Behaviour and structural design of concrete structures exposed to fire

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Preface

This master thesis was written at the Royal Institute of Technology (KTH) in Stockholm, during the year I have spent there on international exchange. Instead of carrying out my five years of studies to ‘Master of Science in Civil Engineering’ solely at the University of Ghent in Belgium, I chose to perform the last year at KTH. Apart from the unforgettable experiences and the heart-warming friendships, this year of exchange also helped me grow in becoming a better engineer. Therefore I would like to use this opportunity to express an additional word of thanks to Prof. dr. ir. Stijn Matthys from Ghent and Ass. prof. dr. ing. Anders Ansell from Stockholm for creating the needed bilateral contract. Their kindness made this exchange, and by extension, this thesis, possible.

This master thesis came about in cooperation with Ass. prof. dr. ing. Anders Ansell, first my Swedish exchange coordinator and now also my thesis supervisor, and unites one of our common interests: the design of concrete structures for the fire situation. I started this work with no previous knowledge whatsoever concerning structural fire design. Neither in Belgium, nor in Sweden, is this subject covered in the education to civil engineer, perhaps the reason why it intrigued me that much. Therefore this master thesis is written with a reader in mind similar to me at that time: familiar with the normal temperature design of concrete structures, but untaught when it comes to the design for fire. This work attempts to fill that gap. I may only hope that someday someone uses this thesis as guidance, but I can already say that one person has greatly learned from this work: me.

My first and biggest gratitude goes out to Ass. prof. dr. ing. Anders Ansell for his support and guidance.

I would also like to thank Prof. dr. ir. Stijn Matthys and Dr. ir.-arch. Emmanuel Annerel for their help and for providing me with some much needed documentation. Furthermore, I would like to thank Techn. lic. ing. Robert Jansson for his review of Chapter 3 and for providing me with some of his pictures, as well as Prof. dr. ir. Johan Silfwerbrand for the inspirational talk. Prof. dr. ir. Johan Silfwerbrand and Univ. lect. Kjell Nilvér also lent me a couple of books from their personal collection, for which I am highly grateful.
Lastly, I would like to express my gratitude towards my friends and family whom supported me during this work. A special word of thanks goes out to my parents, my father for expecting nothing less than the best from me, but always loving me no matter what, and my mother, for always being there for me with never ending patience and advice, especially during these last weeks.

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Abstract

Concrete has an excellent intrinsic behaviour when exposed to fire, especially when compared to other building materials. However, its fire resistance should not be taken for granted and a proper structural fire design is certainly necessary. This design is based on the understanding of both the material and the structural behaviour of concrete exposed to fire. A number of complex physico-chemical reactions occur when concrete is heated, causing mechanical properties as strength and stiffness to deteriorate. Furthermore, the phenomenon of spalling causes pieces of concrete to break off from the surface, reducing the cross-section of an element and possibly exposing the reinforcing to the high temperatures. Spalling can be highly dangerous and is most common in high strength concrete. However, its mechanism is still not fully understood.

The Eurocode provides a number of procedures in order to design concrete structures for the fire situation, both prescriptive as performance based. However, of the latter, only the basic principles are given and several gaps still need to be filled through research. Thus in practical design, either tabulated data or a simplified calculation method is used. In many cases, these design methods fail to predict the true behaviour of concrete structures in real fires. Firstly, the standard heating curve is not able to represent the wide variety of realistic fires. Furthermore, design should investigate the behaviour of the complete structure, including alternative failure modes, whereas member analysis ignores effects as incompatible thermal expansions which can cause high thermal stresses.

Although a lot of research has been performed already, more in-depth study is needed. Several elements of the behaviour of heated concrete still need to be researched. A systematic study of the effects of realistic thermal exposures is needed and a lot more work is required in order to unravel the mystery of spalling. The study of the response of complete concrete structures presents another challenge, requiring large-scale fire tests. The goal is to develop a concrete model that reflects the true behaviour of concrete structures exposed to fire. This model should incorporate the fully coupled hygro-thermal-mechanical behaviour combined with a sophisticated structural analysis, including the effect of transient strain.

Keywords: Concrete, Fire, Design, Eurocode, Modelling, Review
Overzicht

Beton heeft een uitstekend intrinsiek gedrag bij brand, zeker in vergelijking met andere bouwmaterialen. Toch is zijn brandweerstand niet vanzelfsprekend en een degelijk structureel ontwerp voor brand is ongetwijfeld nodig. Dit ontwerp moet gebaseerd zijn op een inzicht in zowel het materiële als het structurele gedrag van verhit beton. Verschillende complexe fysico-chemische verschijnselen vinden plaats tijdens de opwarming, met als gevolg de afname van mechanische eigenschappen als sterkte en stijfheid. Bovendien, veroorzaakt het fenomeen genaamd spatten het afbreken van stukken beton van het oppervlak, wat de dwarsdoorsnede reduceert en mogelijk de wapening blootlegt. Spatten kan zeer gevaarlijk zijn in hogesterktebeton. Echter, het onderliggende mechanisms is nog steeds niet volledig gekend.

De Eurocode voorziet verscheidene procedures om betonconstructies te ontwerpen rekening houdend met de brandsituatie, zowel prescriptief als performantieel. Van de laatste zijn echter enkel de basisprincipes gegeven. In de praktijk, wordt dus meestal ontworpen aan de hand van getabellerde waarden of op basis van een vereenvoudigde berekeningsmethode. In vele gevallen, slagen deze methodes er niet in om het ware gedrag van betonconstructies bij brand te voorspellen. Ten eerste is de standaard brandcurve niet in staat om de grote verscheidenheid aan realistische branden voor te stellen. Bovendien moet het ontwerp de hele constructie analyseren, inclusief de verschillende wijzen van bezwijken, terwijl een elementen-analyse effecten negeert zoals incompatibele thermische uitzettingen die zeer hoge spanningen kunnen veroorzaken.

Hoewel er reeds veel werk is verricht, is er nog nood aan meer diepgaand onderzoek naar het gedrag van beton bij hoge temperaturen. Er is behoefte aan een systematische studie van de gevolgen van een realistische thermische blootstelling en nog meer werk is vereist om het mysterie van het spatten te ontcijferen. Een andere uitdaging is de studie van de reactie van betonconstructies in hun geheel. Dit vraagt voor meer brandtesten op grote schaal. Het doel is de ontwikkeling van een betonmodel dat het volledig gekoppelde hygro-thermisch-mechanisch gedrag omvat, gecombineerd met een geavanceerde structurele analyse, inclusief transiënte rek.

Trefwoorden: Beton, Brand, Ontwerp, Eurocode, Modelleren, Bespreking
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*Latin upper case letters*

$E_d$  
design effect of actions for normal temperature design

$E_{d,fi}$  
constant design value of the relevant effects of actions in the fire situation

$E_{d,fi,t}$  
design value of the relevant effects of actions in the fire situation at time $t$

$E_p$  
modulus of elasticity of prestressing steel at normal temperature

$E_{p,\theta}$  
modulus of elasticity of prestressing steel at temperature $\theta$

$E_s$  
modulus of elasticity of reinforcing steel at normal temperature

$E_{s,\theta}$  
modulus of elasticity of reinforcing steel at temperature $\theta$

$G_k$  
characteristic value of a permanent action

$Q$  
rate of heat release of the fire

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

$Q_{k,i}$  
characteristic value of the accompanying variable action $i$

$R_d$  
design value of the resistance of a member at normal temperature

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

$X_{d,fi}$  
design value of a material property for the fire situation

$X_k$  
characteristic value of a material property for normal temperature

$X_{k,\theta}$  
characteristic value of a material property in the fire situation, generally dependent on the material temperature $\theta$
Latin lower case letters

\(a\) axis distance of reinforcing or prestressing steel from the nearest exposed surface (nominal)

\(b_{\text{min}}\) minimum required dimension of the cross-section of an element

\(c_p(\theta)\) specific heat of concrete as a function of the temperature \(\theta\)

\(c_{p,\text{peak}}\) peak value of the specific heat for concrete that incorporates the effect of moisture content

\(c_v(\theta)\) volumetric specific heat of concrete as a function of the temperature \(\theta\)

\(f_{c,\theta}\) characteristic value of the compressive strength of concrete at temperature \(\theta\)

\(f_{cd}\) design compressive strength of concrete for normal temperature

\(f_{ck}\) characteristic compressive strength of concrete for normal temperature

\(f_{ck,t}\) characteristic value of the tensile strength of concrete at normal temperature

\(f_{ck,t}(\theta)\) characteristic value of the tensile strength of concrete for the fires situation

\(f_{pk}\) characteristic yield strength of prestressing steel at normal temperature

\(f_{pp,\theta}\) proportional limit of prestressing steel at temperature \(\theta\)

\(f_{py,\theta}\) characteristic yield strength of prestressing steel at temperature \(\theta\)

\(f_{sp,\theta}\) proportional limit of reinforcing steel at temperature \(\theta\)

\(f_{sy,\theta}\) characteristic yield strength of reinforcing steel at temperature \(\theta\)

\(f_{yk}\) characteristic yield strength of reinforcing steel at normal temperature
h_{net} \quad \text{net heat flux to unit surface area}

h_{net,c} \quad \text{net heat flux to unit surface area due to convection}

h_{net,r} \quad \text{net heat flux to unit surface area due to radiation}

k_{c,t}(\theta) \quad \text{reduction factor of the characteristic tensile strength of concrete as a function of the temperature } \theta

k_{s}(\theta) \quad \text{strength reduction of reinforcing steel as a function of the steel temperature } \theta

k_{\theta} \quad \text{reduction factor for a strength or deformation property dependent on the material temperature } \theta

q_{f,d} \quad \text{design fire load density}

t \quad \text{time [min]}

\text{t}_h \quad \text{time [hours]}

u \quad \text{moisture content of concrete}

x \quad \text{distance from the surface}

\textit{Greek upper case letters}

\Theta_c \quad \text{temperature within the concrete at a certain depth } x \, [^\circ\text{C}]

\Theta_g \quad \text{gas temperature in the fire compartment, or near the member [^\circ\text{C}]}

\Theta_m \quad \text{surface temperature of the member [^\circ\text{C}]}

\Theta_r \quad \text{effective radiation temperature of the fire environment [^\circ\text{C}]}

\Phi \quad \text{configuration factor}

\textit{Greek lower case letters}

\alpha_c \quad \text{coefficient of heat transfer by convection}
$\gamma_c$ partial safety factor for concrete material properties for normal temperature

$\alpha_{cc}$ coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied

$\theta_{cr}$ critical steel temperature

$\gamma_G$ partial factor for permanent actions

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

$\gamma_Q$ partial factor for variable actions

$\gamma_{Q,i}$ partial factor for variable action $i$

$\gamma_s$ partial safety factor for steel material properties for normal temperature

$\beta$ Reduction factor of the strength properties of prestressing steel

$\varepsilon$ strain

$\varepsilon_c(\theta)$ thermal strain of concrete as a function of the temperature $\theta$

$\varepsilon_{c1,\theta}$ concrete strain corresponding to $f_{c,\theta}$

$\varepsilon_{\text{creep}}$ creep strain

$\varepsilon_{cu1,\theta}$ concrete strain that defines the end of the descending branch

$\varepsilon_f$ emissivity of the fire, or of the flames

$\varepsilon_m$ surface emissivity of the member

$\varepsilon_p(\theta)$ thermal strain of prestressing steel as a function of the steel temperature $\theta$

$\varepsilon_{p,\theta}$ strain of prestressing steel at temperature $\theta$

$\varepsilon_s(\theta)$ thermal strain of reinforcement steel as a function of the steel temperature $\theta$
\( \varepsilon_{s,\theta} \) strain of reinforcing steel at temperature \( \theta \)

\( \varepsilon_{th} \) thermal strain

\( \varepsilon_{tr} \) transient state strain

\( \varepsilon_o \) instantaneous stress-dependent strain

\( \eta_f \) reduction factor for design load level in the fire situation

\( \theta \) temperature [°C]

\( \lambda_c \) thermal conductivity of concrete.

\( \mu_f \) load level or degree of utilisation for fire design

\( \xi \) reduction factor for unfavourable permanent action \( G \)

\( \rho(\theta) \) density of the concrete as a function of the temperature \( \theta \)

\( \sigma \) stress

\( \sigma^* \) stress history

\( \sigma^{\text{rb}} \) Stephan Boltzmann constant (= \( 5.67 \times 10^{-8} \text{ W/m}^2\text{K}^4 \))

\( \psi_0 \) factor for combination value of a variable action

\( \psi_{0,i} \) factor for combination value of the variable action \( i \)

\( \psi_1 \) factor for frequent value of a variable action

\( \psi_{1,i} \) factor for frequent value of the variable action \( i \)

\( \psi_2 \) factor for quasi-permanent value of a variable action

\( \psi_{2,i} \) factor for quasi-permanent value of the variable action \( i \)

\( \psi_f \) combination factor for frequent or quasi-permanent values given either by \( \psi_{1,1} \) or \( \psi_{2,1} \)
1 Introduction

1.1 Background

Concrete is a material that has an excellent intrinsic behaviour when exposed to fire. It does not burn, i.e. it is non-combustible, and it has a high thermal massivity, which significantly slows down the spread of heat through concrete elements. As a matter of fact, in most common fires only the outer layer of the concrete with a thickness of approximately 3 to 5 cm is damaged (Denoël, 2007). Therefore, many concrete buildings that experienced fire, can be fairly simply restored and reused. An excellent example of the good behaviour towards fire of concrete structures is the Windsor Tower in Madrid (Denoël, 2007). The fire occurred on 14 February 2005, during which the building was fortunately unoccupied. Despite that the fire spread over numerous floors and lasted 26 hours, the building remained standing, as can be seen in Figure 1.1 and Figure 1.2. The only part that did collapse where the steel perimeter columns above the 20th floor, which supported the floors.

Figure 1.1: The Windsor Tower in Madrid after a 26 hour fire in 2005. Photograph taken by DavidHT (CC BY 2.0).

Figure 1.2: Close-up of the top floors of the Windsor Tower. Photograph taken by maxirafer (CC BY-NC-SA 2.0).
Nevertheless, the fire resistance of concrete structures should not be taken for granted. A proper structural fire design is needed, however concrete remains a complex material, built up of several constituents that behave differently when heated. Several physico-chemical transformations take place in the concrete resulting in a decrease of strength and stiffness. Also spalling may occur which is the, possibly violent, breaking off of material from the surface of a concrete member, reducing the cross-section and possibly exposing the reinforcement to the high temperatures. Consequently, due to its complexity, the behaviour of concrete exposed to fire is not yet fully understood. The design codes and methods that exist today are greatly empirically based and no model has been developed that fully reflects the true behaviour. This is made even more difficult due to new developments in the field of concrete. New structural works keep pushing the limit, ever meeting new challenges, for example higher structural complexity as with high rise buildings, the use of high strength concrete, more economic designs, or building in extreme environments, e.g. off-shore platforms or earthquake-prone areas. With this, new challenges arise with respect to fire safety. This became also clear by several severe tunnel fires over the past decade. For example the fires in the Great Belt tunnel (Denmark, 1994) and the Channel tunnel (UK/France, 1999) which did not claim any lives, but experienced extensive damage and extreme spalling of the tunnel elements made of the recently developed high-performance concrete (Khoury, 2000). Up to 68% of the thickness of tunnel was spalled away in the Great Belt tunnel and in some places even 100% in the Channel tunnel. The only thing standing between total loss and a situation where effective repair could be carried out was the grout layer between the concrete structure and the water bearing rock layer. This illustrates that fire can have a disastrous effect on concrete structures and should not be overlooked during the design.

1.2 Aims and contents of the report

The aim of the report is to comprehensively describe how concrete structures are to be designed in order to achieve an appropriate fire resistance, according to the methods provided in the Eurocode, the state-of-the-art in design codes in Europe. The goal is not to literally copy the Eurocode but to provide insight and understanding of the different design methods. Attention is also paid to the specific behaviour of concrete under high temperatures and which processes occur within, since an understanding of the material is key to a good design. Furthermore, the
given design methods are critically evaluated and recommendations are given as to where more work is needed.

This text is written in a broad sense so that no previous knowledge of fire design or the combination of concrete and fire is needed. However, it is assumed that the reader is familiar with the material concrete and the basic techniques of structural design for normal temperatures.

This report contains eight distinct Chapters. Chapter 2 gives an introduction to fire physics and explains how a fire ignites and grows. Some basic concepts are also clarified. Whereas Chapter 2 is general and applicable to all materials, Chapter 3 focuses on concrete specific and how it behaves when exposed to fire. This behaviour is then taken and applied to concrete structures in Chapter 4, where it is also illustrated that a concrete structure may be more than simply the sum of its members. Then the focus turns to the actual design of concrete structures. Chapter 5 provides a background to the design regulations that are applied here, the Eurocodes. Since the author studied in both Belgium and Sweden, these two countries serve as an example of the application of the Eurocodes on national level, in Chapter 5 as well as Chapter 6, where the actual structural fire design of concrete structures is studied, both in a prescriptive as in a performance-based way. The different steps of the fire design are identified and clarified and the alternative design procedures are discussed. Additionally, Chapter 7 provides three worked examples of design as an illustration of the ‘simplified calculation method’. In Chapter 8, the knowledge gained is used to take a critical look at the fire design of concrete structures, particularly with respect to the design methods that are most commonly used in practice. Additionally, the status of the research is discussed and suggestions are made in order to improve these design methods.
2 The basics of fire physics and fire safety

2.1 The fire triangle

A fire (Denoël, 2007) can only start when the following three elements are present simultaneously: oxygen (21 % volume in air), combustible materials and a heat source. Together, they make up what is commonly called the fire triangle, which is also shown in Figure 2.1. The first two elements will only start the process of combustion when the inflammation temperature is reached. The combustion of carbon produces carbon dioxide (CO$_2$) and, in case of a lack of oxygen, the well known gas carbon monoxide (CO) which is very dangerous to man.

![Figure 2.1: The fire triangle. Redrawn from Denoël (2007).](image)

2.2 The development of a fire and flashover

This Section discusses the behaviour and the different stages of fires in rooms. The stages are ignition, growth, flashover, fully-developed fire and decay, as can be seen in Figure 2.2. Since their behaviour is completely different, a distinction is made between pre- and post-flashover fires. The information in this Section is based on Buchanan (2002) and Denoël (2007).

