



BSI Standards Publication

Eurocode 2: Design of concrete structures

Part 1-2: General rules - Structural fire design

National foreword

This British Standard is the UK implementation of EN 1992-1-2:2004+A1:2019, incorporating corrigendum July 2008. It supersedes BS EN 1992-1-2:2004, which is withdrawn.

The start and finish of text introduced or altered by corrigendum is indicated in the text by tags. Text altered by CEN corrigendum July 2008 is indicated in the text by AC1 AC1.

The start and finish of text introduced or altered by amendment is indicated in the text by tags. Tags indicating changes to CEN text carry the number of the CEN amendment. For example, text altered by CEN amendment A1 is indicated by A1 A1.

The structural Eurocodes are divided into packages by grouping Eurocodes for each of the main materials, concrete, steel, composite concrete and steel, timber, masonry and aluminium, this is to enable a common date of withdrawal (DOW) for all the relevant parts that are needed for a particular design. The conflicting national standards will be withdrawn at the end of the coexistence period, after all the EN Eurocodes of a package are available.

Following publication of the EN, there is a period allowed for national calibration period during which the National Annex is issued, followed by a coexistence period of a maximum of three years. During the coexistence period Member States will be encouraged to adapt their national provisions.

At the end of this co-existence period, the conflicting parts of national standards will be withdrawn. In this case, there are no corresponding national standards.

The UK participation in its preparation was entrusted to Technical Committee B/525/2, Structural use of concrete.

A list of organizations represented on this committee can be obtained on request to its secretary.

Where a normative part of this EN allows for a choice to be made at the national level, the range and possible choice will be given in the normative text, and a note will qualify it as a Nationally Determined Parameter (NDP). NDPs can be a specific value for a factor, a specific level or class, a particular method or a particular application rule if several are proposed in the EN.

To enable EN 1992-1-2 to be used in the UK, the NDPs will be published in a National Annex, which will be made available by BSI in due course, after public consultation has taken place.

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

ISBN 978 0 580 98289 7

ICS 91.010.30; 13.220.50; 91.080.40

Compliance with a British Standard cannot confer immunity from legal obligations.

This British Standard was published under the authority of the Standards Policy and Strategy Committee on 9 February 2005.

Amendments/corrigenda issued since publication

Date	Text affected
28 February 2010	Implementation of CEN corrigendum July 2008
31 August 2019	Implementation of CEN amendment A1:2019

English version

**Eurocode 2: Design of concrete structures - Part 1-2: General
rules - Structural fire design**

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

Eurocode 2: Planung von Stahlbeton- und
Spannbetontragwerken - Teil 1-2: Allgemeine Regeln -
Tragwerksbemessung für den Brandfall

This European Standard was approved by CEN on 8 July 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.

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

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|---|----------------------------------|
| A Temperature profiles | B Simplified calculation methods |
| C Buckling of columns under fire conditions | D Calculation methods for shear, |
| E Simplified calculation method for beams and slabs | torsion and anchorage |

Foreword

This European Standard EN 1992-1-2, "Design of 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 June 2005, and conflicting National Standards shall be withdrawn at latest by March 2010.

This European standard supersedes ENV 1992-1-2: 1995.

According to the CEN-CENELEC Internal Regulations, the National Standard Organisations of the following countries are bound to implement these 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.

Foreword to amendment A1

This document (EN 1992-1-2:2004/A1:2019) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI.

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 November 2019, and conflicting national standards shall be withdrawn at the latest by November 2019.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN shall not be held responsible for identifying any or all such patent rights.

This document has been prepared under a mandate given to CEN by the European Commission and the European Free Trade Association.

According to the CEN-CENELEC Internal Regulations, the national standards organisations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, Former Yugoslav Republic of Macedonia, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Serbia, Slovakia, Slovenia, Spain, Sweden, Switzerland, Turkey and the 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 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ 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:

EN 1990	Eurocode:	Basis of Structural 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

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 Documents² referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards³. Therefore, technical aspects

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).

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 harmonised ENs and ETAGs/ETAs.

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

- 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 ;
 - 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. ;
 - 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.

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.

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,
- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works⁴. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1992-1-2

EN 1992- 1-2 describes the Principles, requirements and rules for the structural design of buildings exposed to fire, including the following aspects.

Safety requirements

EN 1992-1-2 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.

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 1.

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

"The construction works must be designed and build 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".

According to the Interpretative Document N° 2 "Safety in case of fire" 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 fire regulations or by referring to fire safety engineering for assessing passive and active measures, see EN 1991-1-2.

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 and passive 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 call for specific periods of fire resistance, takes into account (though not explicitly), the features and uncertainties described above.

Application of design procedures is illustrated in Figure 0.1. 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. Additional information for alternative methods in this standard is given in Table 0.1.

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

Design aids

Where simple calculation models are not available, the Eurocode fire parts give 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 1992-1-2, will be prepared by interested external organisations.

The main text of EN 1992-1-2, together with informative Annexes A, B, C, D and E, includes most of the principal concepts and rules necessary for structural fire design of concrete structures.

National Annex for EN 1992-1-2

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 1992-1-2 should have a National Annex containing the Eurocode all Nationally Determined Parameters to be used for the design of buildings, and where required and applicable, for civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1992-1-2 through clauses:

- | | |
|-------------|---------------|
| - 2.1.3 (2) | - 5.3.2 (2) |
| - 2.3 (2)P | - 5.6.1 (1) |
| - 3.2.3 (5) | - 5.7.3 (2) |
| - 3.2.4 (2) | - 6.1 (5) |
| - 3.3.3 (1) | - 6.2 (2) |
| - 4.1 (1)P | - 6.3.1 (1) |
| - 4.5.1 (2) | - 6.4.2.1 (3) |
| - 5.2 (3) | - 6.4.2.2 (2) |

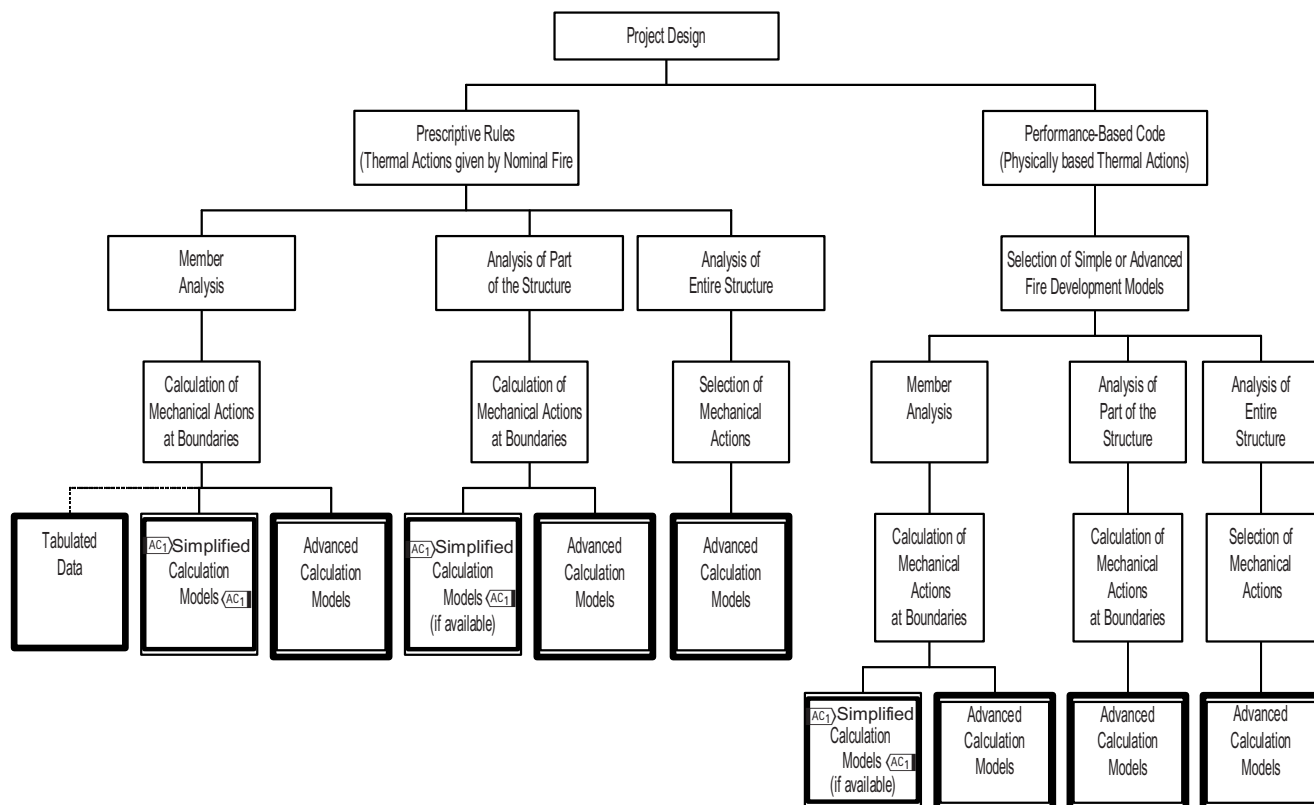


Figure 1 : Alternative design procedures

Table 0.1 Summary table showing alternative methods of verification for fire resistance

	Tabulated data	Simplified calculation methods	Advanced calculation models
Member analysis The member is considered as isolated. Indirect fire actions are not considered, except those resulting from thermal gradients	YES - Data given for standard fire only, 5..1(1) - In principle data could be developed for other fire curves	YES - standard fire and parametric fire, 4.2.1(1) - temperature profiles given for standard fire only, 4.2.2(1) - material models apply only to heating rates similar to standard fire, 4.2.4.1(2)	YES , 4.3.1(1)P Only the principles are given
AC1) Analysis of part of the structure Indirect fire actions within the subassembly are considered, AC1 but no time-dependent interaction with other parts of the structure.	NO	YES - standard fire and parametric fire, 4.2.1(1) - temperature profiles given for standard fire only, 4.2.2(1) - material models apply only to heating rates similar to standard fire, 4.2.4.1(2)	YES 4.3.1(1)P Only the principles are given
Global structural analysis Analysis of the entire structure. Indirect fire actions are considered throughout the structure	NO	NO	YES 4.3.1(1)P Only the principles are given

SECTION 1 GENERAL

1.1 Scope

1.1.1 Scope of Eurocode 2

(1)P Eurocode 2 applies to the design of buildings and civil engineering works in concrete. 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 design.

