.5.

NOTE For verifying resistance requirements based on the standard fire curve, unless otherwise specified, a member analysis is sufficient.

(4) Where application rules given in this Part of EN 1994 are valid only for the standard fire curve, this is identified in the relevant clauses.

(5) The following design methods may be used in order to satisfy 4.6(2):

- use of tabulated design data for specific types of members, see Clause 6;
- use of simplified design methods for specific types of members, see Clause 7; and
- use of advanced design methods for the analysis of members, parts of the structure or the entire structure, see Clause 8.

(6) 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.

4.7 Member analysis

(1) The design effect of actions should be determined for time $t = 0$ using combination factors according to FprEN 1991-1-2:2023, 6.3.

(2) As a simplification, the value of $\eta_{fi} = 0.65$, except for imposed loads according to category E as given in EN 1991-1-1 (areas susceptible to accumulation of goods, including access areas) where the value of $\eta_{fi} = 0.7$ should be used.

NOTE As indicated in FprEN 1991-1-2:2023, 6.3, different values for the reduction factor η_{fi} can be given in the National Annex.

(3) For use of tabulated design data in Clause 6, the load level for fire design $\eta_{fi,t}$ should be calculated from Formula (4.3):

$$\eta_{fi,t} = \frac{E_{d,fi}}{R_d} \quad (4.3)$$

where

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

R_d is the design resistance for normal temperature.

- (4) The effects of thermal deformations resulting from thermal gradients across the cross-section shall be taken into account.
- (5) The effects of axial or in-plane thermal expansions may be neglected.
- (6) The kinematic boundary conditions at the supports and ends of members, applicable at time $t = 0$, may be assumed to remain unchanged throughout the fire exposure.
- (7) Tabulated design data and simplified or advanced design methods given in this document may be used for verifying members under fire conditions.

4.8 Analysis of parts of the structures

- (1) The effect of actions should be determined for time $t = 0$ using combination factors according to FprEN 1991-1-2:2023, 6.3.
- (2) Within the part of the structure to be analysed, the relevant failure mode in fire, the temperature-dependent material properties and member stiffness, effects of thermal expansions and deformations (indirect fire actions) shall be taken into account.
- (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) As an alternative to 4.8(1), the reactions at supports and internal forces and moments at boundaries of part of the structure may be obtained from structural analysis for normal temperature design as given in 4.7.
- (5) A composite floor constructed of primary steel/composite beams, secondary steel/composite beams and composite slabs can be divided into a number of structural zones, each of which being analysed separately. The fire resistance of each zone can be justified by taking into account the tensile membrane effect that can develop in the composite slab supported by the surrounding edge beams, thereby allowing elimination of fire protection to the internal secondary beams. The edge beams should be able to resist the additional vertical loads from the load distribution path of tensile membrane action.
- (6) This tensile membrane effect shall be justified by advanced design methods described in Clause 8 taking into account material and geometrical non-linearities or by use of Annex G, which provides a simplified approach.

4.9 Global structural analysis

- (1) A global structural analysis for the fire situation shall take into account:
 - the relevant failure mode in fire exposure;
 - the temperature-dependent material properties and member stiffness; and
 - effects of thermal expansions and deformations (indirect fire actions).

5 Material properties

5.1 General

- (1) Unless given as design values, the values of material properties given in Clause 5 shall be treated as characteristic values.
- (2) The mechanical properties of concrete, reinforcing steel and prestressing steel at normal temperature (20 °C) shall be taken as those given in EN 1992-1-1 for normal temperature design.

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

5.2 Thermal properties

5.2.1 Carbon steel

5.2.1.1 Emissivity coefficient

(1) The values of surface emissivity ε_m in relation to the different types of steel should be taken from Table 5.1.

Table 5.1 — Surface emissivity of different types of steel

Type of steel	$\varepsilon_m (\leq 500^\circ\text{C})$	$\varepsilon_m (> 500^\circ\text{C})$
Carbon steel	0,7	
HDG steel ^a	0,35	0,7

^a Steel that has been hot-dip galvanized according to EN ISO 1461 and with steel composition according to Category A or B of EN ISO 14713-2:2020, Table 1.

5.2.1.2 Thermal conductivity

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

$$\lambda_a = 54 - 3,33 \cdot 10^{-2} \theta_a \text{ [W/(m.K)] for } 20^\circ\text{C} \leq \theta_a \leq 800^\circ\text{C} \quad (5.1)$$

$$\lambda_a = 27,3 \text{ [W/(m.K)] for } 800^\circ\text{C} < \theta_a \leq 1\,200^\circ\text{C} \quad (5.2)$$

where

θ_a is the steel temperature.