Generally it is found that, when structurally designing a building, the post-flashover fire is of the essence. When designing for life safety in buildings, an understanding of the pre-flashover fire is essential.
2.2.1 Pre-flashover

When all three elements of the fire triangle are present, a fire originates. A small amount of material starts to burn and the first gasses and smoke appear. A plume of smoke develops, transporting the combustion products up to the ceiling. Initially, the combustion process consumes the oxygen from the air in the room, but soon air will flow in through openings like a door, a window or a ventilation opening. The energy released by the fire acts like a pump, pulling the fresh air inside, entraining it in the fire plume where it cools and dilutes the combustion products that are then pushed out. The diluted combustion products gather and stagnate in a hot upper layer in the room, with its thickness and temperature increasing as the fire grows. The cool lower layer exists of fresh air that is slightly heated by mixing and radiation from the upper layer. These two layers are illustrated in Figure 2.3. The cool lower layer allows safe evacuation and is thus essential for life safety. Where the plume reaches the ceiling, the smoke and hot gasses spread radially outwards along the surface. This is called the ceiling jet. The shape and direction of the ceiling jet depends on the type of ceiling. For example, in case of a horizontal and smooth surface, the flow will be the same in each direction.

As the fire continues to burn, the hot upper layer grows and the height of the interface between the two layers drops. When the interface reaches, for example, the top of an open door, the hot gasses are able to escape. The thickness of the upper layer depends on the size and duration of the fire and the size and position
of the openings. When not enough fresh air is fed to the fire, e.g. when the openings are too small, a lack of oxygen occurs and the fire dies.

![Diagram of fire physics](image)

**Figure 2.3: Pre-flashover fire in a room. Based on a figure from Buchanan (2002).**

The previous description presumes only one single item burning. However, combustible materials on floor, walls and ceiling may significantly influence the development of the fire due to rapid spread of flames. In this case, temperatures will be higher and the fire will grow significantly faster in a well-insulated room where the bounding elements absorb the heat less. Computer models predicting the behaviour of a fire in a room with combustible lining are under development (e.g. Wade and Barnett, 1997).

A post-flashover fire is commonly modelled by a two-zone model, consisting of two homogenous layers and the connecting plume. The model uses conservation laws for mass, momentum and energy that are applied to each zone in a dynamic process that calculates the size, temperature and species concentration of each zone as a function of the process of the fire, together with the flow of smoke and toxic products through the openings. This way height, temperature and concentrations of gas species in both layers, as well as floor and wall temperatures, and the heat flux at floor level can be calculated. The model requires the choice of a design fire (see also 6.4.2 and 6.6) that specifies the growth of the fire. The assumption of two distinct layers in the two-zone model, however, does not agree with reality, where the interface forms a gradual, three-dimensional transition of temperature density and smoke. This can be modelled by the more sophisticated field model that uses computational fluid dynamics and three dimensional finite elements. Their high degree of complexity makes them more suitable for research tools rather than design tools. Alternatively, a post-flashover can also be modelled by a localised fire model. Then, only the heat flux through the plume is considered or the heat flux through flames when these impact the ceiling or any other structure.
above the fire. This model is particularly of interest when the fire occurs in an unenclosed space.

2.2.2 Flashover

As long as there is sufficient combustible material in the room and a proper supply of oxygen, the pre-flashover fire continues to grow. The temperatures in the hot layer will increase causing the radiant heat flux to all the objects of the room to increase as well. At a certain point (usually around 500 to 600 °C), this radiant heat flux will reach a critical value and all exposed combustible materials in the room will start to burn, leading to a rapid increase in both heat release rate and temperatures. This transition is called the flashover.

The definition of a flashover is the transition from a localised fire to combustion of all exposed combustible surfaces in a room. Thus, it is not possible for a flashover to occur in an open unenclosed space since, by definition, it can only occur in an enclosed compartment. Furthermore, it may be pointed out that the term flashover describes a transition rather than a precise event (Drysdale, 1998). However, to simplify design in practice, the growth period between the onset of flashover and the maximum heat release is often ignored and it is assumed that when flashover occurs the rate of heat release instantaneously increases to the maximum value set by the available air. This can also be seen in Figure 2.4.

2.2.3 Post-flashover

The behaviour of the fire before and after flashover is completely different. After flashover, there are not two layers anymore but rather one big zone where the flows of air and combustion gases are highly turbulent. The post-flashover fire, also called fully-developed fire or full-room involvement, usually has a temperature of more than 1000 °C. These high temperatures, together with the radiant heat fluxes, cause all the exposed combustible surfaces in the room to pyrolyse, producing large quantities of combustible gases, which burn where there is sufficient oxygen. The amount of available oxygen determines if a fire is either ventilation controlled or fuel controlled, depending on how much oxygen is available. In a typical room, the fire is ventilation controlled, so the rate of combustion depends on the number, the size and the shape of the openings. Usually, it is conservatively assumed that all window glass breaks and falls out due to the rapid increase in temperature during flashover. Typical about ventilation controlled fires are the flames extending out of the windows. Because of the insufficient amount of air inside the room, not all of the combustible gasses can burn. When
these mix with the outside air, an additional combustion takes place, resulting in flames coming through windows. On the other hand, fuel controlled fires especially occur in large, well-ventilated rooms where the surface area of the fuel is limited. The fire is then very similar to a fire in open air, but including the radiative feedback of the hot upper layer of gases or of hot walls and ceiling surfaces. Most fires become fuel controlled in the decay phase.

The post-flashover fire is of most interest when structurally designing a building for fire safety. Estimating the temperature in a post-flashover fire is essential, unfortunately, this cannot be done precisely. In the literature, several measured and predicted temperatures can be found. There exist also a number of computational models. These are usually based on, what is called, a single-zone model, which consider the room to be a well-mixed reactor. One representation of a post-flashover fire is the nominal fire curves which simply give the evolution of the gas temperature, which is assumed to be uniform in the compartment, as a function of time. The most used nominal curve is the standard curve and this is illustrated in Figure 2.4.

![Figure 2.4: The standard fire curve as a representation of a real fire.](image)

### 2.3 Fire safety

Unwanted fire is a destructive force that causes many thousands of deaths and billions of dollars of property loss each year (Buchanan, 2002). Although the probability is low, fire may occur anywhere, in any season, in any phase in the lifetime of a building and often when least expected. The safety of the occupants relies on many factors in the design and construction of the buildings, including the expectation that a certain building or a part of a building subjected to fire will not collapse or allow the fire to spread. Unfortunately it is impossible to prevent all fires. Fire protection therefore consists of reducing the probability of oc-
currence and limiting the consequences, i.e. death, injury and property loss. The essential requirements for the limitation of fire risks consist in (Denoël, 2007 and EN 1992-1-2, 2004):

- maintaining the load-bearing capacity of the structure for a specified period of time
- reducing the development of fire and smoke
- avoiding spread of fire
- ensuring the speedy evacuation of occupants in relative safety
- facilitating the intervention of the fire service

The balance between life safety and property protection varies in different countries, depending on the type of building and its occupancy. There has been a recent trend for national codes to give more emphasis to life safety than to property protection (Buchanan, 2002). It is found that many codes consider fire damage to a building or to goods more as the responsibility of the building owner or the insurer, where as the code intends to provide life safety and protection to the property of other people. An additional goal in many countries is to limit environmental damage in the event of a fire. In agreement with the former, the Eurocode (EN 1992-1-2, 2004) states its general objectives of fire protection as limiting the risks concerning the individual and society, neighbouring property, and where required, environment or directly exposed property.

Given that some fires will always occur, there are many strategies for reducing their impact. The best proven fire safety technology is automatic fire sprinkler systems because they have been shown to have a very high probability of controlling or extinguishing fires (Buchanan, 2002). Other necessary measures are for example to provide facilities for the detection and notification of fires, safe travel paths for the movement of occupants and fire-fighters, barriers to control the spread of fire and smoke, and structures which will not collapse prematurely when exposed to fire. The proper selection, design and use of building materials are crucial.

One part of the overall fire design is fire resistance. This is provided to selected structural members and non-structural barriers in order to prevent the spread of fire and smoke, or to prevent structural collapse during an uncontrolled fire. Fire resistance is often described as passive fire protection, which is always ready and waiting for a fire, as opposed to active fire protection such as automatic sprinklers which are required to activate after a fire is detected. Design strategies of-
ten incorporate a combination of active and passive fire protection measures. Fire resistance is of little significance in the very early stages of a fire, but becomes increasingly important as a fire gets out of control and grows beyond flashover to full room involvement. The importance of fire resistance depends of the size of the building and the fire safety objectives. To provide life safety, fire resistance is essential in buildings where a fire could grow large before all occupants have time to escape. The material concrete exhibits all the qualities for an excellent fire resistance.

2.4 Reaction to fire and resistance to fire

The terms reaction to fire and resistance to fire are very often used and attention should be paid to their definition and how they differ (Denoël, 2007 and KMO Normen-Antennes: Brandpreventie, 2011).

**Reaction to fire** applies to construction materials as such and is a measure of all the properties of a material that relate to the start and the development of a fire. It is characterised by the calorific potential, non-combustibility, inflammability, the means of propagation of flames on the surface of materials and, where applicable, by other properties such as the formation of smoke and the production of toxic gases.

**Resistance to fire** applies to structural elements and is a measure of their capability of maintaining their function (e.g. separating or load-bearing function) during the course of a fire.

The two properties thus have two completely different meanings. The first has an influence on the birth and the development of a fire where the second is of importance for a fire in its full intensity. For example, wood is a material with a poor reaction to fire because it is able to burn. Wooden beams or columns, on the contrary, exhibit a good resistance to fire. This is the opposite for steel, which has a good reaction to fire but a very poor resistance to fire. Concrete combines both qualities, which makes it an excellent material for fire safety.

The structural fire design through the Eurocode focuses on structural behaviour and thus only resistance to fire is covered. As said, this applies to structural elements and not the material, but it should be noted that the properties of a certain material will affect the performance of the element. Therefore, the concrete material and its behaviour when exposed to fire will be discussed in the next Section.
Chapter 3. The material concrete and fire

3 The material concrete and fire

3.1 General

Concrete is an excellent material and is commonly used for all kinds of buildings and civil-engineering projects. This may be for several reasons like for example price, speed of construction, aesthetics or architectural appearance. With this comes an excellent intrinsic behaviour in the fire situation which stands out when compared to many other common building materials. For example, concrete simply does not burn, it is non-combustible. Concrete (Denoël, 2007) will not propagate fire and will not give off smoke or toxic gases, neither will it melt nor will elements detach itself or drip from the concrete. Furthermore, concrete has a high thermal inertia and concrete elements are generally built in a massive manner, especially compared to for example steel members. High thermal massivity makes that concrete can withstand high temperatures for a relatively long time. While the temperature of the fire-exposed sides of the concrete structure is high, the cooler inner core will continue carrying the load. Also the reinforcing steel remains protected for a long time in the duration. After a fire, concrete buildings can often be easily repaired and reused. However, after this list of benefits, concrete should not be taken for granted. Concrete remains fundamentally a complex material and its properties can change dramatically when exposed to high temperatures. The principal effects of fire on concrete are deterioration of mechanical properties as temperature rises, most importantly the loss of compressive strength, and, the forcible ejection of chunks of concrete from the surface of the material, reducing the cross-section and possibly exposing the reinforcement to the fire.

A lot of information has been gathered today, but still a lot of work needs to be done (Fletcher et al., 2007). The behaviour of concrete in fire is not easily defined or modelled. Concrete is far from being a homogenous material, consisting of a composite of cement gel, aggregate, and, frequently, steel (or other) reinforcement, and each of these components have a different reaction to thermal exposure in itself. Furthermore, a member exposed to fire, experiences steep thermal gradients over its cross-section. This is mainly a consequence of the shape of concrete sections and their thermal massivity, more than the thermal conductivity as most people think (fib Bulletin 46, 2008). In other words, at different depths, the member has different temperatures and, consequently, different material
properties. Thus, whereas for example the design of steel members often uses the “lumped parameter” simplification, this is impossible to apply for concrete. Progress has been made on modelling the thermo-mechanical behaviour of concrete but the treatment of detailed behaviours, including hygral effects and spalling, remains a challenge.

3.2 Physical and chemical response to fire

Concrete subjected to heat will undergo changes in its microstructural, thermal, hydral and mechanical behaviour. Strength loss occurs mainly due to the formation of internal cracks and degradation and disintegration of the cement paste. The cohesion between the cement paste and the aggregates is also affected. Understanding the different processes will help understanding how concrete is likely to behave under fire, but also how to optimize the composition of the material for better fire performance. The information in this Section has been based on the works of Fletcher et al. (2007), Khoury (2008:1) and Denoël (2007).

The physical and chemical changes inside concrete can be reversible or non-reversible upon cooling. When the changes are non-reversible, a concrete structure may be significantly weakened after a fire, even if no damage can be visually detected. On the other hand, these changes may be used as indicators of maximum exposed temperature. Most changes, especially at ‘lower’ temperature ranges, will occur in the hardened cement paste. Most commonly used aggregates are stable up to a temperature of 300°C, however their behaviour can differ greatly depending on the type of aggregate.

A description of the different physico-chemical changes is given. In reality, the temperatures and effect can vary slightly, since these depend on a multitude of factors, as described later on. When the temperature of concrete start the rise, at first the material will just expand and normally no damage will occur. The first change that occurs is the evaporation of the free water inside the concrete material. Due to effects of pore pressure and pore size (see further) within the concrete, the boiling temperature may range from 100 to 140°C. This evaporation of water may cause a build-up of pressure within the concrete. Eventually, the chemically bound water will also evaporate, at temperatures between 100 and 800°C. Starting from 300°C the cement past will begin to shrink, while the aggregates expand. Long-term heating at this temperature will significantly reduce the tensile strength. At a temperature of approximately 400°C up to 600°C, the calcium hydroxide (Ca(OH)₂) breaks down into calcium oxide (CaO) and water.
(H\textsubscript{2}O), causing even more water vapour and a significant physical strength reduction. The aggregate is also affected by the fire. For example, quartz-based aggregates experiences a volume expansion, due to a mineral transformation (\(\alpha\)-quartz in to \(\beta\)-quartz), at about 575\(^\circ\)C. Limestone aggregates will start to decompose at approximately 800\(^\circ\)C. Generally speaking, the thermal response of any aggregate may be very straightforward and easy to be found. The difficulty lies in how the concrete material as a whole reacts to the changes inside the aggregate. For example in the case of different thermal expansions between the cement matrix and the aggregate, which may cause cracking and spalling. Some more physico-chemical changes of Portland cement are illustrated in the simplified Figure 3.1. The cooling down of the concrete after the fire also results in physical and chemical changes, e.g. crack development, moisture absorption or rehydration of CaO. Note that most reactions mentioned here, e.g. dehydration or decarbonatation, are endothermic reactions. The reaction will absorb energy in the form of heat, and thus slightly slow down heating. This effect is however conservatively ignored in design calculations.

As said above, the free water in the concrete has a variable boiling point (Jansson R., 2008) which is the result of two effects considering pore pressure and pore size. First, the boiling point of water is dependent on pressure. At 1 atm, water boils at 100\(^\circ\)C, but if the pressure increases, the boiling temperature increases. For example, at 2 atm (which is not uncommon in fire exposed concrete), the boiling point already increases to 120\(^\circ\)C. Secondly, according to Jansson (2008), The-ladersson (1974) described that the capillary forces acting on the water inside a porous media will also lead to a higher evaporation temperature, as the surface tension is temperature dependent. The variable boiling point is also recognised by the Eurocode (EN 1992-1-2, 2004) and can also be seen illustrated in Figure 6.11 where the temperature peak represents the latent heat.

All these physical and chemical changes in both the heating and the cooling down phase of the fire depend on the type of cement paste, the type of aggregate, the bond region and the interaction between them. Consequently, the behaviour of concrete can differ radically depending on which concrete type (i.e. constituents, mix proportions, preparation, etc.) is studied. This is a very important point. Khoury (2008:1) therefore advises not to use the term 'concrete' anymore when dealing with fire but the more specific term 'concrete type'. Because of this range of behaviour, it can easily be seen that significant performance improvements can be made through a smart choice of both the aggregate and cement blend. For ex-
ample, thermally stable aggregates of low thermal expansion (e.g. basalt, granite) or a cement blend including certain replacements (e.g. blast furnace slag).

Figure 3.1: Simplified global presentation of physic-chemical processes in Portland cement concrete during heating presented by a ‘thermometer’ analogy – for guidance only. Redrawn from Khoury (2008:1).

Other important influences on the material properties of concrete as a function of time, next to concrete type are the load level, heating/cooling rate or thermal cycle, moisture condition and whether testing occurred ‘hot’ or after cooling. Fire tests concerning the behaviour of concrete exposed to fire are therefore highly dependent on a very high number of parameters. Caution should thus be made when comparing results from different tests. Over the years, many of these studies have been performed, usually with respect to certain predetermined heating regimes that may not be representative of a realistic fire (Khoury, 2000; Handoo
et al., 2002; Husem, 2006; and Scharlaken and Sucaet, 2010). For example, the specimens are heated very slowly (1 or 2°C per min) or in several intervals of, for example, 100°C where this temperature is maintained for a certain amount of time. In an actual fire situation, heating rates are typically 20 to 30°C per minute according to Khoury and Anderberg (2000). They are nevertheless used, and recommended by RILEM, to separate the actual material from the structural effects due to the heating of small specimens (e.g. cores of 6 cm in diameter and 18 cm tall).

3.3 Spalling

According to Khoury and Anderberg (2000), spalling, in its most general form, is defined as the violent or non-violent breaking off of layers or pieces of concrete from the surface of a structural element when it is exposed to high and rapidly rising temperatures as experienced in fires. Thermal spalling is one of the most complex and hence poorly understood phenomena occurring in concrete exposed to high temperatures. Since decades, research has been conducted as to what factors trigger spalling, what influences the severity and ultimately how spalling works. Up till today, the underlying mechanism is still not fully understood, intriguing many scientists. Thermal spalling was recently brought back to the scene because of the new developments considering high strength concrete, which have shown to have higher susceptibility to spalling during a fire than normal strength concrete. Also several severe tunnel fires in Europe have highlighted the phenomenon. Consequently, the fire resistance of new concrete types must be reconsidered.