(2)P Eurocode 2 is only concerned with requirements for resistance, serviceability, durability and fire resistance concrete structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3)P Eurocode 2 is intended to be used in conjunction with:

- EN 1990 “Basis of structural design”
- EN 1991 “Actions on structures”
- hEN’s for construction products relevant for concrete structures
- ENV 13670-1 “Execution of concrete structures . Part 1: Common rules”
- EN 1998 “Design of structures for earthquake resistance”, when concrete structures are built in seismic regions

(4)P Eurocode 2 is subdivided in various parts:

- Part 1-1: General rules and rules for buildings
- Part 1-2: General rules – Structural fire design
- Part 2: Concrete bridges
- Part 3: Liquid retaining and containment structures

1.1.2 Scope of Part 1-2 of Eurocode 2

(1)P This Part 1-2 of EN 1992 deals with the design of concrete structures for the accidental situation of fire exposure and is intended to be used in conjunction with EN 1992-1-1 and EN 1991-1-2. This part 1-2 only identifies differences from, or supplements to, normal temperature design.

(2)P This Part 1-2 of EN 1992 deals only with passive methods of fire protection. Active methods are not covered.

(3)P This Part 1-2 of EN 1992 applies to 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)P This Part 1-2 of EN 1992 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)P This Part 1-2 of EN 1992 applies to structures, or parts of structures, that are within the scope of EN 1992-1-1 and are designed accordingly. However, it does not cover:

- structures with prestressing by external tendons
- shell structures

(6)P The methods given in this Part 1-2 of EN 1992 are applicable to normal weight concrete up to strength class C90/105 and for lightweight concrete up to strength class LC55/60. Additional and alternative rules for strength classes above C50/60 are given in section 6.

1.2 Normative references

The following normative documents contain provisions that, through reference in this text, constitute provisions of this European Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this European Standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references, the latest edition of the normative document referred to applies.

EN 1363-2: Fire resistance tests – Part 2: Alternatives and additional procedures;

EN 1990: Eurocode: Basis of structural design;

EN 1991-1-2: Eurocode 1 - Actions on structures - Part 1-2: General actions - 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

EN 10080: Steel for the reinforcement of concrete - Weldable reinforcing steel - General

EN 10138-2: Prestressing steels - Part 2: Wire

EN 10138-3: Prestressing steels - Part 3: Strand

EN 10138-4: Prestressing steels - Part 4: Bar

1.3 Assumptions

AC1 The general assumptions given in EN 1990 and EN 1992-1-1 apply. **AC1**

1.4 Distinction between principles and application rules

(1) The rules given in EN 1990 apply.

1.5 Definitions

For the purposes of this Part 1-2 of EN 1992, the definitions of EN 1990 and of EN 1991-1-2 apply with the additional definitions:

1.5.1 Critical temperature of reinforcement: The temperature of reinforcement at which failure of the member in fire situation (*Criterion R*) is expected to occur at a given steel stress level.

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

1.5.3 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.

1.5.4 Part of structure: isolated part of an entire structure with appropriate support and boundary conditions.

1.5.5 Protective layers: Any material or combination of materials applied to a structural member for the purpose of increasing its fire resistance.

1.5.6 Reduced cross section: Cross section of the member in structure fire design used in the reduced cross section method. $\boxed{AC_1}$ It is obtained by removing parts of the $\boxed{AC_1}$ cross section with assumed zero strength and stiffness.

1.6 Symbols

1.6.1 Supplementary symbols to EN1992-1-1

(1)P The following supplementary symbols are used:

Latin upper case letters

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

E_d design effect of actions for normal temperature design

$R_{d,fi}$ design resistance in the fire situation; $R_{d,fi}(t)$ at a given time t .

R 30 or R 60,... fire resistance class for the load-bearing criterion for 30, or 60... minutes in standard fire exposure

E 30 or E 60,... fire resistance class for the integrity criterion for 30, or 60... minutes in standard fire exposure

I 30 or I 60,... fire resistance class for the insulation criterion for 30, or 60... minutes in standard fire exposure

T temperature [K] (cf θ temperature [$^{\circ}\text{C}$]);

X_k characteristic value of a strength or deformation property for normal temperature design

$X_{d,fi}$ design strength or deformation property in the fire situation

Latin lower case letters

a axis distance of reinforcing or prestressing steel from the nearest exposed surface

c_c specific heat of concrete [J/kgK]

$f_{ck}(\theta)$ characteristic value of compressive strength of concrete at temperature θ for a specified strain

$f_{ct,t}(\theta)$ characteristic value of tensile strength of concrete at temperature θ for a specified strain

$f_{pk}(\theta)$ characteristic value of strength of prestressing steel at temperature θ for a specified strain

$f_{sk}(\theta)$ characteristic strength of reinforcing steel at temperature θ for a specified strain

$k(\theta) = X_k(\theta)/X_k$ reduction factor for a strength or deformation property dependent on the material temperature θ

$n = N_{0Ed,fi} / (0,7(A_c f_{cd} + A_s f_{yd}))$ load level of a column at normal temperature conditions

$\langle AC_1 \rangle t$ time in fire exposure (min) $\langle AC_1 \rangle$

Greek lower case letters

$\gamma_{M,fi}$ partial safety factor for a material in fire design

$\eta_{fi} = E_{d,fi}/E_d$ reduction factor for design load level in the fire situation

$\mu_{fi} = N_{Ed,fi}/N_{Rd}$ degree of utilisation in fire situation

$\varepsilon_c(\theta)$ thermal strain of concrete

$\varepsilon_p(\theta)$ thermal strain of prestressing steel

$\varepsilon_s(\theta)$ thermal strain of reinforcing steel

$\varepsilon_{s,fi}$ strain of the reinforcing or prestressing steel at temperature θ

λ_c thermal conductivity of concrete [W/mK]

$\lambda_{0,fi}$ slenderness of the column under fire conditions

$\sigma_{c,fi}$ compressive stress of concrete in fire situation

$\sigma_{s,fi}$ steel stress in fire situation

θ temperature [$^{\circ}\text{C}$]

θ_{cr} critical temperature [$^{\circ}\text{C}$]

1.6.2 Supplementary to EN 1992-1-1, the following subscripts are used:

fi value relevant for the fire situation

t dependent on the time

θ dependent on the temperature

SECTION 2 BASIS OF DESIGN

2.1 Requirements

2.1.1 General

(1)P Where mechanical resistance in the case of fire is required, concrete structures shall be designed and constructed in such a way that they maintain their load bearing function AC1 during the required time of fire exposure. AC1

(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 AC1 during the required time of fire exposure. AC1 This shall ensure, where relevant, that:

- integrity failure does not occur, see EN 1991-1-2
- insulation failure does not occur, see EN 1991-1-2
- thermal radiation from the unexposed side is limited.

Note 1: See EN 1991-1-2 for the definitions.

Note 2: For concrete structures considered in this Part 1-2 thermal radiation criteria are not relevant.

(3)P Deformation criteria shall be applied where the means of protection, or the design criteria for separating elements, 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 4.7,
- the separating elements have to fulfil requirements according to 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) AC1 With the external fire exposure curve (see EN 1991-1-2) the same criteria (R, E, I) should apply, however the reference to this specific curve should be identified by the letters "ef". AC1

(5) AC1 With the hydrocarbon fire exposure curve (see EN 1991-1-2) the same criteria (R, E, I) should apply, however the reference to this specific curve should be identified by the letters "HC". AC1

(6) Where a vertical separating element with or without load-bearing function has to comply with impact resistance requirement (criterion M), the element should resist a horizontal concentrated load as specified in EN 1363 Part 2.

2.1.3 Parametric fire exposure

AC1 (1)P The load-bearing function shall **AC1** be maintained during the complete endurance of the fire including the decay phase, or a specified period of time.

(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 gas temperature in the fire compartment is reached;
- the average temperature rise of the unexposed side of the construction should be limited to $\Delta\theta_1$ and the maximum temperature rise of the unexposed side should not exceed $\Delta\theta_2$ during the decay phase.

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 EN 1991-1-2, the emissivity related to the concrete surface should be taken as 0,7.

2.3 Design values of material properties

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

$$X_{d,fi} = k_{\theta} X_k / \gamma_{M,fi} \quad (2.1)$$

where:

- X_k is the characteristic value of a strength or deformation property (*generally f_k or E_k*) for normal temperature design to EN 1992-1-1;
- 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 safety factor for the relevant material property, for the fire situation.

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

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

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

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

$$X_{d,fi} = \gamma_{M,fi} X_{k,\theta} \quad (2.2b)$$

where:

$X_{k,0}$ is the value of a material property in fire design, generally dependent on the material temperature, see section 3;

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

Note 1: The value of $\gamma_{M,fi}$ for use in a Country may be found in its National Annex. The recommended value is:
For thermal properties of concrete and reinforcing and prestressing steel: $\gamma_{M,fi} = 1,0$
For mechanical properties of concrete and reinforcing and prestressing steel: $\gamma_{M,fi} = 1,0$

Note 2: If the recommended values are modified, the tabulated data may require modification.

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 1992 shall reflect the expected performance of the structure in fire.