NOTE The variation of the thermal conductivity and temperature is illustrated in Figure 5.1.

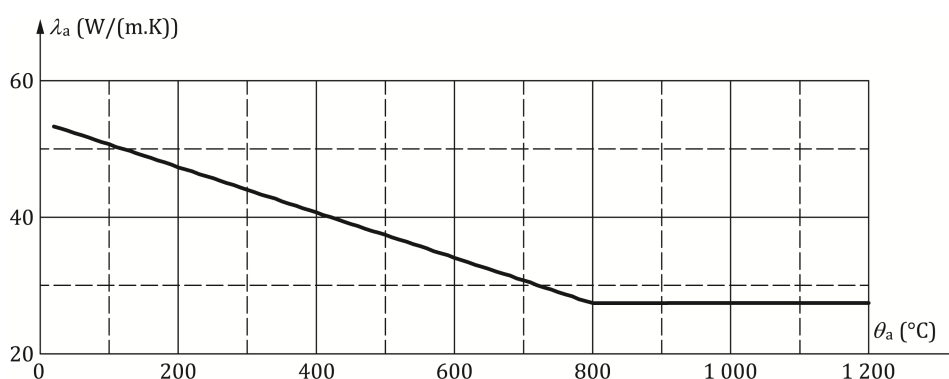


Figure 5.1 — Thermal conductivity of steel as a function of temperature

5.2.1.3 Specific heat

(1) The specific heat of steel c_a valid for all structural and reinforcing steel qualities should be determined from Formulae (5.3) to (5.6):

$$c_a = 425 + 7,73 \cdot 10^{-1} \theta_a - 1,69 \cdot 10^{-3} \theta_a^2 + 2,22 \cdot 10^{-6} \theta_a^3 \text{ [J/(kg.K)] for } 20 \leq \theta_a \leq 600 \text{ °C} \quad (5.3)$$

$$c_a = 666 - \left(\frac{13\,002}{\theta_a - 738} \right) \text{ [J/(kg.K)] for } 600 < \theta_a \leq 735 \text{ °C} \quad (5.4)$$

$$c_a = 545 - \left(\frac{17\,820}{\theta_a - 731} \right) \text{ [J/(kg.K)] for } 735 < \theta_a \leq 900 \text{ °C} \quad (5.5)$$

$$c_a = 650 \text{ [J/(kg.K)] for } 900 < \theta_a \leq 1\,200 \text{ °C} \quad (5.6)$$

where

θ_a is the steel temperature.

NOTE The variation of the specific heat with temperature is illustrated in Figure 5.2.

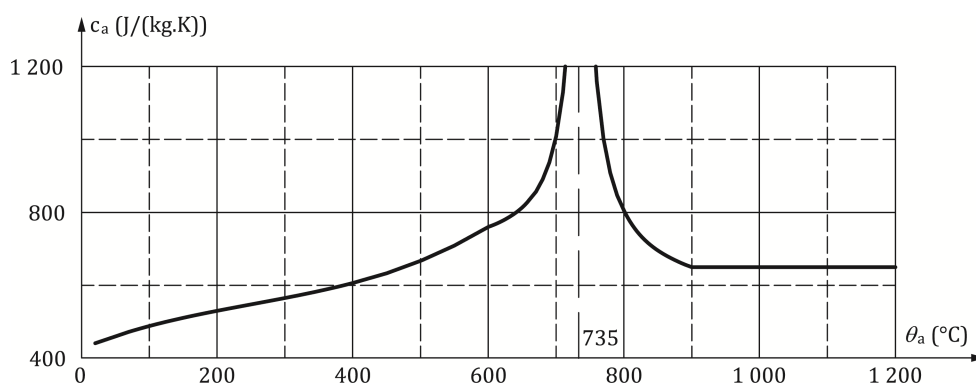


Figure 5.2 — Specific heat of steel as a function of temperature

5.2.1.4 Density

(1) The density of steel ρ_a may be considered as independent of the steel temperature. The following value should be taken:

$$\rho_a = 7850 \text{ [kg/m}^3\text{]} \quad (5.7)$$

5.2.2 Concrete

5.2.2.1 Emissivity coefficient

(1) The emissivity coefficient for concrete surface should be taken as 0.7.