3.3.1 Types of spalling

Gary was probably one of the first researchers to systematically approach spalling in 1916 as part of his study on the effects of fire on concrete houses. Based on his test results, he was able to identify four different types of spalling (Jansson, 2008):

- Aggregate spalling – crater formed spalling producing a popping sound
- Surface spalling – disc shaped violent flaking, especially in pressure stressed walls, producing a cracking sound. Surface spalling can also be seen in Figure 3.2.
- Corner spalling – first seen as violent, by later researchers described as non-violent
- Explosive spalling – very violent spalling with a loud bang
After Gary, most researches adopted this categorisation, possibly with some alterations. For example Khoury (2008:1) mentions two more types of spalling: sloughing-off, occurring when the concrete strength is too low to carry its own weight, and post-cooling spalling, occurring during and after cooling upon absorption of moisture. He describes both types as non-violent of nature but with a possible serious influence. This wide range of types results in many different observations of spalling, in the most varied circumstances. Each type is furthermore influenced in different ways, adding to the complexity of the subject. According to Khoury and Anderberg (2000) aggregate, surface and explosive spalling occurs fairly soon, that is after 7 to 30 minutes of fire exposure. Corner spalling occurs later, when fire has weakened the concrete, at 30 to 90 minutes of heating.

This text will mainly focus on the type of spalling that is to be feared the most and consequently has been researched the most: explosive spalling. According to

Figure 3.2: Surface spalling after a fire in a car park. Photograph taken by Robert Jansson.
Khoury and Anderberg (2000), Gary (1916) observed bursting of entire surfaces of wall slabs up to 1 m² in his test building. Some parts of slabs were thrown 12 meters by the force, illustrating that explosive spalling can be highly dangerous. Many other authors regard explosive spalling as the “main” form of spalling and commonly refer to it as simply spalling. The term spalling can thus have two meanings, either spalling in general or the specific type of explosive spalling. Here, Sections 3.3.3, 3.3.4 and 3.3.5 are solely about explosive spalling and references to the term spalling mean explosive spalling.

3.3.2 Significance
The extent of spalling can vary greatly. Damage can be very superficial but can also have severe consequences, and ultimately structural collapse. Spalling reduces the cross-section of an element, causing higher stresses in the remaining area of concrete. Note also that compressive elements, e.g. columns, could experience premature collapse due to buckling. Furthermore, spalling can significantly reduce or even eliminate the protective concrete cover on the reinforcement steel or tendons. The steel will encounter much higher temperatures, reducing its strength and thus the strength of the concrete structure as a whole. Other than the reduction of the load-bearing capacity of a structure, spalling can also affect its separating function by causing holes through slabs or panels enabling the spread of fire. According to Khoury and Anderberg (2000), thin slabs are particularly susceptible to such “integrity” failure. Furthermore, when a piece of concrete breaks off, it could be thrown away with high speed, possibly damaging life or adjacent structures.

Note that the specific application of the concrete also determines the severity of damage by spalling. For example, aggregate spalling may in most cases be a harmless surface damage, it can have big consequences on concrete pavements used for military aircrafts.

3.3.3 Factors influencing explosive spalling
A multitude of factors influence explosive spalling, that have been identified through extensive testing over many years. The factors can be material, geometry, structurally or environmentally based. Contradictions in the reports from different authors are not uncommon. This could be due to the complexity of the subject, the many parameters that influence explosive spalling, and the fact that specimens and conditions vary markedly from test to test. Furthermore, explosive spalling is a stochastic process (Khoury and Anderberg, 2000). For specimen
from the same batch, treated and tested in the same way, some could spall and some could not. Below, an attempt is made to find a general consensus of opinion. Today, most researchers do agree on the following factors, however, as to why these affect explosive spalling is still under discussion. A summary of the different factors and their influence (Majorana et al., 2010 and Khoury and Anderberg, 2000):

**Heating rate** greatly influences the occurrence of explosive spalling. The probability and severity increase with increase in heating rate. However, when a concrete element does spall, it will in a certain temperature interval, independent on heating rate.

**Heating exposure**: The more faces of a member are exposed to fire, the more likely spalling is to occur. For example, slabs respond generally better to spalling (one face exposure) than beams (3 to 4 face exposure). For the same reasoning are simple external shapes without pronounced projecting features preferred.

**Section size**: Very thin members have a lower probability to spalling. It is understood that this is caused by the moisture that tends to escape more readily, reducing the pore pressures. Oppositely, experiments suggest that explosions are less likely in thick sections greater than about 200-300 mm (e.g. the walls of a nuclear containment).

**Section shape**: ‘Rapidly’ changing cross-sections encourage explosive spalling. For example, corners have an increased susceptibility, especially acute-angled corners. Plain surfaces and rounded corners exhibit the best behaviour.

**Moisture content**: Generally, explosive spalling is possible in normal strength concrete with a moisture content of more than 2% by weight (5% by volume). When there is less than 2% by weight moisture present, spalling is unlikely but this moisture content is difficult to reach in practice. For a given set of conditions, explosive spalling is less likely for concretes with moisture contents less than 3% by weight. However, very dense high strength concrete has experienced spalling with much lower moisture contents (2.3 to 3% by weight). It is believed that this is caused by the low porosity and permeability, making it more difficult for moisture to escape, which in its turn causes higher pore pressures, increasing the risk of spalling, even despite the higher tensile stresses.

It is known that moisture content of concrete decreases by age. The average moisture content of concretes in buildings was found to be about 3% by weight.
two years after construction, indicating that the probability of spalling would be small after this time. Furthermore, the moisture content of a concrete element is very much dependent on it environment and the climate in which it resides. Compare, for example, concrete elements inside buildings to car parks or tunnels.

**Permeability** highly affects the rate of vapour release. Experimental evidence has suggested that spalling is unlikely for a concrete with less than $5 \times 10^{-11} \text{ cm}^2$ permeability. High strength concrete, under hand, has very low permeability and a marked spalling tendency.

**Age of concrete**: The effect of the age of concrete on explosive spalling has been studied but the findings are conflicting. The majority of the reports, however, find that the risk of spalling reduces with increasing age. This might be related to the moisture content.

**Strength of concrete**: Ironically, concrete of poor quality is barely susceptible to spalling, making it a ‘good quality’ concrete for this effect. Whereas spalling in high strength concrete is a common issue. As said before, this is likely due to its increased permeability. The question also arises whether the increased strength of high-strength concrete may have a positive effect.

**Compressive stress and restraint** on a member increases its probability to spall. An increase in compressive stress, either by reduction in section size or an increase in loading, encourages explosive spalling.

**Type of aggregate**: experimental data considering the type of aggregate is sometimes inconsistent. Generally it can be noted that the likelihood for spalling decreases when low thermal expansion aggregates are used. In ascending order, susceptibility to spalling increases when concrete includes: lightweight, basalt, limestone, siliceous, Thames River gravel. However, this only applies for concrete with relatively dry aggregates, since it has been shown that lightweight aggregate concrete has a high susceptibility to spalling if the aggregate is saturated.

**Aggregate size**: Fire tests show that the greater the size of the aggregate, the more likely explosive spalling is to occur.

**Cracking**: The presence of crack is thought to have a dual affect. On the one hand, crack could facilitate moisture migration. On the other hand, cracks could serve as a starting point for crack propagation.
**Reinforcement:** Usually, explosive spalling is limited to the unreinforced part of the concrete section and doesn’t extend beyond the reinforcing layer, e.g. mesh reinforcement in a slab or a cage of bars and links in a beam or column. Caution should be made that reinforcement can become exposed due to spalling. For example, when placing the main reinforcement in corners, it should be noted that corners experience quicker heating than flat surfaces. Therefore, perhaps not all main reinforcement should be placed in the corners or, where possible, only nominal reinforcement should be placed there where the principal steel is located farther inwards.

**Cover to reinforcement:** Tests suggest that the bigger the concrete cover, the bigger the probability for spalling. Once spalling occurs and the reinforcement is exposed, the further behaviour is independent of the original cover. It is found that, if the nominal cover (i.e. the cover to the outermost steel) is bigger than 40 mm for dense or 50 mm for lightweight aggregates concrete, spalling must be feared. Concrete cover thicknesses of 15 mm or less seem less prone to serious spalling, probably because the mass of unsupported concrete is small.

**Supplementary reinforcement** seems not to hinder the event of explosive spalling, but it does limit the damage done. A light mesh is sometimes used to limit the effect of spalling when the concrete cover exceeds 40 mm. However, supplementary reinforcement is difficult to place in thin sections, such as ribbed floors. Furthermore, supplementary reinforcement makes the concrete easier to repair after the fire.

**Additions:** Researchers have attempted to include various additions in the concrete mix in order to improve the behaviour of the concrete towards spalling. This could be steel fibres, polypropylene fibres or the entrainment of air (see also further).

### 3.3.4 Explosive spalling theories

Today, no theory has been able to correctly predict the explosive spalling of concrete. Several hypotheses exist that can vary greatly and researchers commonly contradict each other, illustrating the complexity of the subject. Furthermore, every theory needs to be validated with extensive fire testing, which naturally needs investment. Developing a certain test method is already a challenge in itself. For example, from the moment a sensor is placed, it could change the behav-
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The behaviour of the concrete. Tests should also be performed during the fire exposure and are thus non-destructive. Residual test after cooling wouldn’t necessarily pick up all the hydro-thermo-mechanical transformations that take place dynamically during a fire. The cooling down phase even alters the structure of the concrete. A discussion of testing methods is beyond the scope of this work but it should be noted that this is not a self-evident matter.

Many researchers find that the mechanism of spalling involves pore pressure, thermal stresses, or a combination of both. This is also the case for Khoury. He describes the following effects in many of his works, e.g. Khoury and Anderberg (2000), Khoury (2008:1), Khoury (2008:3), and the international research project NewCon.

**Pore pressure spalling**

Pore pressure is basically a result of the build up of vapour pressures in heated concrete. It is mainly influenced by the permeability of the concrete, the initial water saturation (pore filling) level, the rate of heating and tensile strength of the concrete, along with the section size. The difficulty here is how to predict the generation of pore pressure, both experimentally and theoretically. There are several models describing pore pressure spalling which vary in complexity, from the simple use of steam tables to full solutions of the equations of state using finite-element analysis. There are also authors who believe spalling can be caused by the hydraulic pressure of a saturated pore (100% of water filling).

**Thermal stress spalling**

Thermal spalling results from restrained thermal expansion, due to rapid heating ranges and the low thermal conductivity of the concrete. The heated surface wants to expand but is obstructed by the cooler inner region, resulting in compressive stresses. In its turn, the cooler inner region experiences tensile stresses. Thermal stresses can perfectly occur in dry concrete, and even in ceramics when exposed to thermal shock, illustrating that pore pressure plays a minor role here. Generally, the factors influencing thermal stress are the thermal expansion of the aggregate, the level of applied load, the heating rate and tensile strength of the concrete. There exist also theories considering parasitic thermal stresses which do not rely on thermal gradients but on the differential thermal expansion of the constituents making up the concrete, most known between the cement paste and the aggregates but also between added fibres and the cement paste.
Combined thermal stress and pore pressure spalling
Khoury believes that, in general, explosive spalling occurs due to the combined action of pore pressure spalling and thermal stress spalling. The exact mechanism and the relative contributions of both effects is still a big question mark. How these effects work together could also vary, depending on factors as sections size, concrete compositions and moisture content. For example, perhaps in small specimens with high moisture content, pore pressure spalling could have an upper hand. One of the key roles of the research in this matter is the pore pressures. Many fire tests have already been performed, however, reporting contradicting values of measured pore pressure. Some measure pore pressures exceeding the concrete tensile stress, others find values which would not be sufficient for solely pore pressure spalling. For example, Jansson (2008) measured maximum pressure immediately prior to spalling, 10 cm from the exposed surface. He found a value of 1,1 MPa in small scale slab tests and 0,3 MPa in larger specimens of the same concrete under a slightly higher thermal exposure, both too small to trigger spalling according to simple pore pressure spalling theories, although in reality spalling did occur. Pore pressure spalling theories have, however, not been ruled out. The tests performed by the different researchers are all performed under different circumstances (heating rate, load type, concrete type, etc.). The measurement of the pressures has also been performed using different test methods, which in itself is already difficult to achieve in practice. Even if pore pressure spalling is not the main reason for explosive spalling, it could still serve as an important trigger. Altogether, these, and many other questions, concerning the mechanism of explosive spalling are still in need of answering. Several researchers are, however, dedicated and research is certainly on its way.

3.3.5 Design against explosive spalling
Aside from the effect of spalling, concrete structures can withstand fire fairly well. Mostly, very simple calculation methods are sufficient to check the resistance to fire and many structures barely need extra measurements on top of the normal temperature design. However, it can easily be seen that, when unexpectedly spalling occurs, these fire safety design calculations can become unsafe very quickly. Unfortunately, today, the event of spalling and its severity remains difficult to predict, forming a limited factor in developing robust models for concrete structures subjected to fire. Fire engineering calculations or computer simulations simply won’t be reliable.
Several measurements are known to reduce a member's susceptibility to spalling or the damage done. Their positive effect has been experimentally confirmed, however their effectiveness and feasibility do also play a role. One solution that can be found in the literature is the use of a thermal barrier. It is applied as some sort of coating on the fire exposed sides of the concrete elements. Its benefits are clear, by limiting the flow of heat to the concrete it reduces any effect the fire has on the element, i.e. spalling but also for example the reduction of strength. Possibly the biggest downside of this measurement is its price. Other solutions are, among others, employing an air-entrainment agent (but then strength is reduced), reducing moisture content (difficult in for example outdoor constructions), reducing the permeability (and thus reducing the durability), use of a low thermal expansion aggregate (to reduce thermal stresses), applying thicker sections (important for I-beams and ribbed sections) or adding steel fibres (test results have for now been inconclusive) (Khoury, 2008:1). An appropriate use of reinforcement could limit the damage by spalling but not reduce the probability of occurrence. The most well known solution, though, is without a doubt, the inclusion of polypropylene fibres in the concrete mix.

**Polypropylene fibres**
Since about 1992, polypropylene fibres (PP-fibres) have been included in the cement matrix to combat the effects of explosive spalling and today, it still forms a ‘hot’ topic. Its positive effects are well-established and they are even used in many new tunnels all over Europe today, as for example the City Tunnel in Malmö in Sweden (opened on 4 December 2010) or the two extra bores for the Liefkenshoektunnel in Antwerpen in Belgium (work began in September 2008, completion scheduled for 2014). However, the processes underlying the behaviour of polypropylene-concrete composite and its positive effect on explosive spalling are not fully understood. Is this even possible if even the mechanism of explosive spalling is not yet known? Many researchers are now working on guidelines that determine when PP-fibres should be applies, which type performs the best, what dimensions should be used, etc., but as long as its mechanisms are not fully understood this won’t be much more than simple guesswork and extensive fire testing. Understanding the role of PP-fibres is key in optimizing its design in concrete.

Several theories exist as to how PP-fibres limit the explosive spalling of concrete exposed to fire. According to Jansson (2008), Schneider and Horvath (2003) summarized the different theories that are presently discussed in the literature.
The theories are based on an improvement of the permeability of the concrete due to:

- the formation of capillary pores when the fibres melt and burn
- the development of diffusion open transition zones near the fibres
- additional micro pores, which develop during the addition and mixing of fibres in the concrete mix
- additional micro cracks at the tip of the PP-fibres which develops during heating and melting

All these theories assume that, due to the increased permeability, moisture transport is facilitated.
4 The concrete structure and fire

4.1 Effects of fire on the structural member

The different effects that occur in concrete structures when exposed to fire are briefly discussed here, based on the information in the fib Bulletin 46 (2008). A general understanding of these effects is necessary to produce a good design. Naturally this is built upon how the material concrete behaves under high temperatures, which is described in Chapter 3.

Firstly, the material characteristics of the members are modified when the temperature rises. The strength as well as the stiffness of both the concrete and the steel is reduced. In fact, even the whole stress-strain diagram is modified. The thermal properties, as thermal conductivity and specific heat, are also altered by a thermal elevation. However, these changes are not particularly relevant for the reinforcement since its amount is generally too low to affect the overall temperature distribution. All these effects can be seen in Section 6.8, where the variation of the material properties of concrete and steel with temperature are given in accordance with the Eurocode. Not directly covered by the Eurocode is the bond strength between concrete and steel which also reduces with increase in temperature. However, in practice, failures by debonding in reinforced structures are very rare. The problem is more critical for prestressed elements.

One very important material property that needs more attention is the thermal expansion of concrete and steel, which increases with temperature. The order of magnitude may reach 1% at high temperatures in the range of 800°C. This means that a beam of 5 m long could extent more than 5 cm. These large displacements play a big role in the behaviour of concrete structures. Several effects are to be observed. The first effect is the “average” thermal elongation on a section, as in the example of the beam. This elongation depends on the type of section. Sections that have many exposed surfaces, like columns, beams or T-beams, will exhibit a greater thermal elongation as sections exposed on only one side, like walls or slabs. The second effect is thermal lateral deflections or bowing due to an average thermal gradient over the section. This means that columns heated on four sides, for example, will not experience thermal bowing. Beams heated on three sides show downward thermal bowing. The extent of the bowing depends on how big the average gradient is. Thus, flat slabs and walls heated on one side will
experience the greatest deflections. A T-section exhibits very interesting behaviour since it is essentially made up of two parts, a slab and a beam that is exposed on three sides. Two parts that would have significant different lateral deflections if they would be alone. In an experimental test performed on a specific T-section (fib Bulletin 46, 2008), sever horizontal cracks appeared at the junction between the slab and the web and ultimately, the two parts were separated. Both effects, thermal elongation and thermal lateral deflections will generate important geometrical second order effects, or indirect effects of actions if the expansion is constrained. The former is not usually classified as an internal action, but can have significant consequences, for example in free cantilever walls exposed on one side. Furthermore, the temperature profile inside concrete elements is everything but uniform, causing internal thermal stresses.

Finally, the phenomenon of spalling can cause significant damage to concrete structures. This effect is discussed in Chapter 3. An important note is that spalling is known to be influenced by structural factors as restraint or loading.

4.2 The effect of structural assembly

Here, it is described how the structural behaviour of a member exposed to fire can be, in most cases, enhanced by consideration of the whole structural assembly. This information is based on Buchanan (2002). Note that these effects can be applicable for all types of building materials, however, here, special attention is paid to the behaviour of concrete structures.

4.2.1 Redundancy

Redundancy is the effect where the failure of a single element does not cause failure of the whole structure. This is achieved through load sharing, the loads that were carried by the failed member are redistributed to other, stiffer, and stronger members. This may remind of continuity, which is discussed later, however, in continuous members the load is redistributed within that member. In the case of redundancy, the internal forces are carried on from member to member. The efficiency of this effect depends on the load factor. This term is explained later, in Section 6.7.2, however basically this means that, in the accidental situation of a fire, the total load on the structure is much less than the full design load. Thus fewer members may be necessary to carry the structure, provided that alternative load paths are available so that the load can get to the undamaged members. Ductility is also needed, making this effect more known in steel struc-
tures, however it can occur in concrete structures as well, especially when the reinforcement provides ductility to the members.