(2)P AC1 It shall be verified for the specified duration of fire exposure t : AC1

$$E_{d,fi} \leq R_{d,t,fi} \quad (2.3)$$

where

$E_{d,fi}$ 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

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

(3) The structural analysis for the fire situation should be carried out according to Section 5 of EN 1990.

Note: For verifying standard fire resistance requirements, 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 section 5 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, Section 5.

2.4.2 Member analysis

(1) The effect of actions should be determined for time $t = 0$ using combination factors $\psi_{1,1}$ or $\psi_{1,2}$ according to EN 1991-1-2 Section 4.

(2) As a simplification to (1) the effects of actions may be obtained from a structural analysis for normal temperature design as:

$$E_{d,fi} = \eta_{fi} E_d \quad (2.4)$$

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 for the design load level for the fire situation.

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

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

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

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

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

where

- $Q_{k,1}$ is the principal variable load;
- G_k is the characteristic value of a permanent action;
- γ_G is the partial factor for a permanent action;
- $\gamma_{Q,1}$ is the partial factor for variable action 1;
- ψ_{fi} is the combination factor for frequent or quasi-permanent values given either by $\psi_{1,1}$ or $\psi_{2,1}$, see EN1991-1-2
- ξ is a reduction factor for unfavourable permanent action G

Note 1: Regarding equation (2.5), examples of the variation of the reduction factor η_{fi} versus the load ratio $Q_{k,1}/G_k$ for Expression (2.4) and different values of the combination factor $\psi_{1,1}$ are shown in Figure 2.1 with the following assumptions: $\gamma_{GA} = 1,0$, $\gamma_G = 1,35$ and $\gamma_Q = 1,5$. Expressions (2.5a) and (2.5b) give slightly higher values. Recommended values of partial factors are given in the relevant National Annexes of EN 1990.

Note 2: As a simplification a recommended value of $\eta_{fi} = 0,7$ may be used.

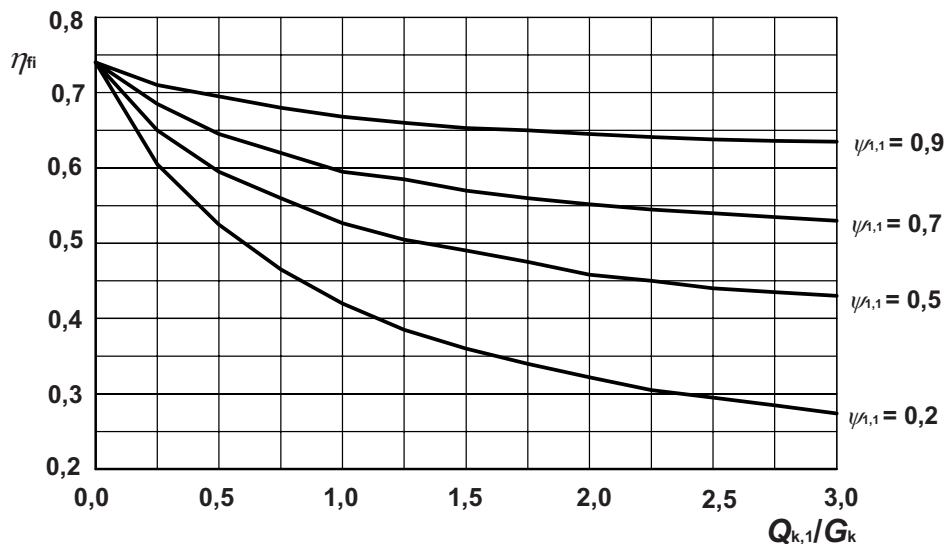


Figure 2.1: Variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$

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

(5) The boundary conditions at supports and ends of member, applicable at time $t = 0$, are assumed to remain unchanged throughout the fire exposure.

(6) Tabulated data, simplified or general calculation methods given in 5, 4.2 and 4.3 respectively are suitable for verifying members under fire conditions.

2.4.3 Analysis of part of the structure

(1) 2.4.2 (1) applies.

(2) As an alternative to carrying out a global structural analysis for the fire situation at time $t = 0$ the reactions at supports and internal forces and moments at boundaries of part of the structure may be obtained from 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 stiffnesses, 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, applicable at time $t = 0$, are assumed to remain unchanged throughout the fire exposure

2.4.4 Global structural analysis

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

SECTION 3 MATERIAL PROPERTIES

3.1 General

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

(2) The values may be used with the simplified (see 4.2) and the advanced calculation method (see 4.3).

Alternative formulations of material laws may be applied, provided the solutions are within the range of experimental evidence.

Note: Material properties for lightweight aggregate concrete are not given in this Eurocode.

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

3.2 Strength and deformation properties at elevated temperatures

3.2.1 General

(1)P Numerical values on strength and deformation properties given in this section are based on steady state as well as transient state tests and sometimes a combination of both. As creep effects are not explicitly considered, the material models in this Eurocode are applicable for heating rates between 2 and 50 K/min. For heating rates outside the above range, the reliability of the strength and deformation properties shall be demonstrated explicitly.

3.2.2 Concrete

3.2.2.1 Concrete under compression

(1)P The strength and deformation properties of uniaxially stressed concrete at elevated temperatures shall be obtained from the stress-strain relationships as presented in Figure 3.1.

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

- the compressive strength $f_{c,\theta}$
- the strain $\varepsilon_{c1,\theta}$ corresponding to $f_{c,\theta}$.

(3) Values for each of these parameters are given in Table 3.1 as a function of concrete temperatures. For intermediate values of the temperature, linear interpolation may be used.

(4) The parameters specified in Table 3.1 may be used for normal weight concrete with siliceous or calcareous (containing at least 80% calcareous aggregate by weight) aggregates.

(5) Values for $\varepsilon_{cu1,\theta}$ defining the range of the descending branch may be taken from Table 3.1, Column 4 for normal weight concrete with siliceous aggregates, Column 7 for normal weight concrete with calcareous aggregates.

Table 3.1: Values for the main parameters of the stress-strain relationships of normal weight concrete with siliceous or calcareous aggregates concrete at elevated temperatures.

Concrete temp. θ [°C]	Siliceous aggregates			Calcareous aggregates		
	$f_{c,\theta} / f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$	$f_{c,\theta} / f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$
1	2	3	4	5	6	7
20	1,00	0,0025	0,0200	1,00	0,0025	0,0200
100	1,00	0,0040	0,0225	1,00	0,0040	0,0225
200	0,95	0,0055	0,0250	0,97	0,0055	0,0250
300	0,85	0,0070	0,0275	0,91	0,0070	0,0275
400	0,75	0,0100	0,0300	0,85	0,0100	0,0300
500	0,60	0,0150	0,0325	0,74	0,0150	0,0325
600	0,45	0,0250	0,0350	0,60	0,0250	0,0350
700	0,30	0,0250	0,0375	0,43	0,0250	0,0375
800	0,15	0,0250	0,0400	0,27	0,0250	0,0400
900	0,08	0,0250	0,0425	0,15	0,0250	0,0425
1000	0,04	0,0250	0,0450	0,06	0,0250	0,0450
1100	0,01	0,0250	0,0475	0,02	0,0250	0,0475
1200	0,00	-	-	0,00	-	-

(6) For thermal actions in accordance with EN 1991-1-2 Section 3 (natural fire simulation), particularly when considering the descending temperature branch, the mathematical model for stress-strain relationships of concrete specified in Figure 3.1 should be modified.

(7) Possible strength gain of concrete in the cooling phase should not be taken into account.