5.2.2.2 Thermal conductivity

(1) The thermal conductivity λ_c of normal weight concrete should be determined from Formulae (5.8) to (5.10):

$$\lambda_c = 2 - 0,2451(\theta_c / 100) + 0,0107(\theta_c / 100)^2 \text{ [W/(m.K)] for } \theta_c \leq 140 \text{ }^\circ\text{C} \quad (5.8)$$

$$\lambda_c = -0,02604\theta_c + 5,324 \text{ [W/(m.K)] for } 140 < \theta_c < 160 \text{ }^\circ\text{C} \quad (5.9)$$

$$\lambda_c = 1,36 - 0,136(\theta_c / 100) + 0,0057(\theta_c / 100)^2 \text{ [W/(m.K)] for } \theta_c \geq 160 \text{ }^\circ\text{C} \quad (5.10)$$

where

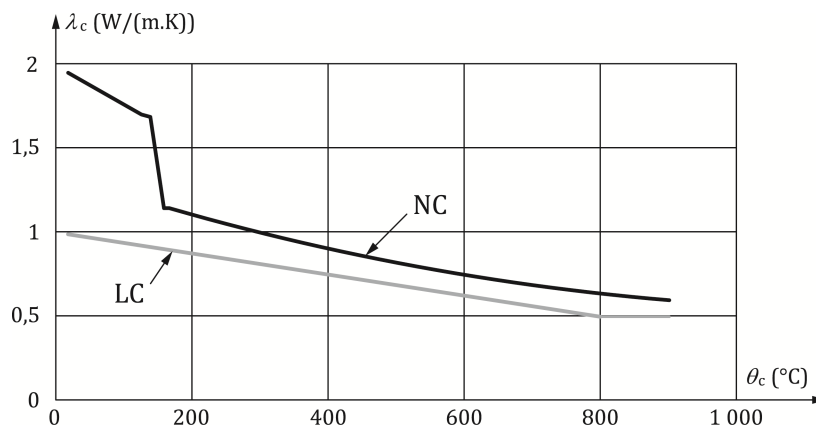
θ_c is the concrete temperature [$^\circ\text{C}$].

(2) The thermal conductivity λ_c of lightweight concrete should be determined from the following:

$$\lambda_c = 1,0 - (\theta_c / 1600) \text{ [W/(m.K)] for } 20 \text{ }^\circ\text{C} \leq \theta_c \leq 800 \text{ }^\circ\text{C} \quad (5.11)$$

$$\lambda_c = 0,5 \text{ [W/(m.K)] for } \theta_c > 800 \text{ }^\circ\text{C} \quad (5.12)$$

NOTE The variation of the thermal conductivity λ_c with temperature is illustrated in Figure 5.3.



Key

- 1 LC
- 2 NC

Figure 5.3 — Thermal conductivity of normal weight concrete (NC) and lightweight concrete (LC) as a function of temperature

5.2.2.3 Specific heat

(1) The specific heat c_c of normal and lightweight aggregate dry concrete (moisture content $u = 0 \%$) should be taken as:

$$c_c = 900 \text{ [J/(kg.K)] for } 20 \text{ }^\circ\text{C} \leq \theta_c \leq 100 \text{ }^\circ\text{C} \quad (5.13)$$

$$c_c = 900 + (\theta_c - 100) \text{ [J/(kg.K)] for } 100 \text{ }^\circ\text{C} < \theta_c \leq 200 \text{ }^\circ\text{C} \quad (5.14)$$

$$c_c = 1000 + (\theta_c - 200) / 2 \text{ [J/(kg.K)] for } 200 \text{ }^\circ\text{C} < \theta_c \leq 400 \text{ }^\circ\text{C} \quad (5.15)$$

$$c_c = 1100 \text{ [J/(kg.K)] for } 400 \text{ }^\circ\text{C} < \theta_c \leq 1200 \text{ }^\circ\text{C} \quad (5.16)$$

where

θ_c is the concrete temperature [$^\circ\text{C}$].