4.2.2 Disproportionate collapse
Disproportional collapse is conceptually the opposite of redundancy, meaning that the failure of one member could cause the failure of a whole structure or part of that structure. The magnitude of the damage is disproportionate to the initial event. A well known example of disproportionate collapse, also called the lack of robustness, is the Ronan Point disaster in the U.K. in 1968. An explosion in one room of the multi-storey building caused a whole section of the building to collapse, with the loss of many lives. Disproportionate collapse can also occur when elements providing lateral restraint to slabs or beams or lost. Design against disproportional collapse requires structural toughness and alternative load paths.

4.2.3 Continuity
A member with flexural continuity possesses a much greater fire resistance than if simply supported. Flexural continuity essentially means that the member is hyper static, meaning that more than one cross-section may fail before its load carrying capability is lost. Examples are continuous beams over several supports or beams built in a rigid frame, i.e. clamped support conditions. A simply supported beam, on the other hand, has no flexural continuity. The mechanism is based on the formation of plastic hinges. These are segments of the beam where large displacements can occur without significant increase in bending moment. The internal forces that were previously carried by these segments are then redistributed to the rest of the beam which is still intact. The large displacements in the plastic hinges require ductile material, as steel members and reinforced concrete. The latter material can even show additional benefits due to different positive and negative bending moment capacities. An example of moment redistribution is given in Section 7.3, where the load-bearing capacity of a reinforced T-beam, continuous over 2 supports, exposed to a standard fire is checked.

4.2.4 Axial restraint
Restraint to axial expansion of concrete members can have a significant influence on its behaviour in fire, positive or negative. Its effects may even be so strong, that it can overshadow other effects as steel cover, size and shape of the member, aggregate type, reinforcement type and load intensity (Fellinger and Breunese, 2004). In order for this phenomenon to occur, the surrounding structure must be rigid enough to provide the restraining forces. Therefore, the effect is the greatest
in for example a localised fire where only a part of a floor or a building is heated, leaving a considerable amount of the remaining structure at normal temperatures.

The effect of axial restraint is explained by an example. In Figure 4.1, a beam is shown located between two rigid supports that allow rotation but no elongation at the ends. The bottom of the beam is exposed to fire. As it heats up, it will try to expand, however this is not allowed by the supports. Thus, an axial thrust force $T$ is developed. Its effect can be best compared to that of an external prestressing. Due to the eccentricity $e$ between the line of action of the thermal thrust and the centroid of the compression block near the top of the beam, an additional moment $T \cdot e$ arises which may help carrying the external load. In fact, the flexural capacity of the beam becomes $M_{R,\text{total}} = M_{R,\text{fire}} + T \cdot e$ where $M_{R,\text{fire}}$ is the moment capacity at elevated temperature. It is even possible, provided that the surrounding structure is stiff enough, that the elevated temperature moment $M_{R,\text{fire}}$ can drop to zero without structural failure. However, the effect of axial restraint can also be negative. If the line of action of the axial thrust forces develops near the top of the beam, the eccentricity $e$ will be negative, thus diminishing the total flexural capacity. Consequently, supports should be adequately designed and executed. Note that, due to this high dependency on the support detail, axial restraint may only be relied upon when explicitly designed for. However, this design can prove to be challenging in some cases, since the position of the axial thrust can vary as the deflection or rotation of the beam varies during the fire exposure. Furthermore, note that axial restraint may also lead to additional failure modes that should be taken in account in the design, for example shear failure of columns and walls or buckling of the beam.

![Figure 4.1: Beam between two rigid supports and the axial restraint thrust forces. Redrawn from Buchanan (2002).](image-url)
Chapter 5. The Eurocodes, background and implementation

5 The Eurocodes, background and implementation

5.1 Introduction

Part of this work is to describe how to structurally design a concrete structure for fire safety. This is done by using design standards and nowadays that is without a doubt the European standards or the Eurocodes (EN), the leading way in structural codes. These apply to all the Member States of CEN (Comité Européen de Normalisation, or European committee for standardization) which are the National Standards Organizations (NSO’s) of the 27 European Union countries and Croatia plus three countries of the European Free Trade Association (Iceland, Norway and Switzerland) (CEN, 2011).

5.2 Contents of the Eurocode

The Structural Eurocodes (EN 1991-1-2, 2002) were created by the Commission of the European Community (CEC) for the development of the European single market, as a set of unified technical rules for the design of buildings and other civil engineering works and construction products that would replace the, sometimes contradictory, rules in the various Member States. In other words, the goal was to create some sort of transnational unity. The commission transferred the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard.

The Eurocodes cover in a comprehensive manner all principal construction materials (concrete, steel, timber, masonry and aluminium), all major fields of structural engineering (basis of structural design, loading, fire, geotechnics, earthquake, etc.) and a wide range of types of structures and products (buildings, bridges, towers and masts, silos, etc.). There are ten different standards, each 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
The directions for fire resistance are not specified in a special Eurocode, but in a special section, Part 1.2, in each Eurocode (except for Eurocode 0, 7 and 8).

The verification procedure in the Eurocodes is based on the limit state concept used in conjunction with partial safety factors. The Eurocodes allow also for design based on probabilistic methods as well as for design assisted by testing, and provide guidance for the use of these methods.

5.3 Primary objectives

The Eurocode is designed to provide design criteria and methods that can be used to meet necessary requirements for mechanical resistance, stability and also resistance to fire, including aspects of durability and economy (Guidance Paper L, 2003). The aim is to create a common understanding between all the parties involved in the design and construction of structures: owners, operators and users, designers, contractors and manufacturers of construction products. As a consequence, the marketing and use of structural components and kits or of materials and constituent products is facilitated, in the Member States but also far beyond. The Eurocodes aim to be a common basis for research and development, in the construction industry and to allow development of common design aids and software. The competitiveness of the European civil engineering firms, contractors, designers and product manufacturers in their global activities is instigated.

5.4 Brief history

This Section gives a short history to help understand the process, that took over 30 years, of how the Eurocodes came about (Guidance Paper L, 2003).

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 structural design of construction works which, in the first stage, would serve as
an alternative to the national rules in force in the Member States and, ultimately, would replace them.

mid 80’s For fifteen years, the Commission, with the help of a Steering Committee containing Representatives of Member States, conducted the development of the Eurocodes programme, which led to the publication of a set of first generation European codes in the 80’s.

1989 The Commission and the Member States decided, on the basis of an agreement with CEN, endorsed by the SCC, to transfer the preparation and the publication of the Eurocodes to CEN through a Mandate, in order that they would, in the future, have the status of European Standards.

1992-1998 Originally, the Eurocodes were elaborated by CEN as 62 pre-standards (ENVs). Most were published between 1992 and 1998, but, due to difficulties in harmonizing all the aspects of the calculation methods, the ENV Eurocodes included “boxed values” which allowed Members States to choose other values for use on their territory. National Application Documents, which gave the details of how to apply ENV Eurocodes in Member States were, generally, issued with a country’s ENV.

1998 The conversion of ENVs into European standards started in 1998.

2007 All 58 different Parts were published by the CEN.

2010 By 2010 the Eurocodes were expected to be fully implemented and to have replaced all national rules, though many countries (e.g. Belgium) are still in a period of co-existence.

The work is, however, not finished. The Eurocode is at this moment the state-of-the-art standard and this must, of course, be kept this way. The CEN/TC250 committee will keep maintaining and developing the Eurocodes. They are responsible of matters such as the correction of errors, technical and editorial improvements, technical amendments with regard to urgent matters of health and safety, resolution of questions of interpretation and elimination of inconsistencies and misleading statements. They will also approve any corrigendum (e.g. removal of printing and linguistic errors) or amendment (e.g. modification, addition or deletion of specific parts), as appropriate. In addition, future editions of the Euro-
codes, such as new Annexes or Parts and eventually new Eurocodes will be needed to include guidance reflecting new European Union policies, innovative design methods, construction techniques, new materials, products and the like.

5.5 Field of application

The Eurocode serves as a reference document in the following three cases (EN 1992-1-2, 2004; Memento 2003, 2003 and Guidance Paper L, 2003). Firstly, the Eurocodes can be used as means to meet the essential requirements of building and civil engineering works. They can also serve as a basis for specifying contracts for construction works and related engineering services. Thirdly, the Eurocodes serve as a reference framework for drawing up harmonised technical specifications for construction products. In approving the mandate to CEN to prepare the EN Eurocodes, the Member States have recognised the Eurocodes as an acceptable solution to achieve the previous cases. Alternative design may be used but must be proven to be technically equivalent to an EN Eurocode solution.

5.6 National implementation

Information in this Section was based on the Joint Research Centre (2008) and the foreword of the different Eurocode Parts.

5.6.1 Timeline

When the CEN makes a Eurocode Part available (Date of Availability), there is still a lot of work cut out for the National Authorities and National Standards Bodies. The National Authorities and National Standards Body should:

- translate the Eurocode Part in authorised national languages
- set the Nationally Determined Parameters (explained in 5.6.3) to be applied on their territory
- publish the National Standard transposing the EN Eurocode Part and the National Annex
- adapt their National Provisions so that the EN Eurocode Part can be used on their territory
- promote training on the Eurocodes

The timeline is illustrated in Figure 5.1.
The implementation consists of three phases:

The first period consist of the translation of the Eurocode Part in authorised national languages. This may be started by the National Standards Bodies at the latest at the Date of Availability and may take up maximum one year.

In the national calibration period, the Member States fix the Nationally Determined Parameters. At the end of this period, the national version of the EN Eurocode Part with the National Annex will be published by the National Standards Bodies. Also, the Member States should adapt the National Provisions so that the Eurocode Part can be used on their territory. This period may take up maximum 2 years.

Hereafter comes the coexistence period. Here, both the Eurocode Part as well as the existing national system can be used. The length of the coexisting period is actually determined by the whole Eurocode Package, and may last up to a maximum time of three years after the national publication of the last Part of the Package. Member States must make sure that all the Parts of the related Package can be used without ambiguity on their territory by adapting their National Provisions as necessary.

Thus, all conflicting National Standards in a Package should be withdrawn a maximum of 5 years after the Date of Availability of the last available Part in the Package.
5.6.2 National Standards
The National Standard transposing the EN Eurocode Part is published by a National Standards Body on behalf of, and with the agreement of the competent National Authorities. It is composed of the EN Eurocode text preceded by the National Title page and by the National Foreword and generally followed by the National Annex, as illustrated in Figure 5.2.

Figure 5.2: The national publication of a EN Eurocode Part. Redrawn from Joint Research Centre (2008).

The National Annex allows a country to choose unique parameters for the design rules, for example to set a certain safety level or to adjust the rules to fit local conditions. The National Annex may contain directly, or by reference to specific provisions, information on the Nationally Determined Parameters (see 5.6.3). It may also contain decisions on the application of informative Annexes and reference to non-contradictory complementary information intended to assist the user in the application of the Eurocode.

Although most Member States make use of the National Annex, it is not a CEN requirement. It has an informative status. A National Annex is not necessary if a Eurocode Part contains no choice open for Nationally Determined Parameters, or if a Eurocode Part is not relevant for the Member State (e.g. seismic design for some countries) or if a Member state chooses to adopt the recommended values provided in a Eurocode Part.
5.6.3 Nationally Determined Parameters

To acknowledge the responsibility of regulatory authorities in each Member State, due to difficulties in harmonizing all the aspect of the calculation methods and to accommodate different levels of safety, Nationally Determined Parameters (NDP) were included in the Eurocode. For example, the Nationally Determined Parameters account for possible differences in geographical or climatic conditions, or in ways of life, as well as different levels of protection that may prevail at national, regional or local level. The NDP’s are published in the National Annex.

In case of the European pre-standards ENV, the term “boxed values” were used and they were specified, among others, in the National Application Documents.

National Determined Parameters can be:

- 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. a snow map
- the procedure to be used where alternative procedures are given in the Eurocode

For most Nationally Determined parameters, the Eurocode itself already provides a recommended value. The European commission urges the Member States to use these recommended values unless geographical, geological or climatic conditions or specific levels of protection make that necessary. Furthermore, a Member State does not necessarily have to choose any NDP’s. In this case the choice of NDP’s will be the responsibility of the designer, if he chooses to deviate from the recommended values, whom must take into account the conditions of the project and the National provisions.

5.6.4 Status of the Eurocode

Belgium

In Belgium (Memento 2003), all the standards are published by the NBN (bureau for nationalisation). In itself, these standards are not compulsory. Nevertheless, they are juridical considered proper workmanship for the application of the ten-year-liability of designers (architects and engineers). By following the standards, technical quality is presumed, by deviating, the need of technical justification arises.
Standards are however compulsory when referred to by the building plan or an equivalent document, like in an open tender or in certain private contracts. In this case, it is sufficient to mention the reference to the standards and if necessary the publication datum. Standard can also be made mandatory in the adaptation of a regulation which is by definition mandatory. An example of a regulation is a Royal Order (in dutch: Koninklijk Besluit or KB) which is a federal governmental decree in the view of practically implementing the application as a law. Even without a reference in the building plan, this last act would impose that the construction industry should comply with the base standards. When, for instance, looking at the Eurocode 2, certain regulations would become obligatory (the principles, the main application rules, the minimal properties of the materials, etc.) while others, without being obligatory, determine the equivalent level.

At the time this text was written (August 2011), the state of the different standards are as follows. Most of the Belgian standards (NBN B XXXX) are cancelled, and all the Parts of the Eurocode have been published (NBN EN XXX), although not all of them have yet a National Annex, called ANB (Annexe Nationale, Nationale Bijlage). The guidelines for which standard to use are:

- Every Eurocode (EN) for which an ANB is published, becomes the reference standard in its domain. The Belgian standards and the European pre-standards (ENV) are cancelled.
- For a given Eurocode may, as long as the ANB is not published, the Belgian standards and the European pre-standards concerning this subject still be used. However, to stay up-to-date it is strongly recommended to use the ENV+DAN. DAN comes from the French “Document d'Application National” meaning National Application Documents.
- A standard is never mandatory in Belgium unless it is explicitly asked for in the building specifications. However, the standards that apply at the time of the project will serve as a reference in case of an incident.

The Belgian National Annex ANB was published for all the Eurocode Parts which are of importance for this thesis (mainly EN 1991-1-2 and 1992-1-2), thus for this specific field of design, i.e. fire design of concrete structures, only the Eurocode may be used in Belgium.

**Sweden**

In Sweden, building regulations are controlled by the Swedish National Board of Housing, Building and Planning, Boverket. It is the central government author-
ity for town and country planning, management of land and water resources, building and housing. In the field of building, Boverket is responsible for developing design and building regulations and other regulative measures for construction, e.g. for certification of persons, Swedish type approval and CE-marking as well as implementation measures concerning EC directives. The Board supports the development of cost and energy efficient, healthy and sustainable buildings as well as accessible public spaces.

In the built environment Boverket publishes two kinds of regulations. The building regulations BBR, abbreviated from Boverkets byggregler, (Boverket, 2011:1) and the design rules EKS, abbreviated from Europeiska konstruktionsstandarder, (Boverket, 2011:2) which exists out of the Swedish National Annexes. The previous design rules BKR, abbreviated from Boverkets Konstruktionsregler, (Boverket, 2000) were cancelled on the first of January 2011 and from the second of May 2011 all developers may only apply the Eurocodes and associated EKS for the design of structures in Sweden.

Europe and world
The status of the Eurocode in Europe is difficult to summarize since the information is constantly changing. It is advised to always check directly with the local authorities what the legal situation is for a country. Although the Eurocodes were expected to be fully implemented and to have replaced all national rules by 2010, not all countries have achieved this. Countries that have, are for example Austria, Czech Republic, Ireland, Romania, Slovakia, Slovenia and Sweden. Other countries still choose to publish both codes in co-existence. However, since it is not expected that the National Standardisation Bodies maintain the withdrawn National Standards in practice, there will be little option but to use the EN Eurocodes. It is also very likely that several involved instances, e.g. international clients and contractors as well as other stakeholders like the insurance industry, will perform pressure, speeding up the implementation of the EN Eurocodes. There are also countries that are on their way to full implementation but have not yet managed to publish all National Annexes, as for example Belgium.

Not only in Europe is the Eurocode the state-of-the-art, but they are also recognised as high-quality design codes in many other countries over the world. Due to the system with National Annexes, they are flexible and easy to be adopted in any country worldwide. Several parties have promoted Eurocodes globally and many countries are already committed to adopting Eurocodes.
6 Structural fire design of concrete structures

6.1 Introduction

As was put forth in the previous Chapter, the Eurocode is the state-of-the-art in structural standards. If it has not yet replaced the old national standards in a Member State, it soon will. Thus, this Chapter studies how a concrete structure should be designed to withstand the effects of fire, according to the Eurocode. The two Parts that will be used frequently in this text are: “EN 1991-1-2: Eurocode 1: Actions on structures – Part 1-2: General actions – Actions on structures exposed to fire, published in November 2002” and “EN 1992-2-2: Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design, published in December 2004”. The first is general and applies to all types of materials, whereas the second is for concrete structures. In addition, these Parts are designed to be used together with other Parts of the Eurocode. When information from another Part is used, this will be clearly stated in the text.

The breakdown of the Eurocode in different Parts is definitely a necessity, considering how much it contains, but it does also diffuse all the information. This may be confusing, especially when only looking for information about a specific topic, as for example the fire design of concrete structures. As an illustration, the fire resistance criteria of reinforced concrete, i.e. integrity, insulation and load-bearing function (explained in Section 6.3), are described in EN 1992-1-2. Their definitions can be found in EN 1991-1-2 and their determination methods in EN 1363. The goal was here to collect all the information needed to determine and understand the fire resistance of a concrete structure, independent of where this information may be found. This text does not literally copy all the design rules given in the Eurocode, but serves as a comprehensible summary and a guide to the use of the Eurocode. Although only concrete structures are studied, many of the matters described are still applicable in a more general way and for other materials, e.g. steel, masonry, etc. No previous knowledge of structural fire design is needed. However, this text does assume that the reader knows how to design a concrete structure for normal temperatures.