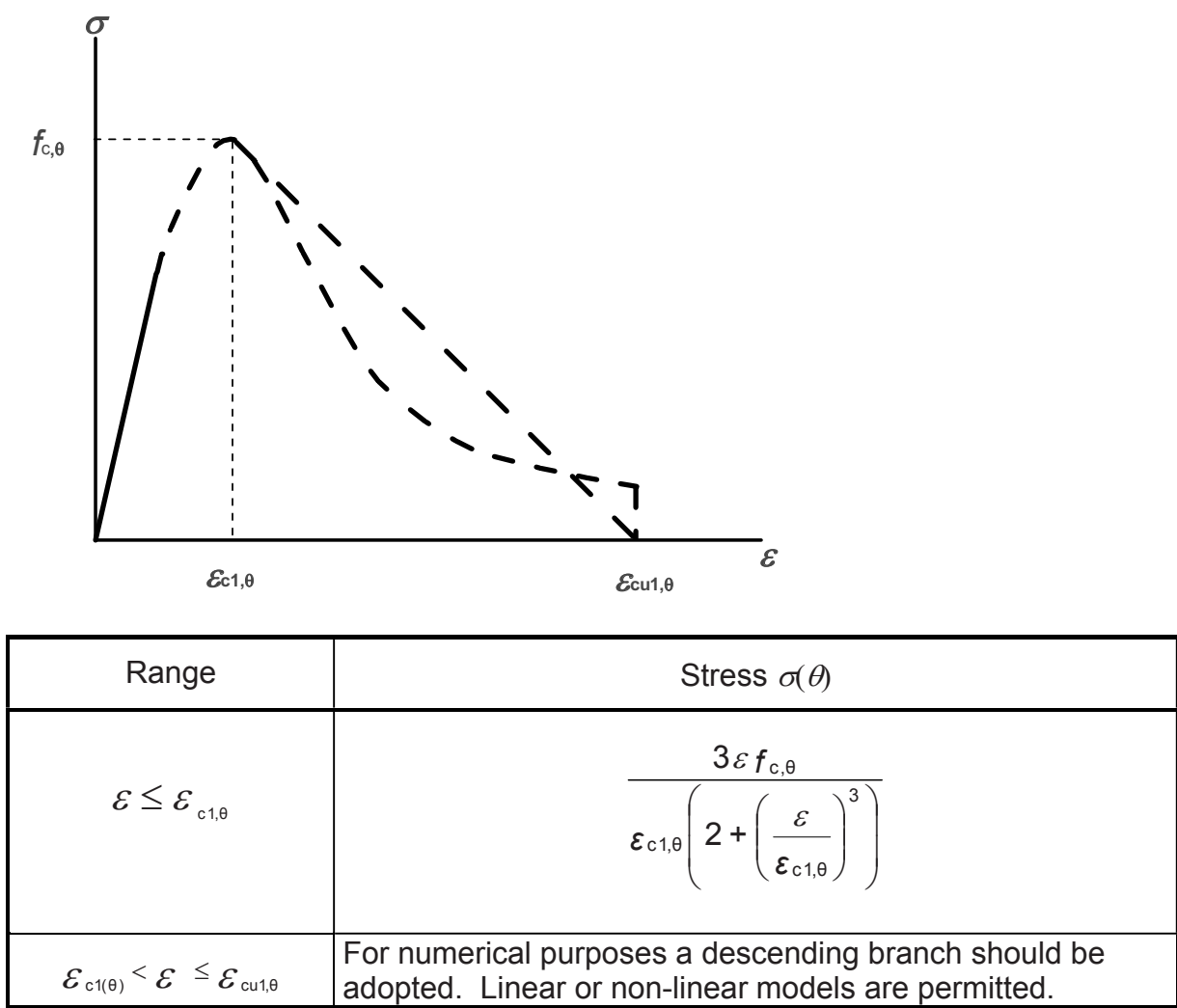


Figure 3.1: Mathematical model for stress-strain relationships of concrete under compression at elevated temperatures.

3.2.2.2 Tensile strength

- (1) The tensile strength of concrete should normally be ignored (conservative). If it is necessary to take account of the tensile strength, when using the simplified or advanced calculation method, this clause may be used.
- (2) The reduction of the characteristic tensile strength of concrete is allowed for by the coefficient $k_{c,t}(\theta)$ as given in Expression (3.1).

$$f_{ck,t}(\theta) = k_{c,t}(\theta) f_{ck,t} \tag{3.1}$$

- (3) In absence of more accurate information the following $k_{c,t}(\theta)$ values should be used (see Figure 3.2):

$$k_{c,t}(\theta) = 1,0 \qquad \text{for } 20\text{ }^{\circ}\text{C} \leq \theta \leq 100\text{ }^{\circ}\text{C}$$

$$k_{c,t}(\theta) = 1,0 - 1,0 (\theta - 100)/500 \qquad \text{for } 100\text{ }^{\circ}\text{C} < \theta \leq 600\text{ }^{\circ}\text{C}$$

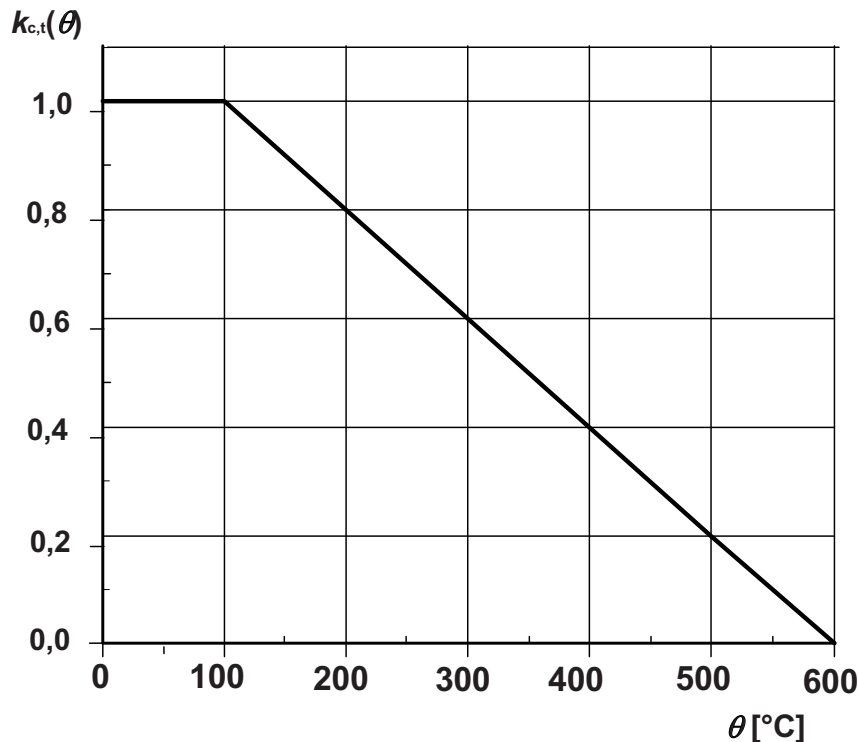


Figure 3.2: Coefficient $k_{c,t}(\theta)$ allowing for decrease of tensile strength ($f_{ck,t}$) of concrete at elevated temperatures

3.2.3 Reinforcing steel

(1)P The strength and deformation properties of reinforcing steel at elevated temperatures shall be obtained from the stress-strain relationships specified in Figure 3.3 and Table 3.2 (a or b). Table 3.2b may only be used if strength at elevated temperatures is tested.

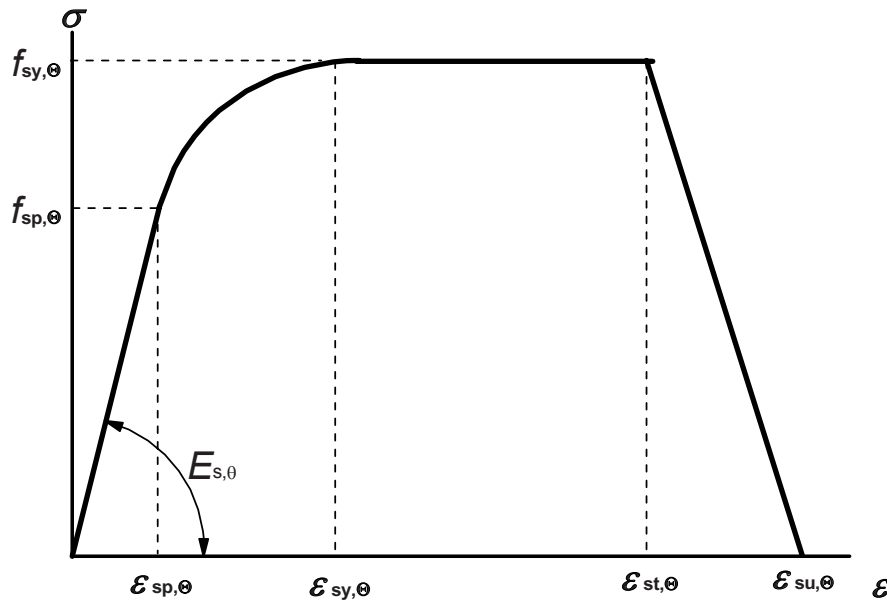
(2) The stress-strain relationships given in Figure 3.3 are defined by three parameters:

- the slope of the linear elastic range $E_{s,\theta}$
- the proportional limit $f_{sp,\theta}$
- the maximum stress level $f_{sy,\theta}$

(3) Values for the parameters in (2) for hot rolled and cold worked reinforcing steel at elevated temperatures are given in Table 3.2. For intermediate values of the temperature, linear interpolation may be used.

(4) The formulation of stress-strain relationships may also be applied for reinforcing steel in compression.

(5) In case of thermal actions according to EN 1991-1-2, Section 3 (natural fire simulation), particularly when considering the descending temperature branch, the values specified in Table 3.2 for the stress-strain relationships of reinforcing steel may be used as a sufficient approximation.



Range	Stress $\sigma(\theta)$	Tangent modulus
$\varepsilon_{sp,\theta}$	$\varepsilon E_{s,\theta}$	$E_{s,\theta}$
$\varepsilon_{sp,\theta} \leq \varepsilon \leq \varepsilon_{sy,\theta}$	$f_{sp,\theta} - c + (b/a)[a^2 - (\varepsilon_{sy,\theta} - \varepsilon)^2]^{0,5}$	$\frac{b(\varepsilon_{sy,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon - \varepsilon_{sy,\theta})^2\right]^{0,5}}$
$\varepsilon_{sy,\theta} \leq \varepsilon \leq \varepsilon_{st,\theta}$	$f_{sy,\theta}$	0
$\varepsilon_{st,\theta} \leq \varepsilon \leq \varepsilon_{su,\theta}$	$f_{sy,\theta} [1 - (\varepsilon - \varepsilon_{st,\theta}) / (\varepsilon_{su,\theta} - \varepsilon_{st,\theta})]$	-
$\varepsilon = \varepsilon_{su,\theta}$	0,00	-
Parameter ^{*)}	$\varepsilon_{sp,\theta} = f_{sp,\theta} / E_{s,\theta} \quad \varepsilon_{sy,\theta} = 0,02$ Class A reinforcement:	$\varepsilon_{st,\theta} = 0,15 \quad \varepsilon_{su,\theta} = 0,20$ $\varepsilon_{st,\theta} = 0,05 \quad \varepsilon_{su,\theta} = 0,10$
Functions	$a^2 = (\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta} + c/E_{s,\theta})$ $b^2 = c (\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta}) E_{s,\theta} + c^2$ $c = \frac{(f_{sy,\theta} - f_{sp,\theta})^2}{(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})E_{s,\theta} - 2(f_{sy,\theta} - f_{sp,\theta})}$	

*) Values for the parameters $\varepsilon_{pt,\theta}$ and $\varepsilon_{pu,\theta}$ for prestressing steel may be taken from Table 3.3. Class A reinforcement is defined in Annex C of EN 1992-1-1.