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

(3) Where the moisture content is not considered explicitly in the calculation method, the function given for the specific heat of concrete may be calculated by a constant value c_c^* between 100 °C and 115 °C and a linear relationship between 115 °C and 200 °C.

$$c_c^* = 900 \text{ for a moisture content of 0 \% of concrete weight and [J/(kg.K)]} \quad (5.17)$$

$$c_c^* = 1\,470 \text{ for a moisture content of 1,5 \% of concrete weight and [J/(kg.K)]} \quad (5.18)$$

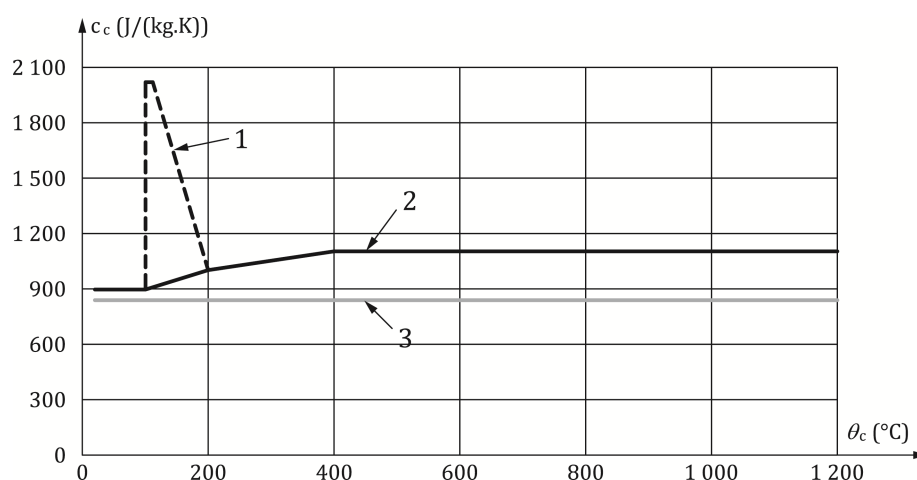
$$c_c^* = 2\,020 \text{ for a moisture content of 3 \% of concrete weight and [J/(kg.K)]} \quad (5.19)$$

$$c_c^* = 5\,600 \text{ for a moisture content of 10 \% of concrete weight and [J/(kg.K)]} \quad (5.20)$$

(4) A linear interpolation may be taken for moisture content within the ranges in (5). Moisture contents up to 10 % may occur for hollow sections filled with concrete.

(5) The moisture content of lightweight 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.

NOTE The variation of the specific heat with temperature is illustrated in Figure 5.4.



Key

- 1 NC with $u = 3 \%$
- 2 NC with $u = 0 \%$
- 3 LC

Figure 5.4 — Specific heat of normal weight concrete (NC) and lightweight concrete (LC) as a function of temperature

5.2.2.4 Density

(1) The variation of concrete density with temperature is influenced by water loss and should be taken as follows:

$$\rho(\theta_c) = \rho(20^\circ\text{C}) \text{ (kg/m}^3\text{) for } 20^\circ\text{C} \leq \theta_c \leq 115^\circ\text{C} \quad (5.21)$$

$$\rho(\theta_c) = \rho(20^\circ\text{C}) (1 - 0.02(\theta_c - 115)/85) \text{ (kg/m}^3\text{) for } 115^\circ\text{C} < \theta_c \leq 200^\circ\text{C} \quad (5.22)$$

$$\rho(\theta_c) = \rho(20^\circ\text{C}) (0.98 - 0.03(\theta_c - 200) / 200) \text{ (kg/m}^3\text{) for } 200^\circ\text{C} < \theta_c \leq 400^\circ\text{C} \quad (5.23)$$

$$\rho(\theta_c) = \rho(20^\circ\text{C}) (0.95 - 0.07(\theta_c - 400) / 800) \text{ (kg/m}^3\text{) for } 400^\circ\text{C} < \theta_c \leq 1\,200^\circ\text{C} \quad (5.24)$$

(2) The density of lightweight aggregate concrete for structural design shall be in the range of:

$$\rho_{c,LC} = 1\,200 \text{ to } 2\,000 \text{ [kg/m}^3\text{]} \quad (5.25)$$

5.2.3 Fire protection materials

(1) The properties and performance of fire protection materials used in design should have been assessed using test procedures given in the relevant parts of EN 13381, as appropriate.