Due to the extensiveness of this Chapter, its content with respect to the different Sections is briefly summarized. First, the scope of this text is given, which is
Behaviour and structural design of concrete structures exposed to fire

identical to the scope of the relevant Eurocode Parts, since it is always important to know in which conditions the following design rules may be used. The objective of every fire design is to achieve an appropriate fire resistance. This concept is thus explained first in Section 6.3. In order to make a proper assessment of the fire resistance, a certain fire design strategy must be followed which is briefly discussed in Section 6.4. Hereafter follow several Sections that highlight different aspects of the fire design procedure: an overview of the alternative design procedures, thermal actions for the temperature analysis, mechanical actions for the structural analysis and the material properties of concrete and steel at elevated temperatures. Ultimately, the last step of the design strategy is explained, that is, the actual assessment of the fire resistance according to the different design procedures. Finally, Section 6.10 provides the additional rules that apply when designing structures of high strength concrete.

6.1.1 Terms used in the Eurocodes
Some terms that are often used in the Eurocodes are clarified (Narayan and Beeby, 2005).

Principles and Application Rules: The Eurocode Parts are divided into clauses which all start with their number in parentheses, i.e. (x). These clauses are either Principles or Application Rules. The Principles are defined as general statements, definitions, requirements and analytical models for which there is no alternative at all or no alternative is permitted in the Part. They are identified by the letter P after the clause number. Application Rules are generally recognised methods, which follow the Principles and satisfy their requirements. The use of alternative design methods are allowed, as long as it can be demonstrated that they comply with the Principles and are at least equivalent with regard to structural safety, serviceability and durability to the rules in the code.

Normative and Informative: The term normative is used for text in the Standards that forms the requirements. Informative is a term used in relation to the Annexes of a Eurocode Part which seek to inform rather than require. However, National Annexes may specify certain Part Annexes as normative or, oppositely, they may prohibit the use.
6.2 Scope

This text follows the Eurocode and thus its scope is identical to that of the Eurocode. It deals with the design of concrete structures and parts thereof for the accidental situation of fire, in terms of adequate load bearing resistance and limiting fire spread, e.g. flames, hot gases, excessive heat, as relevant. Normal temperature design is assumed to be known and mainly the differences from, or supplements to the latter are addressed. Furthermore, only specific aspects of passive fire protection are considered here, whereas active methods do not belong to the scope of this work. For example, requirements concerning sprinkler installations, occupancy of buildings and fire compartments or insulation and coating materials are not covered here.

The calculation methods are provided for all kinds of concrete: normal concrete up to strength class C90/105, lightweight concrete up to strength class LC55/60 and high strength concrete with strength classes above C50/60. Structures that are not covered are structures that are prestressed with external tendons and shell structures. Also, the evaluation of the residual strength after a fire is not covered.

6.3 Fire resistance

The concept of ‘fire resistance’ has been for decades at the heart of research, design and assessment of concrete structures exposed to fire. It captures how an element should withstand a fire. As stated before (Section 2.4), fire resistance can be defined as the ability of an element (not a material) to fulfil its designed function for a period of time in the event of a fire. This designed function can be a load-bearing, separating or load-bearing and separating function (KMO Normen-Antennes: Brandpreventie, 2011 and EN 1992-1-2, 2004). An element with a load-bearing function is an element that requires mechanical resistance in the case of fire, as with beams and columns. An element with a separating function prevents the propagation from a fire from one room to another, as with non-load-bearing walls. This is crucial when compartmentation is needed. Some elements can have both load-bearing and separating functions, as with a floor. The Eurocode then defines three performance criteria (EN 1991-1-2, 2002):

- **load-bearing capacity (R)** = ability of a structure or a member to sustain specified actions during the relevant fire, according to defined criteria
- **integrity (E)** = ability of a separating construction element, when exposed to fire on one side, to prevent the passage through it of flames and hot gases and to prevent the occurrence of flames on the unexposed side

- **insulation (I)** = ability of a separating element construction element, when exposed to fire on one side, to restrict the temperature rise of the unexposed face below specified levels

Criterion R must be fulfilled when load-bearing function is required. When an element has a separating function, criteria E and/or I must be fulfilled, where relevant. There exist many more criteria, but these are usually not required for the concrete structures considered here (see also later). Design is always done with respect to a certain design fire (discussed later) and, naturally, the prevailing criteria must be satisfied for the same duration as this fire. It should be noted, though, that the satisfaction of these criteria means that the element can withstand the design fire, but it does not mean that the element is not damaged, or needs to be replaced.

Construction works, members or products may also be classified with respect to their fire resistance (Commission decision of 3 May 2000, 2000 and Denoël, 2007). They are exposed to the standard fire curve and the duration for which this element meets its respective criteria is measured or calculated. A degree of resistance is then awarded to the member, consisting of one or more letters, R, E, I or a letter from another criteria, followed by a certain time span $t$ equal to or smaller then the duration for which this element can withstand the fire. The time span $t$ is, generally, chosen out of the values 15, 20, 30, 45, 60, 90, 120, 180, 240 and 360 minutes. Common codes are then: for load-bearing elements REI $t$, RE $t$ and R $t$ and for non-bearing elements EI $t$ and E $t$. For example a column that stays stable in a fire for an hour is classified as R60, a fire door that remains fire free for an half an hour is E30 and a wall that remains fire free for an hour is EI60. Accordingly, European product standards have been drawn up for each type of element, e.g. false ceilings, service ducts, façades, walls, valves, concrete hollow core slabs, etc.

In more general terms, required functions and levels of performance of a concrete structure exposed to fire may also be specified by referring to fire safety engineering for assessing passive and active measures. This is more suitable for performance-based design.
6.4 Fire design strategy

In order to check if a member has an appropriate fire resistance, the following process must be followed, consisting of five steps:

- A selection of the relevant design fire scenarios
- The determination of corresponding design fires
- The calculation of the temperature profiles within the structural members
- The calculation of the mechanical behaviour of the structure exposed to fire
- Verification of the fire resistance

Roughly, it can be said that EN 1991-1-2 is concerned with the first two above, while the fire parts of the material codes cover the remaining three, which in the case of concrete is EN 1992-1-2.

6.4.1 Step 1: Consider the relevant design fire scenario

The definition of a design fire scenario by Eurocode 1 is a “specific fire scenario on which an analysis will be conducted”. For instance, where does the fire occur? Is it a building fire, a tunnel fire, or a petrochemical fire? Or how severe is the fire? Perhaps it is a localised fire or a fully developed fire? The particular fire scenario should be chosen based on a fire risk assessment, taking into account the likely ignition sources and any fire detection/suppression systems available. A suitable size, occupancy and ventilation condition of the compartment is identified, representing the “reasonable worst case scenario”. This will then dictate the choice of the design fire to be used in the subsequent analysis.

6.4.2 Step 2: Choose an appropriate design fire

A design fire is defined by the Eurocode (EN 1991-1-2, 2002) as a “specified fire development assumed for design purposes”, in other words, a model that represents the action of a fire (see also 2.2). There exist two types of models, a nominal and a natural fire model, given respectively in Section 6.6.2 and Section 6.6.3. A nominal fire model consists of one simple relationship giving the temperature of the gases in the compartment as a function of time. They are easy to use and they are useful for the purpose of classification and comparison. For example the fire resistance of different construction works, members or product are classified with respect to the standard fire curve, the most common nominal fire curve (as seen in Section 6.3). However, these design fires bear no relationship to the specific characteristics of the considered building, like fire load, thermal properties of
compartment linings, ventilation condition, etc. On the other hand, natural fires are calculation techniques based on a consideration of these physical parameters specific to a particular building. More information about the different design fires can be found in the corresponding Sections. The chosen design fire should be applied only to one compartment of a building at a time, unless the fire design scenario states otherwise.

### 6.4.3 Step 3: Temperature analysis

After choosing a correct design fire, the net heat flux can be determined. In the temperature analysis, this net heat flux, together with the thermal material properties of the members and of any protective surfaces is used to determine the temperature profile inside the concrete members. The analysis is done by combining radiative and convective transfer of heat. Important is the location and orientation of the member in relation to the fire.

The period of time for which the temperature analysis should be conducted depends on the fire design type. In case of a nominal temperature-time curve, the analysis is performed for a limited period of time, without any cooling phase. This period of time may be specified in the national regulations of a country or can be obtained from Annex F of EN 1991-1-2 (2002), when allowed by the National Annex. In case of a natural fire model, the analysis is performed for the whole duration of the fire, including the cooling phase.

### 6.4.4 Step 4: Mechanical analysis

In this step, the combined effects of all the mechanical actions in the fire situation are determined. Which actions that should be taken into account and how to combine these is described in Section 6.7. The effects of actions must then be verified with the residual capacity of the structure, part of the structure or member, according to step 5. The mechanical analysis must be performed for the same duration of the fire exposure as used in the temperature analysis.

### 6.4.5 Step 5: Assessment of the fire resistance

**Verification of the load-bearing capacity**

The load-bearing function of a structure, a part of a structure or a member is verified in the strength domain, meaning that for the relevant duration of fire exposure $t$, the applied loads are less than the load capacity of the structure:

$$ E_{d,fi} \leq R_{d,t,fi} \quad (6.1) $$
Where:

- \( E_{d,f} \) is the design effect of actions for the fire situation, resulting from the mechanical analysis
- \( R_{d,t,f} \) is the corresponding design resistance in the fire situation

The fire exposure time \( t \) is the same as in the temperature analysis. The different methods to verify this equation are given in the next Section, Section 6.5. Note, that for the evaluation of the load-bearing capacity, it is worth assessing all modes of failure, such as rupture by flexion, by shearing load, by buckling or rupture of the anchoring of concrete element reinforcement.

**Verification of integrity and insulation**

Integrity and insulation criteria are verified using solely temperature analysis. Practically, the criterion I, is fulfilled when the average temperature rise over the complete non-exposed surface is limited to 140 °C, and the maximum local temperature rise in one point of that surface does not exceed 180 °C.

**Additional design requirements**

Apart from the fire resistance, the design may require additional criteria. There are for example also deformation criteria. These are usually not required, but they must be checked when the means of protection or the design criteria for separating elements require consideration of the deformation of the load bearing structure. They must not be checked when the efficiency of the means of protection has been obtained by the application of protective layers or when the separating elements have to fulfil requirements according to nominal fire exposure.

The design of joints should always be considered. Rules are given in the Eurocode which are described in Section 6.9.5

In some cases, an extra criterion M, i.e. impact resistance, is required for a vertical separating element (e.g. a fire wall) with or without load-bearing function. This element should then resist a horizontal concentrated load as specified in EN 1363 Part 2.
6.5 Alternative design procedures

6.5.1 Schematisation of the structure

The Eurocode allows the structure to be analyzed in three different manners (EN 1992-2-2 and Leonardo Da Vinci Pilot Project, 2005):

**Global structural analysis:** Here, the concrete structure is studied as a whole. All the interactions between the different elements are modelled, taking into account effects of thermal expansions and deformations. The possible significance of these effects have been identified in Chapter 4. Also the relevant failure mode under fire, the temperature dependent material properties (mechanical and thermal) and member stiffnesses are considered. Naturally, this is the most realistic approach, but it is also the most complex and impossible to execute without a computer.

**Analysis of part of the structure:** With this analysis, the structure is divided into several parts which are then separately designed. This means that the interaction of a subassembly with the rest of the structure is assumed to remain unchanged during the course of the fire. The boundary conditions at supports and forces and moments at boundaries of the part are taken equal to their values at time $t = 0$. It is thus important that these subassemblies are well chosen in order for this assumption to be acceptable. Within this part of the structure, the same
matters need to be taken in account as with the global structural analysis: the relevant failure mode under fire, the temperature dependent material properties (mechanical and thermal) and member stiffnesses, and effects of thermal expansions and deformations. This analysis may thus form a good middle road. It is less complicated than the global analysis since fewer members and parameters are covered at a time. However, within the subassembly the interaction between the different members is still covered. The effects of actions, apart from the internal actions at the boundaries, should be determined for time \( t = 0 \), using the appropriate combination for the fire situation. This is described in Section 6.7.

**Member analysis** involves the biggest number of simplifications and interaction between the different structural members is ignored. This analysis only needs to consider the effects due to deformations resulting from thermal gradients across the cross-section. The effects of axial or in-plane thermal expansions may be neglected. Similar to the analysis of a part of a structure, the effects of actions are determined for \( t = 0 \).

### 6.5.2 Design methods

The Eurocode describes the following design methods in order to satisfy Eq. (6.1):

- Design based on recognised design solutions, i.e. tabulated data for specific types of structural members
- Simplified calculation methods for simulating the behaviour of structural members
- Advanced calculation methods for simulating the behaviour of structural members, parts of the structure or the entire structure

Alternatively to design by calculation, according to EN 1990 Section 5, design may also be based on fire tests or on a combination of fire tests and calculation. However, fire tests do have limits. They are costly and they only allow elements of limited length to be tested without generally being able to simulate the real boundary conditions provided by the structure. Though, design through fire tests are useful for example when dealing with of a new type of concrete with poorly known material properties or a specific detail that is difficult to model.

The three calculation methods are now briefly discussed. They each require a separate way of working, have a different level of calculation and may, or may
not, apply to the previously given schematisations of the structure. The actual design according to these methods is then given in Section 6.9.

**Recognised design solutions**, or **tabulated data**, are purely based on empirical data, combined with experience and theoretical evaluation of test results. It bypasses the procedure of selecting design fires and determining the applied loads and the load capacity of the structure. They are only based on the standard fire, a type of nominal fire curve, and only member analysis is possible. For columns, walls, beams and slabs, the Eurocode contains several tables which give minimal dimensions of the cross-section and of the axis distance of the reinforcement till the closest concrete surface, for different fire loads. This makes it easy to use, but also quite conservative and only useful for simple and common cases.

**Simplified calculation methods** use the same procedure as with a normal-temperature design but taking into account the strength loss in both concrete and steel due to high temperatures. This is done by reducing the cross-section which in turn is done by ignoring any concrete over a certain limit temperature and possibly reducing the strength in the remaining concrete. Although this method takes more work than the first one, it is still fairly simple and most calculations can be done by hand. The application domain, however, stays limited. The most challenging here is the determination of the temperature profile in the concrete. Simplified calculation methods only apply to members and in some cases to parts of structures. They mostly make use of the standard fire curve.

**Advanced calculation methods** allow a complete temperature and mechanical analysis of the structure. The continuous alterations of the thermal and mechanical characteristics of the materials and their influence on each other and the complete structure must be taken into account. Advanced calculation methods account for the boundary conditions and the non-homogeneous distribution of the temperature inside the elements. They provide a very realistic analysis, however, the use of sophisticated computer programs, combined with a thorough background-knowledge, is necessary. Advanced calculation methods can be used with all types of analysis: member, part of the structure or entire structure and with all types of design fires.

**6.5.3 Discussion**
The Eurocode allows structural fire design based on either a prescriptive approach, with thermal actions given by a nominal fire, or a performance-based approach, with physically based thermal actions. The alternative design procedures
are summarized in both Figure 6.2 and Table 6.1. Important here is to understand the interconnection between the different schematisations of the structure and the different calculation methods.

**Figure 6.2: The alternative design procedures. Redrawn from EN 1992-1-2 (2004).**

**Table 6.1: Summary table showing alternative methods of verification for fire resistance. Redrawn from EN 1992-1-2 (2004).**

<table>
<thead>
<tr>
<th></th>
<th>Tabulated data</th>
<th>Simplified calculation methods</th>
<th>Advanced calculation model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member analysis</td>
<td>YES</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td>Analysis of parts of the structure</td>
<td>NO</td>
<td>YES</td>
<td>YES</td>
</tr>
<tr>
<td>Global structural analysis</td>
<td>NO</td>
<td>NO</td>
<td>YES</td>
</tr>
</tbody>
</table>
The tables with recognised design solutions are only developed based on the standard fire. In principle, tables based on other design fires are possible, but they need to be developed. Simplified calculation methods can be used together with standard fires and parametric fires which are a type of natural fire model. However the temperature profiles given in the Eurocode that can be used to determine the temperature inside the structural members are only developed for the standard fire. Similarly, the material models given in the Eurocode are only valid for heating rates similar to the standard fire curve. Thus design through natural fire models is possible, but the information needed is not directly provided by the Eurocode.

Member analysis is mainly used for the verification of standard fire resistance requirement. When dealing with a real fire development, the purpose is often to have a realistic evaluation of the fire behaviour of a real building, implying that interaction between the members should be considered in the model. Hence, it is required to perform a global analysis of the entire structure, including the parts that are directly exposed to the fire, but also the parts which are not exposed.

### 6.6 Thermal actions for temperature analysis

To be able to perform a temperature analysis, a design fire must be selected and the corresponding net heat flux must be calculated.

#### 6.6.1 The net heat flux

Thermal actions are defined by the net heat flux $h_{\text{net}}$ [W/m$^2$] to the surface of the regarded member and consist of the sum of heat transfer by convection as by radiation:

$$h_{\text{net}} = h_{\text{net,c}} + h_{\text{net,r}}$$

(6.2)

With the net convective heat flux $h_{\text{net,c}}$ [W/m$^2$] defined by:

$$h_{\text{net,c}} = \alpha_c \cdot (\Theta_g - \Theta_m)$$

(6.3)

Where

- $\alpha_c$ is the coefficient of heat transfer by convection [W/m$^2$K]
- $\Theta_g$ is the gas temperature in the vicinity of the fire exposed member [°C]
- $\Theta_m$ is the surface temperature of the member [°C]
The net radiative heat flux \( h_{\text{net},r} \) [W/m\(^2\)] is defined by:

\[
\dot{h}_{\text{net},r} = \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma^{sb} \cdot \left[ (\Theta_r + 273)^4 - (\Theta_m + 273)^4 \right]
\]  

(6.4)

Where

- \( \Phi \) is the configuration factor
- \( \varepsilon_m \) is the surface emissivity of the member
- \( \varepsilon_f \) is the emissivity of the fire
- \( \sigma^{sb} \) is the Stephan Boltzmann constant (= \( 5.67 \times 10^{-8} \) W/m\(^2\)K\(^4\))
- \( \Theta_r \) is the effective radiation temperature of the fire environment [°C]
- \( \Theta_m \) is the surface temperature of the member [°C]

The value of the coefficient of heat transfer by convection \( \alpha_c \) is given in the Eurocode depending on which design fire is chosen. The respective values can be found in the following Sections 6.6.2 and 6.6.3. The surface emissivity of concrete is \( \varepsilon_m = 0.7 \) (EN 1992-1-2, 2004) and, generally, the emissivity of the fire is taken as \( \varepsilon_f = 1 \). For concrete, the configuration factor should be taken as at least \( \Phi = 1 \), however a lower value may also be used to take into account possible position and shadow effects. An example of a method of calculation for doing this is given in Annex G of EN 1991-1-2 (2002). In Belgium this Annex has been made normative whereas in Sweden it remains informative. The radiation temperature \( \Theta_r \) may be represented by the gas temperature \( \Theta_g \) around that member, if the member is fully engulfed by the fire. The surface temperature \( \Theta_m \) results from temperature analysis and the gas temperature \( \Theta_g \) follows from the choice of design fire (a nominal temperature-time curve or a natural fire model).