Figure 3.3: Mathematical model for stress-strain relationships of reinforcing and prestressing steel at elevated temperatures (notations for prestressing steel “p” instead of “s”)

Table 3.2a: Class N values for the parameters of the stress-strain relationship of hot rolled and cold worked reinforcing steel at elevated temperatures

Steel Temperature θ [°C]	$f_{sy,\theta} / f_{yk}$		$f_{sp,\theta} / f_{yk}$		$E_{s,\theta} / E_s$	
	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	0,00	0,00	0,00	0,00	0,00	0,00

Table 3.2b: Class X values for the parameters of the stress-strain relationship of hot rolled and cold worked reinforcing steel at elevated temperatures

Steel Temperature θ [°C]	$f_{sy,\theta} / f_{yk}$	$f_{sp,\theta} / f_{yk}$	$E_{s,\theta} / E_s$
	hot rolled and cold worked	hot rolled and cold worked	hot rolled and cold worked
20	1,00	1,00	1,00
100	1,00	1,00	1,00
200	1,00	0,87	0,95
300	1,00	0,74	0,90
400	0,90	0,70	0,75
500	0,70	0,51	0,60
600	0,47	0,18	0,31
700	0,23	0,07	0,13
800	0,11	0,05	0,09
900	0,06	0,04	0,07
1000	0,04	0,02	0,04
1100	0,02	0,01	0,02

Note: The choice of Class N (Table 3.2a) or X (Table 3.2b) to be used in a Country may be found in its National Annex. Class N is generally recommended. Class X is recommended only when there is experimental evidence for these values.

3.2.4 Prestressing steel

(1) The strength and deformation $\boxed{\text{AC1}}$ properties of prestressing steel at elevated temperatures should be obtained by the same $\boxed{\text{AC1}}$ mathematical model as that presented in 3.2.3 for reinforcing steel.

(2) Values for the parameters for cold worked (wires and strands) and quenched and tempered (bars) prestressing steel at elevated temperatures are given by $f_{py,\theta} / (\beta f_{pk})$, $f_{pp,\theta} / (\beta f_{pk})$, $E_{p,\theta} / E_p$, $\varepsilon_{pt,\theta}$ [-], $\varepsilon_{pu,\theta}$ [-]. The value of β is given by the choice of Class A or Class B.

For Class A, β is given by Expression (3.2) (see Table 3.3):

$$\beta = \left[\left(\frac{\varepsilon_{ud} - f_{p0,1k} / E_p}{\varepsilon_{uk} - f_{p0,1k} / E_p} \right) \times \left(\frac{f_{pk} - f_{p0,1k}}{f_{pk}} \right) + \frac{f_{p0,1k}}{f_{pk}} \right] \quad (3.2)$$

Where the definitions and values for ε_{ud} , ε_{uk} , $f_{p0,1k}$, f_{pk} and E_p at normal temperature are given in Section 3.3 of EN 1992-1-1.

For Class B, β is equal to 0,9 (see Table 3.3).

Note: The choice of Class A or Class B for use in a Country may be found in its National Annex.

Table 3.3: Values for the parameters of the stress-strain relationship of cold worked (cw) (wires and strands) and quenched and tempered (q & t) (bars) prestressing steel at elevated temperatures

Steel temp. θ [°C]	$f_{py,\theta} / (\beta f_{pk})$		$q \ \& \ t$	$f_{pp,\theta} / (\beta f_{pk})$		$E_{p,\theta} / E_p$		$\varepsilon_{pt,\theta} \ [-]$	$\varepsilon_{pu,\theta} \ [-]$
	Class A	Class B	cw	q & t	cw	q & t	cw, q&t	cw, q&t	
1	2a	2b	3	4	5	6	7	8	9
20	1,00	1,00	1,00	1,00	1,00	1,00	1,00	0,050	0,100
100	1,00	0,99	0,98	0,68	0,77	0,98	0,76	0,050	0,100
200	0,87	0,87	0,92	0,51	0,62	0,95	0,61	0,050	0,100
300	0,70	0,72	0,86	0,32	0,58	0,88	0,52	0,055	0,105
400	0,50	0,46	0,69	0,13	0,52	0,81	0,41	0,060	0,110
500	0,30	0,22	0,26	0,07	0,14	0,54	0,20	0,065	0,115
600	0,14	0,10	0,21	0,05	0,11	0,41	0,15	0,070	0,120
700	0,06	0,08	0,15	0,03	0,09	0,10	0,10	0,075	0,125
800	0,04	0,05	0,09	0,02	0,06	0,07	0,06	0,080	0,130
900	0,02	0,03	0,04	0,01	0,03	0,03	0,03	0,085	0,135
1000	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,090	0,140
1100	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,095	0,145
1200	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,100	0,150
Note: For intermediate values of temperature, linear interpolation may be used.									

(3) When considering thermal actions according to EN 1991-1-2 Section 3 (natural fire simulation), particularly when considering the decreasing temperature branch, the values for the stress-strain relationships of prestressing steel specified in (2) may be used as a sufficiently precise approximation.

3.3 Thermal and physical properties of concrete with siliceous and calcareous aggregates

3.3.1 Thermal elongation

(1) The thermal strain $\varepsilon_c(\theta)$ of concrete may be determined from the following with reference to the length at 20°C :

Siliceous aggregates:

$$\begin{aligned}\varepsilon_c(\theta) &= -1,8 \times 10^{-4} + 9 \times 10^{-6} \theta + 2,3 \times 10^{-11} \theta^3 & \text{for } 20^\circ\text{C} \leq \theta \leq 700^\circ\text{C} \\ \varepsilon_c(\theta) &= 14 \times 10^{-3} & \text{for } 700^\circ\text{C} < \theta \leq 1200^\circ\text{C}\end{aligned}$$

Calcareous aggregates:

$$\begin{aligned}\varepsilon_c(\theta) &= -1,2 \times 10^{-4} + 6 \times 10^{-6} \theta + 1,4 \times 10^{-11} \theta^3 & \text{for } 20^\circ\text{C} \leq \theta \leq 805^\circ\text{C} \\ \varepsilon_c(\theta) &= 12 \times 10^{-3} & \text{for } 805^\circ\text{C} < \theta \leq 1200^\circ\text{C}\end{aligned}$$

Where θ is the concrete temperature (°C).

(2) The variation of the thermal elongation with temperatures is illustrated in Figure 3.5.

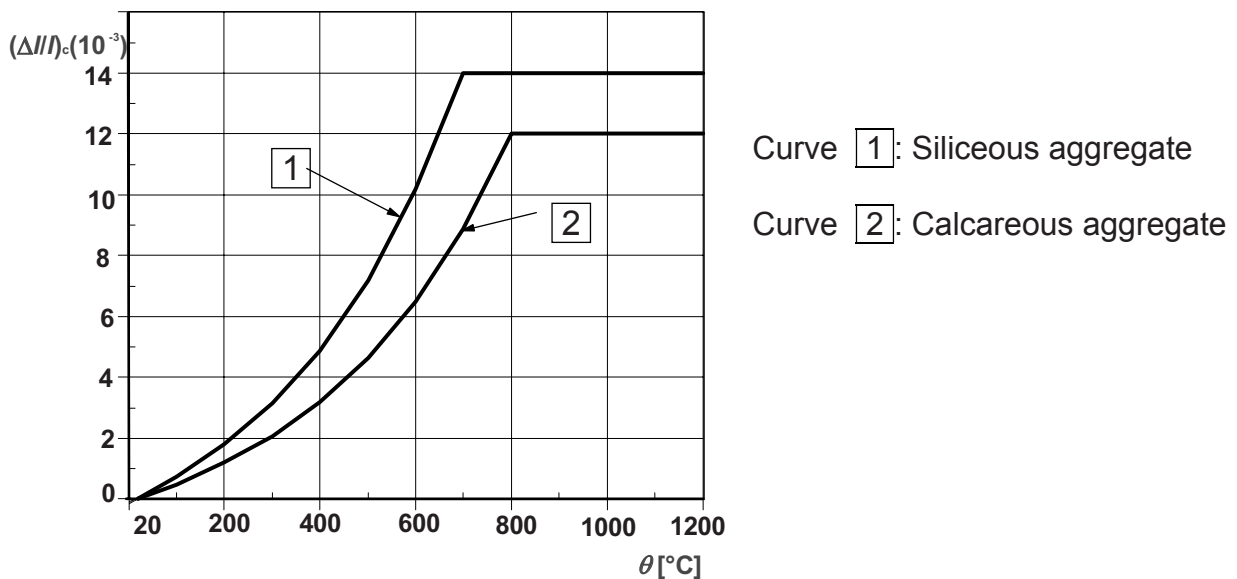


Figure 3.5 Total thermal elongation of concrete

3.3.2 Specific heat

(1) The specific heat $c_p(\theta)$ of dry concrete ($u = 0\%$) may be determined from the following:

Siliceous and calcareous aggregates:

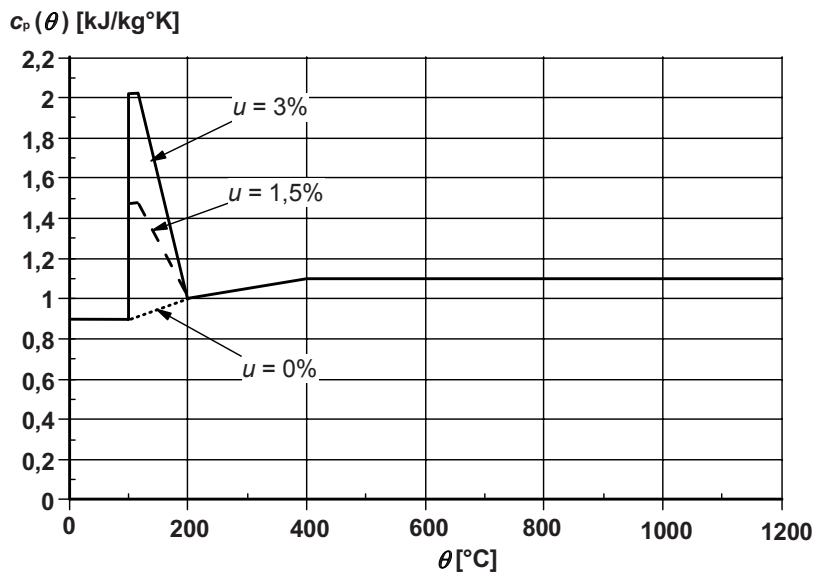
$$\begin{aligned}c_p(\theta) &= 900 \text{ (J/kg K)} & \cdot & \quad 20^\circ\text{C} \leq \theta \leq 100^\circ\text{C} \\ c_p(\theta) &= 900 + (\theta - 100) \text{ (J/kg K)} & & \quad \text{for } 100^\circ\text{C} < \theta \leq 200^\circ\text{C} \\ c_p(\theta) &= 1000 + (\theta - 200)/2 \text{ (J/kg K)} & & \quad \text{for } 200^\circ\text{C} < \theta \leq 400^\circ\text{C} \\ c_p(\theta) &= 1100 \text{ (J/kg K)} & & \quad \text{for } 400^\circ\text{C} < \theta \leq 1200^\circ\text{C}\end{aligned}$$

where θ is the concrete temperature ($^{\circ}\text{C}$). $c_p(\theta)$ (kJ/kg K) is illustrated in Figure 3.6a.

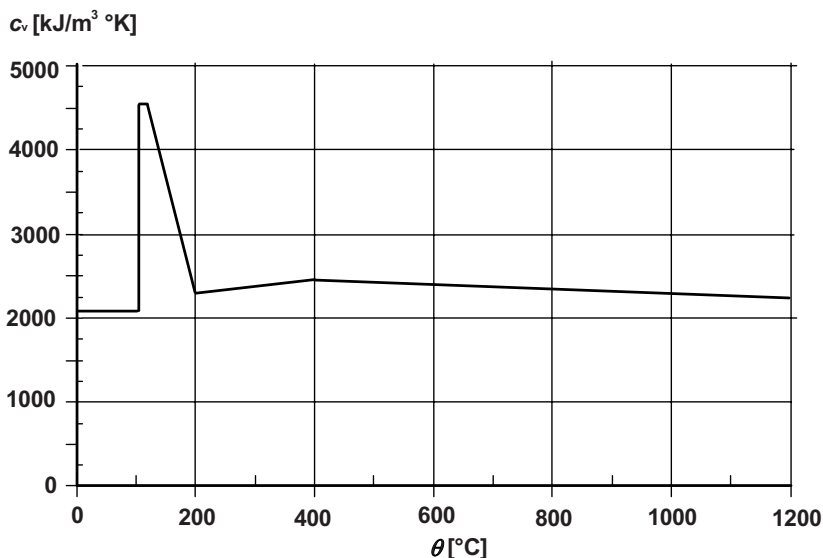
(2) Where the moisture content is not considered explicitly in the calculation method, the function given for the specific heat of concrete with siliceous or calcareous aggregates may be modelled by a constant value, $c_{p,\text{peak}}$, situated between 100°C and 115°C with linear decrease between 115°C and 200°C .

$$\begin{aligned} c_{p,\text{peak}} &= 900 \text{ J/kg K for moisture content of 0 \% of concrete weight} \\ c_{p,\text{peak}} &= 1470 \text{ J/kg K for moisture content of 1,5 \% of concrete weight} \\ c_{p,\text{peak}} &= 2020 \text{ J/kg K for moisture content of 3,0 \% of concrete weight} \end{aligned}$$

And linear relationship between $(115^{\circ}\text{C}, c_{p,\text{peak}})$ and $(200^{\circ}\text{C}, 1000 \text{ J/kg K})$. For other moisture contents a linear interpolation is acceptable. The peaks of specific heat are illustrated in Figure 3.6a.



a) Specific heat, $c_p(\theta)$, as function of temperature at 3 different moisture contents, u , of 0, 1,5 and 3 % by weight for siliceous concrete



b) Volumetric specific heat, $c_v(\theta)$ as function of temperature at a moisture content, u , of 3% by weight and a density of 2300 kg/m^3 for siliceous concrete

Figure 3.6: Specific heat and volumetric specific heat

(3) The variation of density with temperature is influenced by water loss and is defined as follows

$$\begin{aligned}\rho(\theta) &= \rho(20^\circ\text{C}) && \text{for } 20^\circ\text{C} \leq \theta \leq 115^\circ\text{C} \\ \rho(\theta) &= \rho(20^\circ\text{C}) \cdot (1 - 0,02(\theta - 115)/85) && \text{for } 115^\circ\text{C} < \theta \leq 200^\circ\text{C} \\ \rho(\theta) &= \rho(20^\circ\text{C}) \cdot (0,98 - 0,03(\theta - 200)/200) && \text{for } 200^\circ\text{C} < \theta \leq 400^\circ\text{C} \\ \rho(\theta) &= \rho(20^\circ\text{C}) \cdot (0,95 - 0,07(\theta - 400)/800) && \text{for } 400^\circ\text{C} < \theta \leq 1200^\circ\text{C}\end{aligned}$$

(4) The variation of volumetric specific heat $c_v(\theta)$ (product of $\rho(\theta)$ and $c_p(\theta)$) is illustrated in Figure 3.6b for concrete with a moisture content of 3% by weight and a density of 2300 kg/m³.

3.3.3 Thermal conductivity

(1) The thermal conductivity λ_c of concrete may be determined between lower and upper limit values, given in (2) below.

Note 1: The value of thermal conductivity may be set by the National annex within the range defined by lower and upper limit.

Note 2: Annex A is compatible with the lower limit. The remaining clauses of this part 1-2 are independent of the choice of thermal conductivity. For high strength concrete, see 6.3.

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

$$\lambda_c = 2 - 0,2451 (\theta / 100) + 0,0107 (\theta / 100)^2 \text{ W/m K} \quad \text{for } 20^\circ\text{C} \leq \theta \leq 1200^\circ\text{C}$$

where θ is the concrete temperature.

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

$$\lambda_c = 1,36 - 0,136 (\theta / 100) + 0,0057 (\theta / 100)^2 \text{ W/m K} \quad \text{for } 20^\circ\text{C} \leq \theta \leq 1200^\circ\text{C}$$

where θ is the concrete temperature.

(3) The variation of the upper limit and lower limit of thermal conductivity with temperature is illustrated in Figure 3.7.

3.4 Thermal elongation of reinforcing and prestressing steel

(1) The thermal strain $\varepsilon_s(\theta)$ of steel may be determined from the following with reference to the length at 20°C :

Reinforcing steel:

$$\begin{aligned}\varepsilon_s(\theta) &= -2,416 \times 10^{-4} + 1,2 \times 10^{-5} \theta + 0,4 \times 10^{-8} \theta^2 && \text{for } 20^\circ\text{C} \leq \theta \leq 750^\circ\text{C} \\ \varepsilon_s(\theta) &= 11 \times 10^{-3} && \text{for } 750^\circ\text{C} < \theta \leq 860^\circ\text{C} \\ \varepsilon_s(\theta) &= -6,2 \times 10^{-3} + 2 \times 10^{-5} \theta && \text{[AC1] for } 860^\circ\text{C} < \theta \leq 1200^\circ\text{C} \text{ [AC1]}\end{aligned}$$

Prestressing steel:

$$\varepsilon_p(\theta) = -2,016 \times 10^{-4} + 10^{-5} \theta + 0,4 \times 10^{-8} \theta^2 \quad \text{for } 20^\circ\text{C} \leq \theta \leq 1200^\circ\text{C}$$

where θ is the steel temperature ($^{\circ}\text{C}$)

(2) The variation of the thermal elongation with temperatures is illustrated in Figure 3.8.