5.3 Mechanical properties

5.3.1 Carbon steel

5.3.1.1 Thermal expansion

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

$$\Delta l / l = -2,416 \cdot 10^{-4} + 1,2 \cdot 10^{-5} \theta_a + 0,4 \cdot 10^{-8} \theta_a^2 \text{ for } 20^\circ\text{C} < \theta_a \leq 750^\circ\text{C} \quad (5.26)$$

$$\Delta l / l = 11 \cdot 10^{-3} \text{ for } 750^\circ\text{C} < \theta_a \leq 860^\circ\text{C} \quad (5.27)$$

$$\Delta l / l = -6,2 \cdot 10^{-3} + 2 \cdot 10^{-5} \theta_a \text{ for } 860^\circ\text{C} < \theta_a \leq 1\,200^\circ\text{C} \quad (5.28)$$

where

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

Δl is the temperature induced expansion of the steel member

θ_a is the steel temperature [$^\circ\text{C}$].

NOTE The variation of the thermal expansion with temperature is illustrated in Figure 5.5.

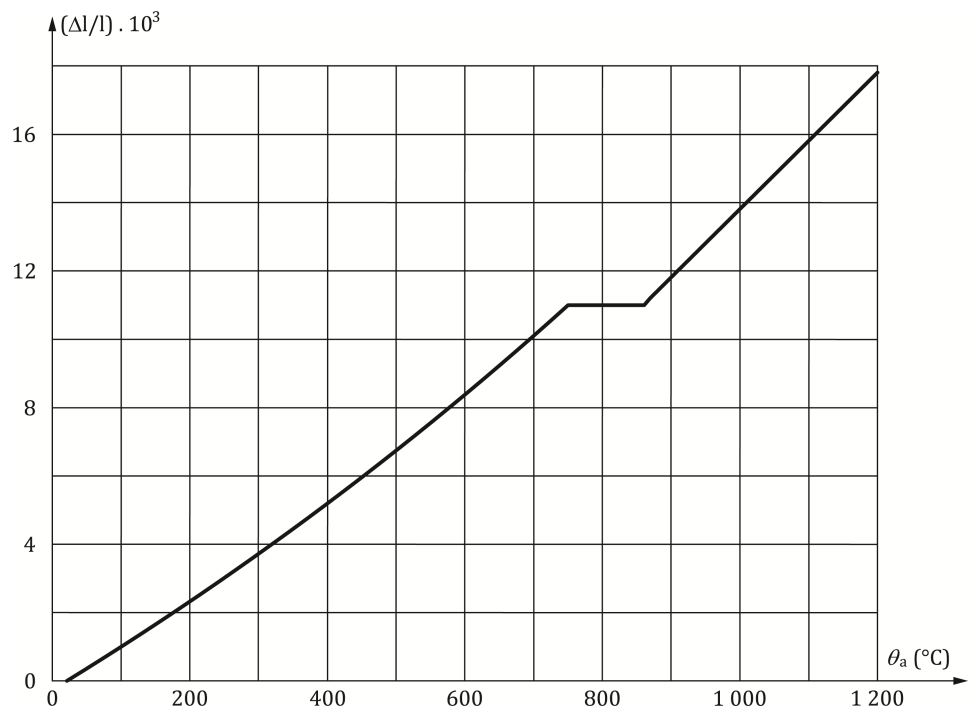
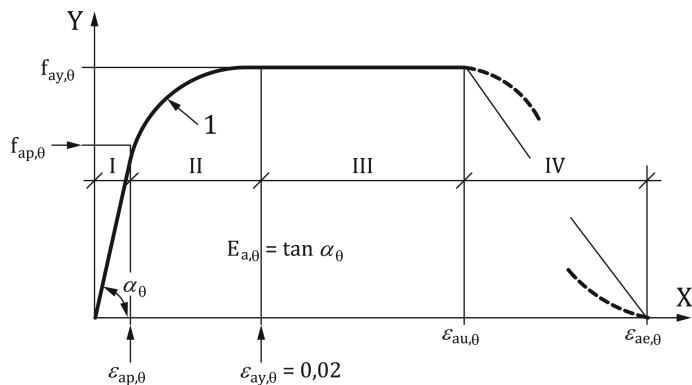


Figure 5.5 — Thermal expansion of steel as a function of temperature

5.3.1.2 Strength and stiffness properties of structural steel

(1) For heating rates between 2 and 50 K/min, the strength and stiffness properties of structural steel at elevated temperatures should be obtained from the stress-strain relationship given in Figure 5.6 and Table 5.2.

- NOTE The stress-strain relationships given in Figure 5.6 and Table 5.2 are defined by three parameters:
- slope of the linear elastic range of the stress-strain relationship of structural steel at elevated temperatures $E_{a,\theta}$;
 - the proportional limit $f_{ap,\theta}$;
 - the maximum stress level or effective yield strength $f_{ay,\theta}$.