6.6.2 Nominal temperature-time curves

Nominal temperature-time curves represent a fire through an explicit expression giving the gas temperature as a function of only one parameter: time. They bear no relationship to the specific characteristics (fire load, thermal properties of compartment linings, ventilation condition) of the building considered. The temperature is assumed to be uniformly distributed over the compartment. None of the nominal temperature-time curves model the cooling down phase of a fire. There exist numerous different types of curves but only three are included in the Eurocode.
**Standard temperature-time curve**

The standard temperature-time curve or ISO-curve is mainly used for building and is given by (see also Figure 6.3):

\[ \Theta_g = 20 + 345 \log_{10}(8t + 1) \]  \hspace{1cm} (6.5)

Where:

- \( \Theta_g \) is the gas temperature in the fire compartment [°C]
- \( t \) is the time [min]

The standard curve was designed to represent a typical building fire, that is, a cellulosic fire with materials as wood, paper, fabric, etc. as the source for the fuel. It can be easily seen on Figure 6.3 that, after a quarter of an hour the temperature reaches 745 °C and continues to rise with approximately 100 °C each time the time is doubled. In the equation, two phases of a fire can be seen: the flash-over with rapid temperature rise until about 800 °C and after, the period where the fire is completely developed. This analogy with a natural fire is also illustrated in Figure 2.4.

**External fire curve**

The external fire curve is used for structural members in a façade external to the main structure and is given by (see also Figure 6.3):

\[ \Theta_g = 660(1 - 0.687e^{-0.32t} - 0.313e^{-3.8t}) + 20 \]  \hspace{1cm} (6.6)

Where:

- \( \Theta_g \) is the gas temperature near the member [°C]
- \( t \) is the time [min]

The external fire curve reaches a constant temperature of 660 °C after 22 min.

**Hydrocarbon curve**

The hydrocarbon temperature-time curve is mainly used for petrochemicals and oil riggs and is given by (see also Figure 6.3):

\[ \Theta_g = 1080(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) + 20 \]  \hspace{1cm} (6.7)
Where:

- \( \Theta_g \) is the gas temperature in the fire compartment [°C]
- \( t \) is the time [min]

The hydrocarbon curve (Denoël, 2007), was developed in the 1970s by the petrol company Mobil. It shows a very fast increase of the temperature until 900 °C in the first 5 min followed by a constant temperature of 1100 °C. The research was conducted to start up a test-procedure for fire protection materials used on oil rigs and petrol complexes.

![Figure 6.3: Nominal temperature-time curves.](image)

### 6.6.3 Natural fire models

Natural fires are calculation techniques based on a consideration of the physical parameters specific to a particular building. They approximate a realistic fire much better, including the cooling phase, but they are also more complicated in use.

**Simplified fire models**

The simplified fire models are based on specific physical parameters with a restricted field of application. The design fire load density can be calculated using a method in Annex E of EN 1991-1-2 (2002). According to the Swedish National Annex, however, Annex E cannot be applied, instead they specify that the fire
load must be the value that occupies 80% of the observed values of representative statistic material. In Belgium, Annex E remains informative.

In case of a compartment fire, a uniform temperature distribution as a function of time is assumed. Any appropriate fire model may be used, but the model should be based on at least the fire load density and the ventilation conditions. In case of internal members of fire compartments, one example of a simplified fire model is the set of parametric temperature-time curves in Annex A of EN 1991-1-2 (2002). They produce a time-temperature relationship for any combination of fuel load, ventilation openings and wall lining materials. This approach provides a quick and easy approximation of the gas temperature inside the compartment and is well suited for the use on modern spreadsheets, however, only the post-flashover phase is covered. Annex A is recommended by the Swedish Annex. On the contrary, the Belgian National Annex finds these curves rather unsafe and only allows their use for preliminary design. At the implementation stage, a calculation following a zone model (see Advanced fire models) must be carried out if the designer wishes to go beyond the ISO curve.

For external members of compartment fires (e.g. balcony walkway supports), the radiative heat flux component consists of two parts: one from the radiation from the fire compartment and one from the flames emerging from the openings. For these members, a possible method of calculation is given in Annex B of EN 1991-1-2 (2002), which stays informative in Belgium and Sweden. This method allows for the calculation of the maximum compartment temperature, the size and temperature of the flame plume emerging from the openings and the heat transfer parameters for radiation and convection.

In many cases, flash-over is unlikely to occur. In those cases, thermal actions of a localised fire should be taken into account, naturally implying a non-uniform temperature distribution as a function of time (see Figure 6.4). This type of fire might seem more innocent than a generalised fire but it may also cause the destruction of a building or a part of the building. An example of a procedure for calculating temperatures in the case of a localised fire is presented in Annex C of EN 1991-1-2 (2002). In both Sweden and Belgium, this method is normative.
Figure 6.4: Localised fire according to the model of Annex C of EN 1991-1-2 (2002).

**Advanced fire models**

The Eurocode allows one of the following advanced fire models (EN 1991-1-2, 2002 and Denoël, 2007). **One-zone models** assume that the whole compartment is fully involved in the fire at the same time and the same temperature applies throughout. This could be used to model a post-flashover fire. The model allows the calculation of the evolution of the gas temperature as a function of time, by integrating the ordinary differential equations expressing mass and energy balances. **Two-zone models** separate the height of the compartment into two gaseous layers each with their own thermal environment. Similar to one-zone models, they allow the calculation of the evolution of the gas temperature as a function of time for both the top and bottom layer, based on mass and energy balances written for both layers. Two-zone models could be used to model pre-flashover fires (see also Figure 2.3). The third type of advanced fire models is **computational fluid dynamic (CFD) models**, which give the temperature completely dependent on both time and space. They analyze flow of fluids, transfers of heat and associated phenomena, by resolving the fundamental equations of Fluid Mechanics. The models are highly complex to solve and very sensitive to hypotheses, which makes them more suitable for research than for design. They could be used to analyse fires in which there are no definite boundaries to the gaseous state like very large compartments such as airport terminals, atria and sports stadia. They are also often used to model smoke movement. Which advanced fire model is to be used for calculating the heating conditions may be specified in the National Annex. Annex D of EN 1991-1-1 provides a method for the calculation of thermal
actions in case of one-zone, two-zone or CFD-models and is informative in both Belgium as Sweden.

All these models should include gas properties, as well as mass exchange and energy exchange. They are often solved through iterative procedures. As with simplified fire models, calculation methods for the design fire load density $q_{f,d}$ and the rate of heat release $Q$ are given in Annex E. In case of a localised fire, a combination of results obtained with a two-zone model and a localised fire approach may be considered in order to obtain a more accurate temperature distribution along a member. This distribution will then be constructed by taking the maximum value of both fire models at each location.

6.7 Mechanical actions for structural analysis

6.7.1 Actions that should be considered

This Section describes which mechanical actions should be considered when designing for fire. Firstly, the same actions assumed for normal temperature design should be taken into account, if they are likely to occur in the event of the fire. Representative values of variable actions should be defined for the accidental situation, in accordance with EN 1990. Indirect actions, e.g. forces and moments, due to imposed and constrained expansion and deformations caused by temperature changes within the structure should be considered as well. However, there are two exceptions to this rule: the case where the resulting actions are a priori either negligible or favourable should not be included. As with the case where they are a result of conservatively chosen support models and boundary conditions, and/or implicitly considered by conservatively specified fire safety requirements. For an assessment of indirect actions, the following should be considered according to the Eurocode (EN 1991-1-2, 2002):

- constrained thermal expansion of the members themselves, e.g. columns in multi-storey frame structures with stiff walls
- differing thermal expansion within statically indeterminate members, e.g. continuous floor slabs
- thermal gradients within cross-sections giving internal stresses
- thermal expansion of adjacent members, e.g. displacement of a column head due to the expanding floor slab, or expansion of suspended cables
- thermal expansion of members affecting other members outside the fire compartment
The design values of these indirect actions are determined using the thermal and mechanical material properties discussed further in this text. For members subjected to the standard fire, only indirect actions resulting from thermal gradients within the cross-section, i.e. internal stresses, need to be considered.

Additional actions should not be considered when they occur simultaneous with other independent accidental actions. Fire induced additional action (i.e. partial collapse) may need to be considered during the fire exposure, depending on the accidental design situation to be considered.

### 6.7.2 Effects of actions and combination rules

**General**

Actions on structures due to fire are considered as accidental loads, hence they must be combined according to the accidental design situations in EN 1990, resulting in the design value $E_{d,fi,t}$. The value of the variable action $Q_{k,1}$ may be represented by either the quasi-permanent value $\psi_{2,1} Q_{k,1}$ or the frequent value $\psi_{1,1} Q_{k,1}$. This choice may be specified in the National Annex but the Eurocode recommends the quasi-permanent value. The Belgian National Annex agrees with this recommendation, except for wind loads where the frequent value is advised. In Sweden the frequent value must be used in all cases. The values of $\psi_{1,1}$ and $\psi_{2,1}$ are given in Annex A of EN1990.

It may be noted that the load-weighting coefficients in the combinations of actions are considerably lower in the case of a fire, compared to normal temperature design, usually in the order of 50 to 70 % (Denoël, 2007). This reduction is based on the fact that the probability of the occurrence of a fire together with extreme loads is extremely small.

**Simplifications**

A constant value of the effects of actions $E_{d,fi}$ may be used throughout the fire exposure, but only when indirect actions need not to be explicitly considered. Thus this simplification is applicable for member analysis and for the determination of the reactions at supports and internal actions at the boundaries when analysing of a part of structure. Furthermore, the effect of actions in the fire condition may be determined from those used in normal temperature design.

$$ E_{d,fi,t} = E_{d,fi} = \eta_d E_d $$  \hspace{1cm} (6.8)
Where:

- $E_d$ is the design value of the relevant effects of actions from the fundamental combination according to EN 1990
- $E_{d,f}$ is the corresponding constant design value in the fire situation
- $\eta_f$ is a reduction factor for design load level in the fire situation $E_{d,f}/E_d$

The Eurocode provides formulas to determine $\eta_f$, which depend on what load combination of EN 1990 is used to determine $E_d$. For load combination (6.10) of EN 1990:

$$\eta_f = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$  \hspace{1cm} (6.9)

and for load combinations (6.10a) and (6.10b) the smaller value of the two following expressions:

$$\eta_f = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} \psi_{\psi,1} Q_{k,1}}$$  \hspace{1cm} (6.10)

$$\eta_f = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$  \hspace{1cm} (6.11)

With

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

The Eurocode also provides a figure illustrating Eq. (6.9), here Figure 6.5. It gives the variation of the reduction factor $\eta_f$ as a function of the load ratio $Q_{k,1}/G_k$ and different values of the combination factor $\psi_{1,1}$. Expressions (6.10) and (6.11) give slightly higher values.
Figure 6.5: Variation of the reduction factor $\eta_{fi}$ as a function of the load ratio $Q_{k,1}/G_k$ and different values of the combination factor $\psi_{1,1}$. Made assumptions: $\gamma_\varepsilon = 1.35$ and $\gamma_q = 1.5$. Redrawn from EN 1992-1-2 (2004).

### 6.8 Material Properties

Here, the material properties at elevated temperatures of both concrete and steel are given according to the Eurocode (EN 1992-1-2, 2004). They are to be used in the temperature and mechanical analysis of the fire design strategy, for both simplified and advanced calculation methods, although also alternative material laws may be applied, provided that they comply with experimental evidence. A simplified calculation method only requires the properties of strength, whereas an advanced calculation method also uses the thermal properties and the properties of deformation. The design method using tabulated data does not require the use of these properties. The material laws given are greatly based on fire tests. The mechanisms behind the behaviour of the material or underlying physical and chemical phenomena are not addressed in the Eurocode. However, here, this is covered in Chapter 3.

The material properties in this Chapter are initially given as characteristic values. How to calculate the design values is covered in Section 6.8.4. Furthermore, many mechanical properties are given by a reduction factor that represents the decrease of the property for an increase of the material temperature. The corresponding mechanical property at normal temperature (20°C) can be found in EN 1992-1-1. Note also that the numerical values on strength and deformation prop-
roperties given here are based on steady state tests, transient tests, or a combination of both. Since creep effects are not explicitly considered, the given material models are only applicable for heating rates between 2 and 50°C per minute. If the heating rate deviates from this range, the reliability of the strength and deformation properties must be demonstrated explicitly. Furthermore, the concrete material properties given apply to normal strength concrete, i.e. up to C50/60. Additional rules for high strength concrete can be found in Section 6.10, particularly with respect to the strength reduction. The properties of concrete with lightweight aggregates is given in EN 1994-1-2, Section 3.3.3 (NBN EN 1992-1-2-ANB, 2010). Finally, the behaviour of self-compacting concrete is not included in the Eurocode due to the small amount of experimental data the conflicting results (Denoël, 2007 and Ye et al., 2007).

6.8.1 Material properties of concrete

Concrete in compression

Both the compressive strength and the stiffness of uniaxially stressed concrete are reduced when the temperature rises. The reduction of the modulus of elasticity is even bigger since the peak stress strain increases with temperature. This is all reflected by Figure 6.6 where different stress-strain diagrams are plotted for different temperatures and for a certain type of concrete. Note, that the Eurocode makes a difference between siliceous aggregate concrete and calcareous aggregate concrete. The latter is defined as a concrete containing more than 80 % in mass in calcareous aggregates.

The stress-strain relationship is characterised by the following three variables, which are also illustrated in Figure 6.7:

- $f_{c,\theta}$ is the characteristic value of the compressive strength of concrete at temperature $\theta$
- $\varepsilon_{c1,\theta}$ is the strain corresponding to $f_{c,\theta}$
- $\varepsilon_{cu1,\theta}$ is the strain that defines the end of the descending branch
Figure 6.6: Stress-strain relationship of a siliceous aggregate concrete C30/37 for different values of the concrete temperature. The descending branch is chosen to be linear. Based on a figure from the fib Bulletin 46 (2008).

The mathematical expression of the ascending branch is:

\[
\sigma = \frac{3\varepsilon f_{c,\theta}}{\varepsilon_{c1,\theta} \left( 2 + \left( \frac{\varepsilon}{\varepsilon_{c1,\theta}} \right)^3 \right)} \quad \text{for } \varepsilon \leq \varepsilon_{c1,\theta}
\]  

(6.12)

The descending branch must be adopted to avoid numerical instability during modelling. The shape may be chosen and both a linear as a non-linear shape is permitted (see also Figure 6.7). The main parameters of the stress-strain curve are dependent on temperature and their value, intended for normal weight concrete, is found in Table 6.2. Linear interpolation may be used. This mathematical model, particularly the descending branch, is less suitable for natural fire models, hence an alternative model should be used (see also Section 6.9.3). Which modifications that are necessary are, however, not specified in the Eurocode. It is also noted that possible gain of strength in the cooling down phase of a fire should not be taken into account.
Table 6.2: Main parameters of the stress-strain curve for compressed normal weight concrete with siliceous or calcareous aggregates at elevated temperatures. Linear interpolation may be used. Copied from EN 1992-1-2 (2004).

<table>
<thead>
<tr>
<th>Concrete temp. θ [°C]</th>
<th>Siliceous aggregates</th>
<th>Calcareous aggregates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_{c,0}/f_{c,ck} )</td>
<td>( \varepsilon_{c,1,θ} )</td>
</tr>
<tr>
<td>1 2 3 4 5 6 7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1,00</td>
<td>0,0025</td>
</tr>
<tr>
<td>100</td>
<td>1,00</td>
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<tr>
<td>200</td>
<td>0,95</td>
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<td>0,0250</td>
</tr>
<tr>
<td>1200</td>
<td>0,00</td>
<td>-</td>
</tr>
</tbody>
</table>
Concrete in tension

In general, the tensile strength of concrete should be ignored. This makes a conservative design. However, if the tensile strength should appear to be necessary, the following model may be used, when in absence of more accurate information.

\[
\begin{align*}
    k_{c,t}(\theta) &= 1,0 & \text{for } 20^\circ C \leq \theta \leq 100^\circ C \\
    k_{c,t}(\theta) &= 1,0 - 1,0 \frac{(\theta - 100)}{500} & \text{for } 100^\circ C \leq \theta \leq 600^\circ C
\end{align*}
\]  

(6.13)

Where \(k_{c,t}(\theta)\) is the reduction factor of the characteristic tensile strength of concrete, defined as \(f_{ck,t}(\theta) = k_{c,t}(\theta)f_{ck,t}\). The model is illustrated in Figure 6.8.
Figure 6.8: Reduction factor $k_{c,t}(\theta)$ of the tensile strength ($f_{ck,t}$) of concrete at elevated temperatures. Redrawn from EN 1992-1-2 (2004).

In Figure 6.9, the reduction of the compressive strength with temperature is plotted together with the reduction in tensile strength. This also shows that the tensile strength decreases faster than the compressive strength.

Figure 6.9: Comparison of the reduction in compressive and tensile strength of siliceous and calcareous concrete for elevated temperatures
**Thermal elongation**
The thermal strain $\varepsilon_c(\theta) = (\Delta l/l)_c$ [-] of concrete as a function of the concrete temperature is displayed in Figure 6.10.

![Figure 6.10: Total thermal elongation $\varepsilon_c$ of concrete as a function of concrete temperature $\theta$. Redrawn from EN 1992-1-2 (2004).](image)

The mathematical expression is:

Siliceous aggregates:

$$
\begin{align*}
\varepsilon_c(\theta) &= -1.8 \times 10^{-4} + 9 \times 10^{-6} \theta + 2.3 \times 10^{-11} \theta^3 \\
&= 14 \times 10^{-3} \\
&\quad \text{for} \quad 20^\circ\text{C} \leq \theta \leq 700^\circ\text{C} \\
&= 12 \times 10^{-3} \\
&\quad \text{for} \quad 700^\circ\text{C} \leq \theta \leq 1200^\circ\text{C}
\end{align*}
$$

(6.14)

Calcareous aggregates:

$$
\begin{align*}
\varepsilon_c(\theta) &= -1.2 \times 10^{-4} + 6 \times 10^{-6} \theta + 1.4 \times 10^{-11} \theta^3 \\
&= 12 \times 10^{-3} \\
&\quad \text{for} \quad 20^\circ\text{C} \leq \theta \leq 805^\circ\text{C} \\
&= 14 \times 10^{-3} \\
&\quad \text{for} \quad 805^\circ\text{C} \leq \theta \leq 1200^\circ\text{C}
\end{align*}
$$

(6.15)

**Specific heat**
The specific heat $c_p(\theta)$ [kJ/kg°C] of siliceous and calcareous concrete, as a function of concrete temperature, is given in Figure 6.11 for three values of the moisture content. The peak between 100 °C and 200 °C corresponds to the heat that is
needed to evaporate the water inside the concrete. This peak (Jansson, 2008) has a certain width due to a variable boiling point for the water as a result of pore size and pore pressure (see also Section 3.2).