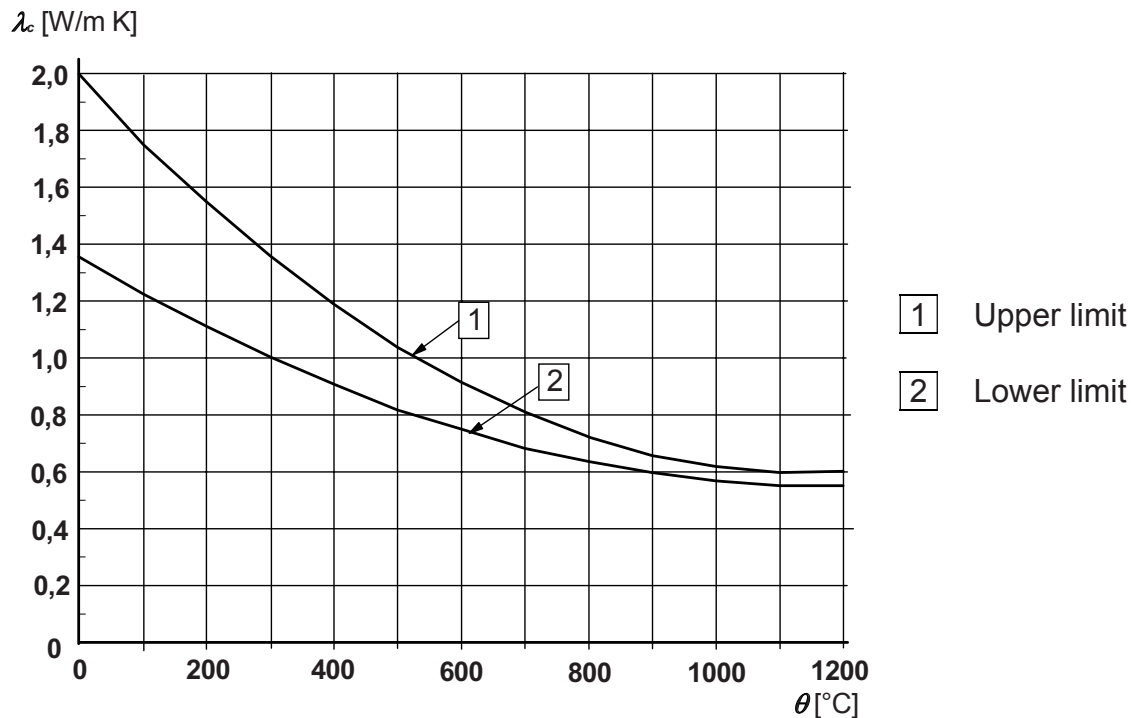


Figure 3.7: Thermal conductivity of concrete

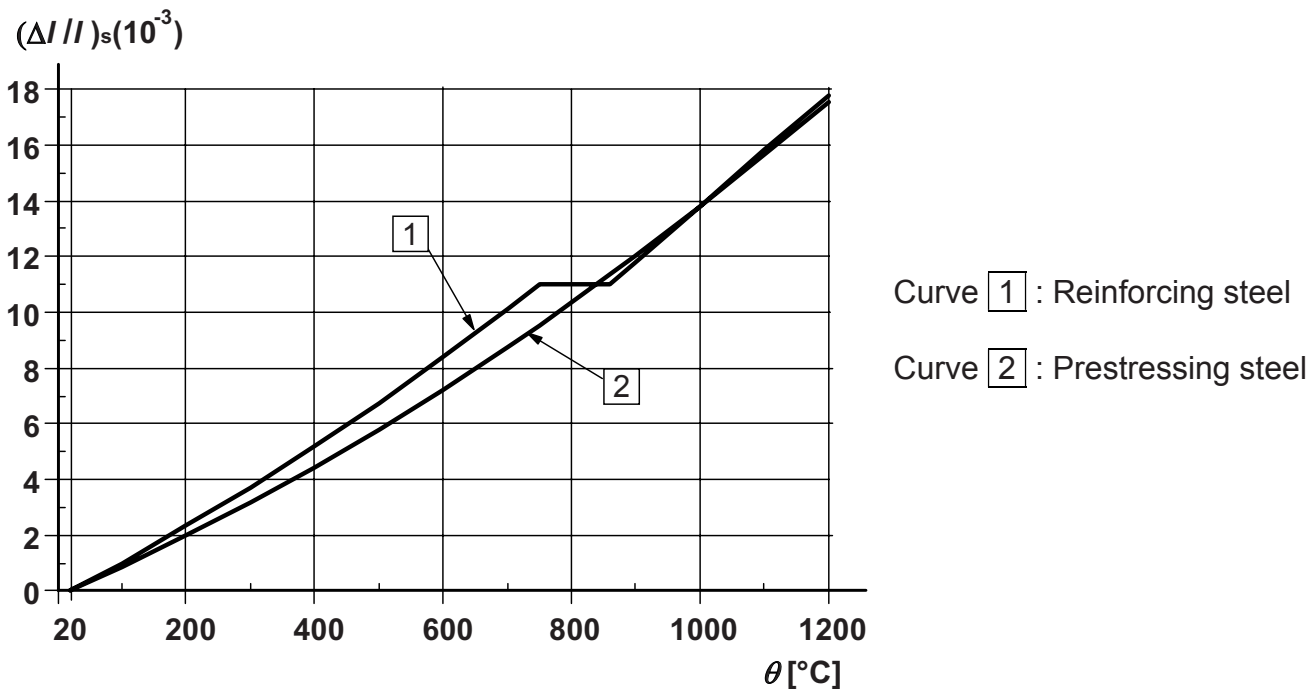


Figure 3.8: Total thermal elongation of steel

SECTION 4 DESIGN PROCEDURES

4.1 General

(1)P The following design methods are permitted in order to satisfy 2.4.1 (2)P:

- detailing according to recognised design solutions (tabulated data or testing), see Section 5
- simplified calculation methods for specific types of members, see 4.2
- advanced calculation methods for simulating the behaviour of structural members, parts of the structure or the entire structure, see 4.3.

Note 1: When calculation methods are used, reference is made to 4.6 for integrity function (E).

Note 2: For insulation function (I) the ambient temperature is normally assumed to be 20°C.

Note 3: The decision on the use of advanced calculation methods in a country may be found in its National Annex.

(2)P Spalling shall be avoided by appropriate measures or the influence of spalling on performance requirements (R and/or EI) shall be taken into account, see 4.5.

(3) Sudden failure caused by excessive steel elongation from heating for prestressed members with unbonded tendons should be avoided.

4.2 Simplified calculation method

4.2.1 General

(1) Simplified cross-section calculation methods may be used to determine the ultimate load-bearing capacity of a heated cross section and to compare the capacity with the relevant combination of actions, see 2.4.2.

Note 1: Informative Annex B provides two alternative methods, B.1 “500°C isotherm method” and B.2 “Zone method” for calculating the resistance to bending moments and axial forces. Second order effects may be included with both models. The two methods are applicable to structures subjected to a standard fire exposure. Method B.1 may be used in conjunction with both standard and parametric fires. Method B.2 is recommended for use with small sections and slender columns but is only valid for standard fires.

A1 **Note 2:** Tabulated data for the fire design of slender reinforced columns in braced and unbraced systems is given in Annex C. **A1**

(2) For shear, torsion and anchorage see 4.4.

Note: Informative Annex D provides a simplified calculation method for shear, torsion and anchorage.

(3) Simplified methods for the design of beams and slabs where the loading is predominantly uniformly distributed and where the design at normal temperature is based on linear analysis may be used.

Note: Informative Annex E provides a simplified calculation method for the design of beams and slabs.

4.2.2 Temperature profiles

(1) Temperatures in a concrete structure exposed to a fire may be determined from tests or by calculation.

Note: The temperature profiles given in Annex A may be used to determine the temperatures in cross-sections with siliceous aggregate exposed to a standard fire up to the time of maximum gas temperature. The profiles are conservative for most other aggregates.

4.2.3 Reduced cross-section

(1) Simplified methods using a reduced cross-section may be used.

Note: Informative Annex B provides two methods using a reduced cross section.

The method described in Annex B.1 is based on the hypothesis that concrete at a temperature more than 500 °C is neglected in the calculation of load-bearing capacity, while concrete at a temperature below 500 °C is assumed to retain its full strength. This method is applicable to a reinforced and prestressed concrete section with respect to axial load, bending moment and their combinations.

[AC1] The method described in Annex B.2 is based on the principle that cross-section is reduced by ignoring an ineffective zone at the fire-exposed surfaces. **[AC1]** The calculation should follow a specific procedure. The method is applicable to a reinforced and prestressed concrete section with respect to axial load, bending moment and their combinations.

4.2.4 Strength reduction

4.2.4.1 General

(1) Values for the reduction of the characteristic compressive strength of concrete, and of the characteristic strength of reinforcing and prestressing steels are given in this section. They may be used with the simplified cross-section calculation methods described in 4.2.3.

(2) The values for strength reduction given in 4.2.4.2 and 4.2.4.3 below should only be applied for heating rates similar to those appearing under standard fire exposure until the time of the maximum gas temperature.

(3) Alternative formulations of material laws may be applied, provided the solutions are within the range of experimental evidence.

4.2.4.2 Concrete

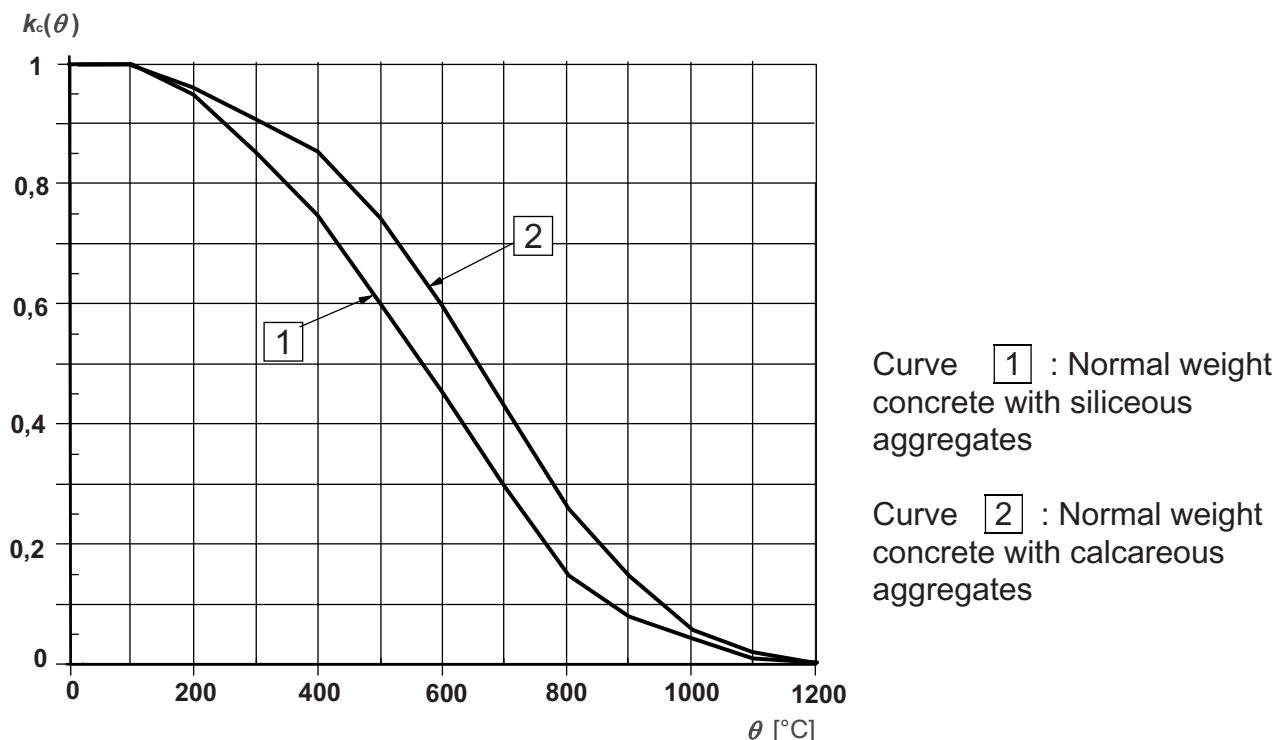


Figure 4.1: Coefficient $k_c(\theta)$ allowing for decrease of characteristic strength (f_{ck}) of concrete

(1) The reduction of the characteristic compressive strength of concrete as a function of the temperature θ may be used as given in Table 3.1 Column 2 for siliceous aggregates and Column 5 for calcareous aggregates (see Figure 4.1).