Key
1 elliptic branch

Figure 5.6 — Mathematical model for stress-strain relationships of structural steel at elevated temperatures

Table 5.2 — Relation between the various parameters of the mathematical model of Figure 5.6

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

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

(3) The reduction factors k_θ for the stress-strain relationship of steel at elevated steel temperatures given in Table 5.3 shall be applied to the appropriate value E_a or f_{ay} in order to determine the parameters in (1). For intermediate values of the temperature, linear interpolation may be used.

(4) Alternatively, for temperatures below 400 °C, the stress-strain relationships specified in (1) may be extended by allowing for strain hardening, provided local or member buckling does not lead to premature collapse. Where strain hardening is allowed for, use Annex A (normative).

NOTE Values for $\varepsilon_{au,\theta}$ and $\varepsilon_{ae,\theta}$ defining the range of the maximum stress branches and decreasing branches according to Figure 5.6, are given in normative Annex A.

(5) In case of thermal actions according to FprEN 1991-1-2:2023, 5.3, (physically based models), particularly when taking into account the decreasing temperature branch, the values specified in Table 5.3 for the stress-strain relationships of structural steel up to and including S500 may be used as a sufficiently precise approximation. For intermediate values of the steel temperature, linear interpolation may be used.

Table 5.3 — Reduction factors k_θ for stress-strain relationships of structural steel

Steel Temperature θ_a [°C]	$k_{E,\theta} = \frac{E_{a,\theta}}{E_a}$	$k_{p,\theta} = \frac{f_{ap,\theta}}{f_{ay}}$	$k_{y,\theta} = \frac{f_{ay,\theta}}{f_{ay}}$	
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	

5.3.1.3 Strength and stiffness properties of reinforcing steel

- (1) The strength and stiffness properties of reinforcing steels at elevated temperatures may be obtained by the same mathematical model as that presented in 5.3.1.2 for structural steel.
- (2) The three main parameters for hot rolled and cold worked reinforcing steel given in Table 5.4 should be used. For intermediate values of the temperature, linear interpolation may be used.
- (3) The strength and stiffness properties of prestressing steels given in prEN 1992-1-2:2023, 5.3.3.1 should be used.

Table 5.4 — Reduction factors k_θ for stress-strain relationships of hot rolled and cold worked reinforcing steel at elevated temperature

Steel Temperature θ_s [°C]	$k_{y,\theta} = \frac{f_{ay,\theta}}{f_{ay}}$		$k_{p,\theta} = \frac{f_{ap,\theta}}{f_{ay}}$		$k_{E,\theta} = \frac{E_{s,\theta}}{E_s}$	
	hot-rolled	cold-worked	hot-rolled	cold-worked	hot-rolled	cold-worked
1	2	3	4	5	6	7
20	1,00	1,00	1,00	1,00	1,00	1,00
100	1,00	1,00	1,00	0,96	1,00	1,00
200	1,00	1,00	0,81	0,92	0,90	0,87
300	1,00	1,00	0,61	0,81	0,80	0,72
400	1,00	0,94	0,42	0,63	0,70	0,56
500	0,78	0,67	0,36	0,44	0,60	0,40
600	0,47	0,40	0,18	0,26	0,31	0,24
700	0,23	0,12	0,07	0,08	0,13	0,08
800	0,11	0,11	0,05	0,06	0,09	0,06
900	0,06	0,08	0,04	0,05	0,07	0,05
1000	0,04	0,05	0,02	0,03	0,04	0,03
1100	0,02	0,03	0,01	0,02	0,02	0,02
1200	0	0	0	0	0	0

(4) For the limits of the decreasing branch:

— $\varepsilon_{au,\theta} = 0,05$ and $\varepsilon_{ae,\theta} = 0,10$ for Class A reinforcement

— $\varepsilon_{au,\theta} = 0,15$ and $\varepsilon_{ae,\theta} = 0,20$ for Class B and Class C reinforcement

(5) In case of thermal actions according to FprEN 1991-1-2:2023, 5.3 (physically based fire models), particularly when taking into account the decreasing temperature branch, the values specified in Table 5.3 for the stress-strain relationships of structural steel and Table 5.4 for the stress-strain relationships of reinforcing steel, may be used as a sufficiently precise approximation.