![Figure 6.11: Specific heat $c_p$ of siliceous and calcareous concrete as a function of concrete temperature $\theta$ for three different moisture contents $u$: 0%, 1.5% and 3% of concrete weight. Linear interpolation is allowed for other moisture contents. Redrawn from EN 1992-1-2 (2004).](image)

The mathematical expression for dry concrete is:

$$
\begin{align*}
    c_p(\theta) &= 900 \text{ (J/kg°C)} \quad \text{for } 20 \text{°C} \leq \theta \leq 100 \text{°C} \\
    c_p(\theta) &= 900 + (\theta - 100) \text{ (J/kg°C)} \quad \text{for } 100 \text{°C} \leq \theta \leq 200 \text{°C} \\
    c_p(\theta) &= 1000 + (\theta - 200) / 2 \text{ (J/kg°C)} \quad \text{for } 200 \text{°C} \leq \theta \leq 400 \text{°C} \\
    c_p(\theta) &= 1100 \text{ (J/kg°C)} \quad \text{for } 400 \text{°C} \leq \theta \leq 1200 \text{°C}
\end{align*}
$$

(6.16)

For moist concrete, a peak value $c_{p,\text{peak}}$ is introduced between 100°C and 115°C, followed by a linear decrease between 115°C and 200°C. In case of other moisture contents, linear interpolation is allowed.

- $c_{p,\text{peak}} = 900 \text{ J/kg°C for } u = 0 \%$
- $c_{p,\text{peak}} = 1470 \text{ J/kg°C for } u = 1.5 \%$
- $c_{p,\text{peak}} = 2020 \text{ J/kg°C for } u = 3 \%$
The volumetric specific heat $c_v(\theta)$ [kJ/m$^3$°C] is also defined in the Eurocode. It is the product of the density of the concrete $\rho(\theta)$ and the specific heat $c_p(\theta)$. The density is influenced by water loss and its mathematical expression is:

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

(6.17)

An example of the volumetric specific heat for a concrete with 3% moisture content and a density of 2300 kg/m$^3$ is given in Figure 6.12.

\[\text{Figure 6.12: Volumetric specific heat } c_v \text{ of siliceous and calcareous concrete as a function of concrete temperature } \theta \text{ for a concrete with 3% moisture content and a density of 2300 kg/m}^3. \text{ Redrawn from EN 1992-1-2 (2004).}\]

**Thermal conductivity**

The Eurocode determines only a lower and an upper limit for the thermal conductivity $\lambda_c$ [W/m°C] of concrete. Additional rules may be given in the National Annex of the Member State. For example, in Sweden, the lower limit must be applied. In Belgium, the lower limit is used for calcareous concrete and the average of the two limits for siliceous concrete. It can be read in the Eurocode (EN
1992-1-2, 2004) that the lower limit of thermal conductivity was obtained through the comparison of temperatures measured in fire tests performed on different types of concrete. This limit gives more realistic temperatures than the upper limit, which was obtained from tests on composite steel/concrete structures.

The upper limit is defined as:

$$\lambda_c = 2 - 0.2451(\theta / 100) + 0.0107(\theta / 100)^2$$

for $20^\circ C \leq \theta \leq 1200^\circ C$ \hspace{1cm} (6.18)

The lower limit is defined as:

$$\lambda_c = 1.36 - 0.136(\theta / 100) + 0.0057(\theta / 100)^2$$

for $20^\circ C \leq \theta \leq 1200^\circ C$ \hspace{1cm} (6.19)

Both relations are illustrated in Figure 6.13.

![Figure 6.13](image)

**Figure 6.13:** Upper and lower limit of the thermal conductivity of concrete $\lambda_c$ as a function of the concrete temperature $\theta$. Redrawn from EN 1992-1-2 (2004).

The two previously given thermal properties summarized: for an increase in temperature, the thermal conductivity will decrease and the specific heat will increase. As a result, the thermal diffusivity decreases with increasing temperature.

### 6.8.2 Material properties of steel

The Eurocode only gives the strength and deformation properties and the thermal elongation of the steel. The other thermal properties are generally not needed, because the amount of steel is in most concrete structures so low that it
hardly influences the total temperature distribution (Denoël, 2007). When nevertheless needed, the other properties can be found in Eurocode 3 (EN 1993-1-2, 2005).

**Strength and deformation properties of reinforcing steel**
Like concrete, the strength and stiffness of steel are reduced when the temperature increases. This is also shown in Figure 6.14 and Figure 6.15.

![Figure 6.14: Reduction factor $f_{sy,\theta}/f_{yk}$ of the yield strength as a function of the steel temperature $\theta$ of hot rolled and cold worked reinforcement steel (class N).](image1)

![Figure 6.15: Reduction factor $E_{s,\theta}/E_s$ of the tangent modulus as a function of the steel temperature $\theta$ of hot rolled and cold worked reinforcement steel (class N).](image2)
These two parameters, the maximum stress level \( f_{sy,\theta} \) and the slope of the linear elastic range \( E_{s,\theta} \), together with the proportional limit \( f_{sp,\theta} \) define the stress-strain relationship of the reinforcement steel, shown in Figure 6.16. The linear descending branch is again there for numerical reasons. This formulation may be used for steel in both compression and in tension. There are clearly four different parts with each its own mathematical expression:

1/ \( \varepsilon \leq \varepsilon_{sp,\theta} \)

\[
\sigma = \varepsilon E_{s,\theta}
\]  

(6.20)

The tangent modulus of this part is \( E_{s,\theta} \).

2/ \( \varepsilon_{sp,\theta} \leq \varepsilon \leq \varepsilon_{sy,\theta} \)

\[
\sigma = f_{sp,\theta} - c + \left( b / a \right) \left[ \alpha^2 - (\varepsilon_{sy,\theta} - \varepsilon)^2 \right]^{0.5}
\]  

(6.21)

With:

\[
\alpha^2 = (\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta} + c / E_{s,\theta})
\]  

(6.22)

\[b^2 = c(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})E_{s,\theta} + c^2\]  

(6.23)

\[c = \frac{(f_{sy,\theta} - f_{sp,\theta})^2}{(\varepsilon_{sy,\theta} - \varepsilon_{sp,\theta})E_{s,\theta} - 2(f_{sy,\theta} - f_{sp,\theta})}\]  

(6.24)

The tangent modulus of this part is \( \frac{b(\varepsilon_{sy,\theta} - \varepsilon)}{a \left[ \alpha^2 - (\varepsilon - \varepsilon_{sy,\theta})^2 \right]^{0.5}} \).

3/ \( \varepsilon_{sy,\theta} \leq \varepsilon \leq \varepsilon_{sy,\theta} \)

\[
\sigma = f_{sy,\theta}
\]  

(6.25)

4/ \( \varepsilon_{sy,\theta} \leq \varepsilon \leq \varepsilon_{su,\theta} \)

\[
\sigma = f_{sy,\theta} \left[ 1 - (\varepsilon - \varepsilon_{st,\theta})/(\varepsilon_{su,\theta} - \varepsilon_{st,\theta}) \right]
\]  

(6.26)
The strain parameters are:

\[
\varepsilon_{sp,\theta} = \frac{f_{sp,\theta}}{E_{s,\theta}}
\]

\[
\varepsilon_{sy,\theta} = 0.02
\]

\[
\varepsilon_{st,\theta} = 0.15
\]

\[
\varepsilon_{su,\theta} = 0.20
\]

But in case of class A reinforcement (see Annex C of EN 1992-1-1):

\[
\varepsilon_{st,\theta} = 0.05
\]

\[
\varepsilon_{su,\theta} = 0.10
\]

Figure 6.16: Stress-strain diagram of the reinforcement steel and indication of the different parts. Redrawn from EN 1992-1-2 (2004).

The values for the parameters of the stress-strain relationship are given in Table 6.3, where linear interpolation may be used. The Eurocode (EN 1992-1-2, 2004) contains two classes of values: Class N and Class X. Class X may, however, only be used if strength is tested at elevated temperatures. Furthermore, according to both the National Annexes of Sweden and Belgium, Class N should be used. Hence, here, only Class N values are given. For Class X values, the reader is referred to the Eurocode.
Table 6.3: Values of the material properties for the stress-strain relationship of hot rolled and cold worked reinforcement steel at elevated temperatures (Class N). Linear interpolation may be used. Copied from EN 1992-1-2 (2004).

<table>
<thead>
<tr>
<th>Steel temperature</th>
<th>( f_{sy,\theta}/f_{yk} )</th>
<th>( f_{sp,\theta}/f_{yk} )</th>
<th>( E_{p,\theta}/E_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \theta[^{\circ}C] )</td>
<td>hot rolled</td>
<td>cold worked</td>
<td>hot rolled</td>
</tr>
<tr>
<td>1</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
</tr>
<tr>
<td>20</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
</tr>
<tr>
<td>100</td>
<td>1,00</td>
<td>1,00</td>
<td>0,81</td>
</tr>
<tr>
<td>200</td>
<td>1,00</td>
<td>1,00</td>
<td>0,61</td>
</tr>
<tr>
<td>300</td>
<td>1,00</td>
<td>0,94</td>
<td>0,42</td>
</tr>
<tr>
<td>400</td>
<td>0,78</td>
<td>0,67</td>
<td>0,36</td>
</tr>
<tr>
<td>500</td>
<td>0,47</td>
<td>0,4</td>
<td>0,18</td>
</tr>
<tr>
<td>600</td>
<td>0,23</td>
<td>0,12</td>
<td>0,07</td>
</tr>
<tr>
<td>700</td>
<td>0,11</td>
<td>0,11</td>
<td>0,05</td>
</tr>
<tr>
<td>800</td>
<td>0,06</td>
<td>0,08</td>
<td>0,04</td>
</tr>
<tr>
<td>900</td>
<td>0,04</td>
<td>0,05</td>
<td>0,02</td>
</tr>
<tr>
<td>1000</td>
<td>0,02</td>
<td>0,03</td>
<td>0,01</td>
</tr>
<tr>
<td>1200</td>
<td>0,00</td>
<td>0,00</td>
<td>0,00</td>
</tr>
</tbody>
</table>

**Strength and deformation properties of prestressing steel**

For the strength and deformation properties of prestressing steel, the mathematical model as for reinforcement steel may be used. The different parameters of this model are \( f_{py,\theta}/(\beta f_{pk}) \), \( f_{pp,\theta}/(\beta f_{pk}) \), \( E_{p,\theta}/E_p \), \( \varepsilon_{pt,\theta} \) and \( \varepsilon_{pu,\theta} \). Their values can be found in Table 6.4. A distinction is made between cold worked (wires and strands) and quenched and tempered (bars) prestressing steel. The value of \( \beta \) depends on the choice of Class A or Class B. This choice may be specified in the National Annex. In both Sweden and Belgium, class A must be used. However, Belgium allows a first approximation with \( \beta=0,95 \).
For Class A (definition and values of the parameters is given in Section 3.3 of EN 1992-1-1):

\[
\beta = \left[ \frac{\varepsilon_{ud} - f_{p0,1k} / E_p}{\varepsilon_{uk} - f_{p0,1k} / E_p} \right] \left[ \frac{f_{pk} - f_{p0,1k}}{f_{pk}} \right] + \frac{f_{p0,1k}}{f_{pk}}
\]  

(6.27)

For Class B:

\[
\beta = 0.9
\]  

(6.28)

Table 6.4: Values of the material properties for the stress-strain relationship of cold worked (cw) (wires and strands) and quenched and tempered (q & t) (bars) prestressing steel at elevated temperatures. Linear interpolation may be used. Copied from EN 1992-1-2 (2004).
The variation of the reduction factor of the strength is also illustrated in Figure 6.17.

**Figure 6.17**: Reduction factor $f_{py,\theta}/(\beta f_{pk})$ of the yield strength as a function of the steel temperature $\theta$ of cold worked (wires and strands) and quenched and tempered (bars) prestressing steel. Redrawn from EN 1992-1-2 (2004).

**Thermal elongation of reinforcing and prestressing steel**

The thermal strain $\varepsilon_s(\theta)$ [-] of reinforcement and prestressing steel as a function of the steel temperature is displayed in Figure 6.18.

**Figure 6.18**: Thermal elongation of reinforcing steel and prestressing steel as a function of the steel temperature. Redrawn from EN 1992-1-2 (2004).
The mathematical expression is:

For reinforcing steel:

\[
\begin{align*}
\varepsilon_s &= -2.416 \times 10^{-1} + 1.2 \times 10^{-5} \theta + 0.4 \times 10^{-8} \theta^2 & \text{for } 20°C \leq \theta \leq 750° \\
\varepsilon_s &= 11 \times 10^{-3} & \text{for } 750°C \leq \theta \leq 860° \quad (6.29) \\
\varepsilon_s &= -6.2 \times 10^{-3} + 2 \times 10^{-5} \theta & \text{for } 860°C \leq \theta \leq 1200°
\end{align*}
\]

For prestressing steel:

\[
\varepsilon_p = -2.016 \times 10^{-1} + 10^{-5} \theta + 0.4 \times 10^{-8} \theta^2 & \text{for } 20°C \leq \theta \leq 1200° \quad (6.30)
\]

6.8.3 Comparison of the properties

Inhibited expansion will not cause rupture in the concrete nor the steel (Denoël, 2007). This can be seen in Figure 6.19, where the extension due to free expansion of siliceous aggregate concrete, which is the conservative choice, is always smaller than the ultimate strain \( \varepsilon_{c1,\theta} \). Later, it will be shown that this effect in concrete is really a consequence of the transient strain (see Section 6.9.3). Similarly for reinforcement and prestressing steel, the conservative choice of the free extension stays smaller than the yield strain \( \varepsilon_{sy,\theta} = \varepsilon_{py,\theta} \). Even though rupture due to inhibited expansion must not be feared, the thermal expansion of concrete and steel could still reach significant values, e.g. 1% in the range of 800°C (fib Bulletin 46, 2008). This will cause either large displacements which give rise to second order effects, or it will cause indirect effects of actions if the expansion is prohibited (see also Chapter 4).

In Figure 6.20 and Figure 6.21 a comparison is made between concrete (choice of siliceous aggregate) and steel (choice of hot rolled) concerning both the reduction in strength as in stiffness. The concrete stiffness reduction factor is defined as \( E_{c,\theta}/E_c(20°C) \) where \( E_{c,\theta} = f_{c,\theta} / \varepsilon_{c,\theta} \). It can be concluded that the stiffness of concrete reduces quicker than the stiffness of steel. This makes that concrete can resist forces due to restrain relatively easy (Denoël, 2007). The stiffness reduction, however, only effects the overall stiffness of compressed concrete elements relatively little, since only a small outer layer of the concrete element is heated. Steel, on the other hand, has a much higher (25 times) thermal diffusivity and a very low massivity. The overall stiffness can be thus be significantly reduced, which may change the buckling behaviour.
Figure 6.19: Comparison of the ultimate strain versus the free expansion of concrete (siliceous aggregate) and steel (reinforcement). Redrawn from (Denoël, 2007).

Figure 6.20: Comparison of the strength reduction of concrete (siliceous aggregate) and reinforcement steel (hot rolled). Redrawn from (Denoël, 2007).

Figure 6.21: Comparison of the stiffness reduction of concrete (siliceous aggregate) and reinforcement steel (hot rolled). Redrawn from (Denoël, 2007).
6.8.4 Design values of material properties

Now, the material properties of concrete and steel are known. These are, however, only the characteristic values. Thus, in Eurocode 2 (EN 1992-1-2, 2004), the design value of a mechanical (strength and deformation) material property for the fire situation $X_{d,fi}$ is defined as:

$$X_{d,fi} = \frac{k_d X_k}{\gamma_{M,fi}}$$  \hspace{1cm} (6.31)

Where:

- $X_k$ is the characteristic value of a strength or deformation property (generally $f_k$ or $E_k$) for normal temperature
- $k_\theta$ is the reduction factor for a strength or deformation property ($X_{k,\theta}/X_k$), dependent on the material temperature $\theta$. Values of this factor for both concrete and steel were given in the previous Sections.
- $\gamma_{M,fi}$ is the partial safety factor for the strength or deformation property, in the fire situation

Equivalent, the design value of a thermal material property for the fire situation $X_{d,fi}$ is defined as follows:

In case an increase of the property is favourable for safety

$$X_{d,fi} = \frac{X_{k,\theta}}{\gamma_{M,fi}}$$  \hspace{1cm} (6.32)

In case a decrease of the property is favourable for safety

$$X_{d,fi} = \gamma_{M,fi} X_{k,\theta}$$  \hspace{1cm} (6.33)

Where:

- $X_{k,\theta}$ is the characteristic value of a thermal material property in the fire situation, generally dependent on the material temperature $\theta$
- $\gamma_{M,fi}$ is the partial safety factor for the thermal material property, in the fire situation

The value of $\gamma_{M,fi}$ may be defined in the National Annex of the respective Member State. The Eurocode, however, recommends the use of $\gamma_{M,fi} = 1.0$ for both mechanical and thermal material properties. Sweden and Belgium both comply with this
recommended value. As a consequence there is no change and the characteristic material property values may be used as design values.

**Influence of time**
The fib bulletin 46 (2008) mentions the influence of time. In normal temperature design, the design value of the concrete compressive strength is calculated as (EN 1992-1-1)

\[
f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}
\]  

(6.34)

The characteristic compressive strength is divided by a partial safety factor and multiplied with a factor \( \alpha_{cc} \), a “coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied”. The reason for the factor \( \alpha_{cc} \) is that the characteristic compressive strength was obtained by testing, made within a certain time scale, usually some seconds or minutes. Concrete structures, on the other hand, are loaded for a much bigger period of time, typically several decades. Would the loading on the specimen be maintained for a much longer period of time, the resistance would be smaller, in the order of 0 to 20% lower.