4.2.4.3 Steel

(1) For tension reinforcement the reduction of the characteristic strength of reinforcing steel as a function of the temperature θ is given in Table 3.2a. For tension reinforcement in beams and slabs where $\varepsilon_{s,fi} \geq 2\%$, the strength reduction for Class N reinforcement may be used as given in Table 3.2a, Column 2 for hot rolled and Column 3 for cold worked reinforcing steel (see Figure 4.2a, curve 1 and 2). The strength reduction for Class X reinforcement may be used as given in Table 3.2b for hot rolled and cold worked reinforcing steel (see Figure 4.2b, curve 1).

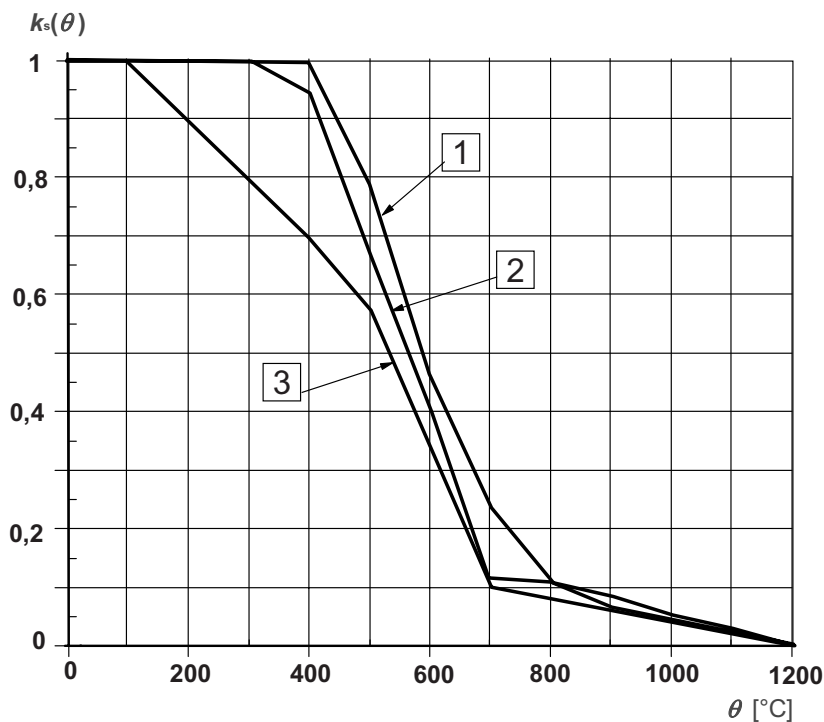
For compression reinforcement in columns and compressive zones of beams and slabs the strength reduction at 0,2% proof strain for Class N reinforcement should be used as given below. This strength reduction also applies for tension reinforcement where $\varepsilon_{s,fi} < 2\%$ when using simplified cross-section calculation methods (see Figure 4.2a, curve 3):

$$\begin{aligned} k_s(\theta) &= 1,0 && \text{for } 20^\circ\text{C} \leq \theta \leq 100^\circ\text{C} \\ k_s(\theta) &= 0,7 - 0,3 (\theta - 400)/300 && \text{for } 100^\circ\text{C} < \theta \leq 400^\circ\text{C} \\ k_s(\theta) &= 0,57 - 0,13 (\theta - 500)/100 && \text{for } 400^\circ\text{C} < \theta \leq 500^\circ\text{C} \\ k_s(\theta) &= 0,1 - 0,47 (\theta - 700)/200 && \text{for } 500^\circ\text{C} < \theta \leq 700^\circ\text{C} \\ k_s(\theta) &= 0,1 (1200 - \theta)/500 && \text{for } 700^\circ\text{C} < \theta \leq 1200^\circ\text{C} \end{aligned}$$

Similarly the strength reduction at 0,2% proof strain for Class X reinforcement may be used as given below. This strength reduction also applies for tension reinforcement where $\varepsilon_{s,fi} < 2\%$ (see Figure 4.2b, curve 2).

$$\begin{aligned} k_s(\theta) &= 1,0 && \text{for } 20^\circ\text{C} \leq \theta \leq 100^\circ\text{C} \\ k_s(\theta) &= 0,8 - 0,2 (\theta - 400)/300 && \text{for } 100^\circ\text{C} < \theta \leq 400^\circ\text{C} \\ k_s(\theta) &= 0,6 - 0,2 (\theta - 500)/100 && \text{for } 400^\circ\text{C} < \theta \leq 500^\circ\text{C} \\ k_s(\theta) &= 0,33 - 0,27 (\theta - 600)/100 && \text{for } 500^\circ\text{C} < \theta \leq 600^\circ\text{C} \\ k_s(\theta) &= 0,15 - 0,18 (\theta - 700)/100 && \text{for } 600^\circ\text{C} < \theta \leq 700^\circ\text{C} \\ k_s(\theta) &= 0,08 - 0,07 (\theta - 800)/100 && \text{for } 700^\circ\text{C} < \theta \leq 800^\circ\text{C} \\ k_s(\theta) &= 0,05 - 0,03 (\theta - 900)/100 && \text{for } 800^\circ\text{C} < \theta \leq 900^\circ\text{C} \\ k_s(\theta) &= 0,04 - 0,01 (\theta - 1000)/100 && \text{for } 900^\circ\text{C} < \theta \leq 1000^\circ\text{C} \\ k_s(\theta) &= 0,04 (1200 - \theta)/200 && \text{for } 1000^\circ\text{C} < \theta \leq 1200^\circ\text{C} \end{aligned}$$

(2) The reduction of the characteristic strength of a prestressing steel as a function of the temperature, θ , should be in accordance with 3.2.4 (2). Values may be taken from Table 3.3, Column 2a or 2b for cold worked steel and Column 3 for quenched and tempered prestressing steel (see Figure 4.3).

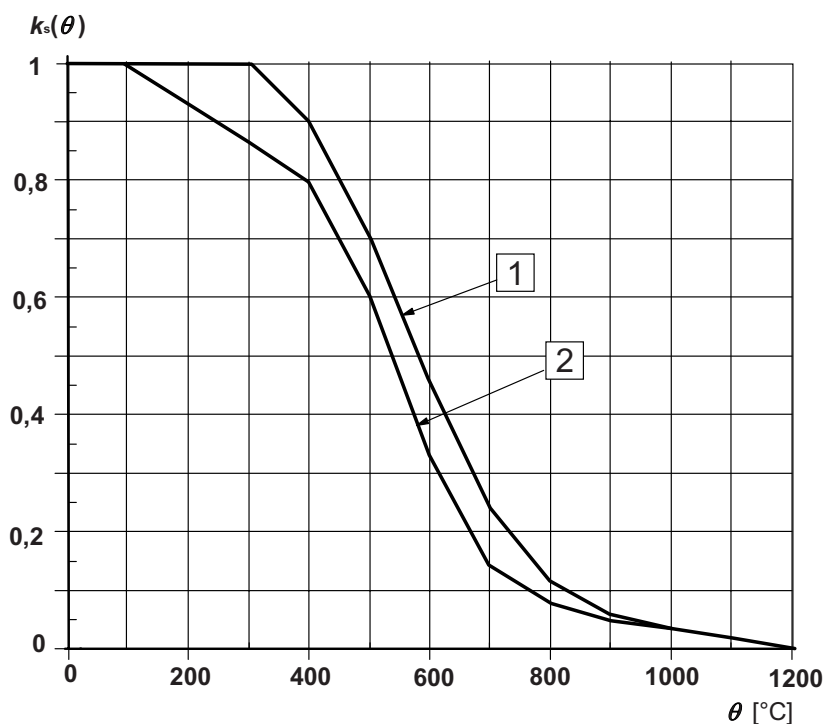


Curve **1** : Tension reinforcement (hot rolled) for strains $\varepsilon_{s,fi} \geq 2\%$

Curve **2** : Tension reinforcement (cold worked) for strains $\varepsilon_{s,fi} \geq 2\%$

Curve **3** : Compression reinforcement and tension reinforcement for strains $\varepsilon_{s,fi} < 2\%$

Figure 4.2a: Coefficient $k_s(\theta)$ allowing for decrease of characteristic strength (f_{yk}) of tension and compression reinforcement (Class N)



Curve **1** : Tension reinforcement (hot rolled and cold worked) for strains $\varepsilon_{s,fi} \geq 2\%$

Curve **2** : Compression reinforcement and tension reinforcement (hot rolled and cold worked) for strains $\varepsilon_{s,fi} < 2\%$

Figure 4.2b: Coefficient $k_s(\theta)$ allowing for decrease of characteristic strength (f_{yk}) of tension and compression reinforcement (Class X)