5.3.2 Concrete

5.3.2.1 Thermal expansion

(1) The thermal expansion $\Delta l/l$ of concrete should be determined from the following reference to the length at 20 °C regardless of the concrete strength:

Siliceous aggregates:

$$\Delta l/l = -1,8 \cdot 10^{-4} + 9 \cdot 10^{-6} \theta_c + 2,3 \cdot 10^{-11} \theta_c^3 \quad \text{for } 20 \text{ °C} \leq \theta_c \leq 700 \text{ °C} \quad (5.29)$$

$$\Delta l/l = 14 \cdot 10^{-3} \quad \text{for } 700 \text{ °C} < \theta_c \leq 1\,200 \text{ °C} \quad (5.30)$$

Calcareous aggregates:

$$\Delta l / l = -1,2 \cdot 10^{-4} + 6 \cdot 10^{-6} \theta_c + 1,4 \cdot 10^{-11} \theta_c^3 \quad \text{for } 20 \text{ °C} \leq \theta_c \leq 700 \text{ °C} \quad (5.31)$$

$$\Delta l / l = 12 \cdot 10^{-3} \quad \text{for } 700 \text{ °C} < \theta_c \leq 1\,200 \text{ °C} \quad (5.32)$$

Lightweight aggregate concrete:

$$\Delta l / l = 8 \cdot 10^{-6} (\theta_c - 20) \quad (5.33)$$

where

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

Δl is the temperature induced expansion of the concrete member

θ_c is the concrete temperature

5.3.2.2 Strength and stiffness properties of concrete

(1) The strength and stiffness properties of concrete under uniaxial stresses at elevated temperatures should be obtained from the stress-strain relationships as presented in Figure 5.8.

(2) The stress-strain relationships given in Figure 5.8 should be defined by three parameters:

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

(3) Values of the above parameters should be taken from Table 5.5. as a function of concrete temperature. For intermediate values of the temperature, linear interpolation may be used.

(4) The parameters specified in Table 5.5 should be used for normal weight concrete with siliceous or calcareous (containing at least 80 % calcareous aggregate by weight) aggregates.

Table 5.5 — 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 θ_c	NC	NC	$\varepsilon_{c1,\theta} \cdot 10^3$	$\varepsilon_{cu1,\theta} \cdot 10^3$
[°C]	Siliceous aggregates	Calcareous aggregates	NC	NC
20	1	1	2,5	20,0
100	1	1	4,0	22,5
200	0,95	1	5,5	25,0
300	0,85	1	7,0	27,5
400	0,75	0,88	10,0	30,0
500	0,60	0,76	15,0	32,5
600	0,45	0,64	25,0	35,0
700	0,30	0,52	25,0	37,5
800	0,15	0,40	25,0	40,0
900	0,08	0,28	25,0	42,5
1 000	0,04	0,16	25,0	45,0
1 100	0,01	0,04	25,0	47,5
1 200	0	0	—	—

(5) The values of the $\varepsilon_{cu,\theta}$ and $\varepsilon_{ce,\theta}$ for lightweight aggregate concrete should be based on testing.

(6) For thermal actions in accordance with FprEN 1991-1-2:2023, 5.3 (Physically based models), when taking into account the cooling phase, the strength of concrete heated to a maximum temperature $\theta_{c,max}$ and having cooled down to 20 °C may be taken according to Formula (5.34):

$$f_{c,\theta,20^\circ C} = \phi f_{ck} \quad (5.34)$$

where

$$20^\circ C \leq \theta_{max} < 100^\circ C ; \phi = k_{c,\theta_{max}} \quad (5.35)$$

$$100^\circ C \leq \theta_{max} < 300^\circ C ; \phi = (-0,0005\theta_{max} + 1,05)k_{c,\theta_{max}} \quad (5.36)$$

$$\theta_{max} \geq 300^\circ C ; \phi = 0,9k_{c,\theta_{max}} \quad (5.37)$$

where

f_{ck} is characteristic value of the compressive cylinder strength of concrete at 28 days and at 20 °C.

$k_{c,\theta_{max}}$ is the reduction factor which corresponds to $k_{c,\theta}$ at the maximum temperature $\theta_{c,max}$, it is taken according to Table 5.5.

$\varepsilon_{c1,\theta}$ and $\varepsilon_{cu1,\theta}$ are the values of the strain-stress relationship during the cooling phase, which may be taken as the same as those corresponding to the maximum temperature $\theta_{c,max}$.