Naturally, realistic fires last only a short period of time, typically several minutes or hours, and the multiplicative factor \( \alpha_{cc} \) is not included in the expression (6.31). Caution should, however, be made when designing concrete structures that are exposed to high temperatures for a much longer duration, e.g. industrial furnaces. The fib bulletin 46 (2008) advises to experimentally determine the value of the factor \( \alpha_{cc} \) for these cases and certainly not assume that it keeps the same value as at room temperatures.

### 6.9 Assessment of the fire resistance

#### 6.9.1 General

As mentioned before, the Eurocode (EN 1992-1-2, 2004) allows three methods of design to solve Eq. (6.1):

- Design based on recognised design solutions, i.e. tabulated data for specific types of structural members
- Simplified calculation methods for simulating the behaviour of structural members
- Advanced calculation methods for simulating the behaviour of structural members, parts of the structure or the entire structure

These methods were already briefly explained in Section 6.5. Here, they are studied in detail, particularly with respect to how they must be applied in order to design concrete structures for the fire situation.

6.9.2 Simplified calculation methods

A simplified calculation method uses the same procedure as normal-temperature design but taking into account the strength loss in both concrete and steel due to the high temperatures. The ultimate-load-bearing capacity of a cross-section under fire conditions is determined which then is compared to the relevant combination of actions. The methods serve mostly only members, but in some cases also parts of structures can be covered. The design can be based on both nominal and parametric fires, however, the standard fire curve is most often used.

Concrete and reinforcing temperatures

Determining the temperatures of the concrete and the reinforcement at any depth and at any period of time during the fire exposure is a crucial step. The Eurocode allows both testing and calculation methods. A huge amount of test have already been performed in the past, resulting in many design charts that can be found in the literature. These give thermal gradients for certain concrete types and certain types of members (beams, columns, slabs and walls) exposed to the standard fire. One set of charts can be found in Annex A of Eurocode 2. Note, however, that a design chart is always made based on several assumptions (e.g. aggregate type, moisture content, etc.) and its application domain is limited.

In other cases, thermal calculations are to be preferred. For example, when the appropriate design chart cannot be found, or when a more realistic design fire is preferred. Common assumptions for calculations are that the heat transfer is a function of the concrete alone, and that the temperature of the reinforcing steel equals the temperature of the surrounding concrete (Buchanan, 2002 and fib Bulletin 46, 2008). This is based on the fact that the thermal conductivity of steel is much higher than that of concrete and that the steel bars are commonly parallel to the fire-exposed surfaces, thus barely affecting the heat transfer perpendicular to the surfaces. The only accurate way of calculating the temperatures inside reinforced concrete members is through a two-dimensional finite-element computer program which gives the temperature distribution with time over the cross-section. The advantage of such a program is that a lot of different variables can
be taken in to account and modified easily (e.g. combination of materials, voids, etc.). For simple members consisting of normal-weight concrete, empirical hand calculation methods are available, derived from computer-based thermal analyses (Wickström, 1986 and Hertz, 1981). Wickström’s method applies to slabs (i.e. one-dimensional heat transfer) of normal-weight concrete in the standard fire and gives the concrete temperature at a certain depth $x$ (m) in to the slab at time $t_h$ (hours):

$$\Theta_m = \eta_m \Theta_g$$  \hspace{1cm} (6.35)

$$\eta_m = 1 - 0.0616 t_h^{-0.88}$$  \hspace{1cm} (6.36)

Where $\Theta_m$ is the fire-exposed surface temperature and $\Theta_g$ is the fire temperature. The concrete temperature $\Theta_c$ is a factor $\eta_x$ of $\Theta_m$, with $\eta_x$ given by:

$$\eta_x = 0.18 \ln(t_h / x^2) - 0.81$$  \hspace{1cm} (6.37)

Hence the concrete temperature is given by:

$$\Theta_c = \eta_x \eta_w \Theta_g$$  \hspace{1cm} (6.38)

This method can also be used for heat conduction in two directions, e.g. corners of beams, by introducing a factor $\eta_y$ which is calculated in the same way as $\eta_x$. The concrete temperature is then:

$$\Theta_c = \left[ \eta_w \left( \eta_x + \eta_y - 2 \eta_x \eta_y \right) + \eta_w \eta_w \right] \Theta_g$$  \hspace{1cm} (6.39)

Wickström (1986) shows how these equations can be modified to suit other types of concrete, and also gives approximate methods to determine concrete temperatures in realistic fires with a decay period. Empirical calculations in the decay period are however less accurate because the maximum concrete temperatures occur a considerable time after the maximum fire temperature has passed, therefore, it is recommended to use a finite-element model in the case of real fires. Note that the parametric temperature-time curves found in Annex A of EN 1991-1-2 (2002) have been based on the work carried out by Wickström (Moore and Lennon, 2007).

**Reduced cross-section**

A simplified calculation method may use a reduced cross-section. In Annex B of EN 1992-1-2 (2004), two examples of this method are given. The Annex retains
its informative status in both Sweden and Belgium. The first method is the 500 °C isotherm method that assigns full strength to the concrete with a temperature below 500 °C and no strength at all to the concrete with temperature above 500 °C. The method may be used for structures subjected to both a standard and a parametric fire. Secondly, the zone method subdivides the concrete section into a number \((n \geq 3)\) of zones of equal thickness parallel to the exposed surface. Each zone then receives a mean temperature and a corresponding reduction factor of the strength and stiffness (if applicable). The method is recommended for the design of small sections and slender columns, but is only valid for standard fire exposure. Both methods can be applied to sections of reinforced and prestressed concrete, with respect to axial load, bending moment and their combinations (for example an eccentric load). Second order effects may be included in both models. Several examples illustrating the 500°C isotherm method are given in Chapter 7.

One note of caution is that the choice of the isotherm temperature may differ from 500°C, depending on the type of concrete that is used. For certain concretes, the isotherm temperature may be well below 500°C, or even below 400°C (Andersberg, 2004). Also for high strength concrete, these values are modified (see Section 6.10).

**Strength reduction**

The strength reduction of the materials can be found in Section 6.8 and may be used together with the simplified reduced cross-section calculation methods. That is, the characteristic compressive strength of concrete can be found in Table 6.2 and is illustrated in Figure 6.9. The reduction of the characteristic strength of prestressing steel can be found in Table 6.4 and is illustrated in Figure 6.17. Similar values for reinforcement steel are given in Table 6.3 and Figure 6.14, however, the Eurocode also gives a more specific approach for the simplified calculation methods:

- Tension reinforcement of beams and slabs where \(\varepsilon_{s,fi} \geq 2\%\) may have a strength reduction according to column 2 of Table 6.3 for hot rolled and column 3 for cold worked reinforcing steel (class N)
- Tension reinforcement where \(\varepsilon_{s,fi} < 2\%\) and compression reinforcement of columns and compression zones of beams and slabs may have the strength reduction at 0,2% proof strain for Class N which is
The strength reduction of reinforcement steel is also illustrated in Figure 6.22. Note that the values of these strength reduction factors may only be used with heating rates similar to those in the standard heating curve until the maximum gas temperature is reached. However, these material models given in the Eurocode are solely informative. Alternative models may be used, if these are confirmed with experimental evidence.

![Graph showing reduction factor $f_{sy,\theta}/f_{yk}$ of the yield strength as a function of the steel temperature $\theta$ of tension and compression reinforcement steel (class N).]

**Figure 6.22**: Reduction factor $f_{sy,\theta}/f_{yk}$ of the yield strength as a function of the steel temperature $\theta$ of tension and compression reinforcement steel (class N).

### 6.9.3 Advanced calculation method

Advanced calculation methods provide a more realistic analysis of a structure. A reliable approximation of the expected behaviour of the structure under fire conditions is achieved by modelling the fundamental physical behaviour. These methods may be used together with any type of design fire (nominal or natural fires), with any type of analysis (member, part of a structure or global analysis)
and with any type of cross-section. Any potential failure modes not covered by an advanced calculation method (e.g. local buckling or shear and bond failure) must be eliminated by appropriate means or detailing in the design of the structure.

The Eurocode only gives the basic principles of this method. The designer applying this method must be an expert in the field of fire engineering. The use of advanced calculation methods is therefore often determined in the National Annex. For example, in Sweden, there are no restrictions to the use of the method. In Belgium, the use is specified in the ruling Belgian regulations.

An advanced calculation method exists out of two parts, which continuously influence each other, the thermal response model and the mechanical response model.

**Thermal response model**

With the thermal response model, the development and distribution of the temperature inside the members are determined, through the acknowledged principles and assumptions of the theory of heat transfer. The model requires both the relevant thermal actions from Section 6.6 and the temperature dependent material properties from Section 6.8. Often the reinforcement is neglected while determining a temperature profile because the amount of steel in most reinforced concrete structures is so low that it hardly effects the temperature distribution. Another element that may be neglected is the influence of moisture and moisture migration. This is a conservative approach. It is possible to include the effects of non-uniform thermal exposure and of heat transfer to adjacent building components.

**Mechanical response model**

Through the mechanical response model, the mechanical behaviour of the structure or any part of the structure is determined, while applying the acknowledged principles and assumptions of the theory of structural mechanics. The mechanical response model should, among others, take account of:

- The effect of the temperature on the material properties
- The effects of thermally induced strains and stresses, both due to temperature rise and due to temperature differentials
- Geometrical non-linear effects, where relevant

The deformations at ultimate limit state, resulting from the calculation methods, must be limited to make sure that all parts of the structure maintain compatible.
Plastic methods of analysis may be used in determining the load-bearing capacity of individual members, sub-assemblies or entire structures exposed to fire (see EN 1992-1-1, Section 5). The plastic rotation capacity of a section should then be estimated, taking account of the increased ultimate strains $\varepsilon_{cu}$ and $\varepsilon_{su}$ in hot condition. $\varepsilon_{cu}$ will also be affected by the confinement reinforcement provided. The compressive zone of a section, especially if directly exposed to fire (e.g. hogging in continuous beams), should be checked and detailed with particular regard to spalling or falling-off of concrete cover. When analysing individual members or sub-assemblies, the boundary conditions should be checked and detailed so that failure due to the loss of adequate support for the members does not occur.

**Constitutive models**

The mechanical behaviour of concrete under elevated temperatures is predicted by a constitutive model. In agreement with the findings of many researchers, the Eurocode (EN 1992-1-2, 2004) allows the strain $\varepsilon$ to be split up in four components:

$$\varepsilon = \varepsilon_{th} + \varepsilon_{\sigma} + \varepsilon_{\text{creep}} + \varepsilon_{tr}$$

(6.41)

Where:

- $\varepsilon_{th}$ is the thermal strain
- $\varepsilon_{\sigma}$ is the instantaneous stress-dependent strain
- $\varepsilon_{\text{creep}}$ is the creep strain
- $\varepsilon_{tr}$ is the transient state strain

The Eurocode doesn’t elaborate further on the subject, but more information can, for example, be found in (Buchanan, 2002).

The stress-dependent strain $\varepsilon_{\sigma}$, also called the mechanical or stress-related strain, is the strain that results in stresses in the structural members. It is a function of both the externally applied stress $\sigma$ and the temperature $\theta$. The stress-dependent strain is based on the relevant stress-strain relationships of the material. When performing a member analysis on, for example, a simply supported beam that is free to expand under elevated temperature, the stress-dependent strain is the only component of strain that needs to be considered.

Thermal strain $\varepsilon_{th}$ is the thermal expansion that occurs when a material is heated. It is only dependent on the material temperature $\theta$. This strain is not important for the analysis of a simply supported member, but has a significant ef-
fect in the behaviour of frames and complex structural members in fire, especially when deformations are inhibited by other parts of the structure.

Creep is the time-dependent effect that describes the deformation of a material when subjected to a constant load for significant time. The normal temperature course can be seen in Figure 6.23. Creep, however, becomes more important at elevated temperatures. It can accelerate when the load capacity reduces, which may cause secondary and even tertiary creep as shown in Figure 6.24. The creep strain $\varepsilon_{\text{creep}}$ is dependent on the applied stress $\sigma$, the temperature $\theta$ and time $t$. Fire engineering calculations, both hand and computer methods, usually do not incorporate creep strain because of its complexity and insufficient input data. These calculations are already highly complicated without a time-dependent property. Creep strain is then usually implicitly incorporated by using a stress-strain relationship which includes an allowance for the expected amount of creep.

Figure 6.23: Creep at normal temperatures.

Figure 6.24: Creep at elevated temperatures.
Transient state strain $\varepsilon_{tr}$ is the expansion that occurs during the first time heating of concrete under compression. It is unrecoverable and large, and it is caused by changes in the chemical composition of the concrete, thus it is unique for this material. This strain is a function of the applied stress $\sigma$, temperature $\theta$ and in some models the stress history $\sigma^*$ (discussed later). This strain component was discovered in the mid 1970s (Anderberg, 2004). Before, the understanding of the mechanical behaviour of concrete structures was very poor. Scientist could not explain as to why their models, based on elastic strains, were completely inadequate. They predicted that a concrete member, unloaded and restrained against thermal expansion during heating, would fail at about 350°C due to high thermal stresses. This does not comply with reality, where an unloaded restrained member will never fail by itself. The discovery of the phenomenon of transient strain provided the answer. This strain contributes to a significant relaxation and redistribution of the thermal stresses. For the first time, the realistic behaviour of fire exposed concrete elements could be analytically predicted with an acceptable accuracy. Above 100°C, transient strain is essentially a function of temperature and not of time, thus making it relatively easy to model. It is quite dominating and much larger than the elastic strain. Any structural analysis of concrete exposed to high temperatures that ignores transient strain will, therefore, be completely inappropriate and will yield erroneous results, particularly for columns exposed to fire, since these are largely in a state of compression. Unfortunately, this phenomenon is often ignored by practising engineers and is not very well incorporated in codes and standards (discussed in Chapter 8).

Most researches now roughly incorporate these four types of strain in their constitutive models, however the elaboration might differ slightly. In practice, these four components of strain appear to be difficult to separate and during an experiment it is hard to know which strains are being influenced (Fletcher et al., 2007). Many researchers thus gather two or even three of these strains together into one effective term, the transient creep strain ($\varepsilon_{tr} + \varepsilon_{creep}$) or the load induced strain LITS ($\varepsilon_{tr} + \varepsilon_{creep} + \varepsilon_\sigma$). Constitutive models were developed by, among others, Anderberg and Thelandersson in 1976, Khoury et al. in 1983 and 1985, Terro 1991 and 1998, Schneider in 1988 and Li and Purkiss in 2005 (Anderberg, 2004 and Fletcher et al., 2007).

Researchers also stress the importance of load history on the stress-strain curve (Anderberg, 2004 and Khoury, 2000). Tests have shown that both compressive strength and elastic modulus reduce far less with increase in temperature when
the concrete was loaded before and during heating. Khoury (2000) believes that this beneficial effect is a result of the compressive stresses that ‘compacts’ the concrete during heating and inhibits the developments of cracks. Anderberg (2004) states that concrete has a memory. This effect should be incorporated in the constitutive model. However, its influence is again not fully appreciated by structural engineers and is not adequately covered in codes and standards (discussed in Chapter 8).

**Validation**

The validity of an advanced calculation methods must be verified. Firstly, the results of the calculation methods, i.e. temperatures, deformations and/or fire resistance times, shall be verified with test results. Secondly, critical parameters, e.g. buckling length, the size of the elements or load level, shall be checked with the aid of a sensitivity analysis to ensure that the model complies with sound engineering principles.

**6.9.4 Tabulated Data**

This method of design gives recognised design solutions based on empirical data, combined with experience and theoretical evaluation of test results. The data was obtained for the more common structural elements, using approximate conservative assumptions. The tables cover only a limited number of cases, but more specific tabulated data may be found in product standards for some specific types of concrete, or they may also be developed based on the simplified or advanced calculation models. Some aspects which are important for the application of the tables are explained here, according to the Eurocode and Denoël (2007). For the actual tables and their respective specific design rules, the reader is referred to Chapter 5 of EN 1992-1-2. Tables exist for the design of columns, walls (load-bearing and non load-bearing), tensile members, beams and slabs.

**Application domain**

The application domain of this method is limited. The values in the tables are only based on the standard fire, with an exposure up to 240 minutes, and the limitations of member analysis apply. Furthermore, the data is obtained for normal weight concrete (2000 to 2600 kg/m³) made with siliceous aggregates. When calcareous or lightweight aggregates are used in beams or slabs, the values for the minimum dimension of the cross-section may be reduced by 10 %. In case of high strength concrete, attention should be made that some extra rules apply to
the use of the tables (see Section 6.10). Furthermore, each type of element comes with its own specific application domain which is given with the tables.

An advantage of this method is that it requires little extra work. No further checks must be made concerning shear and torsion capacity and anchorage details. Also spalling is already included in the design, except for the surface reinforcement (see 6.9.5). Joints, however, still need to be checked.

**Use of the tables**

The tables give pairs of values, written as $x/x$, as a function of required fire resistance and, possibly, the load level. The first number represents the minimum required dimension of the cross-section of the element ($b_{\text{min}}$). The second number gives the axis distance $a$ of reinforcing or prestressing steel from the nearest exposed surface. Several combinations of minimal dimension and axis distance are given. A bigger cross-section could correspond to a smaller axis distance, and vice versa. This is based on the observation that in a more massive cross-section, the heath can be transferred to the core of the element instead of piling up in the peripheral zone where the reinforcement is situated.

The axis distance $a$ is a nominal value. The associated nominal cover then equals the value of the axis distance minus half of the reinforcing diameter minus the diameter of a possible stirrup. As a reminder, the concrete cover that can be found on plans is the nominal cover and corresponds to the height of the spacers. It equals the sum of the minimal concrete cover and the installation tolerance (typically 10 mm for cast in situ concrete and 5 mm for prefabricated concrete). A value of 5 mm may be deduced for high strength concrete and plates (see EN 1992-1-1).

**Load level**

When designing according to tabulated data, some tables include a load level $\mu_{fi}$. This is defined as:

$$ E_{d,fi} = \mu_{fi} R_d $$  \hspace{1cm} (6.42)

Where:

- $R_d$ is the design value of the resistance of the member at normal temperature
- $\mu_{fi}$ is the load level or degree of utilisation for fire design.
The axis distance and critical temperatures

The tables are based on a critical steel temperature: 500°C for reinforcing steel, 400°C for prestressed bars and 350°C for prestressed wires and strands (Denoël, 2007). These values are not randomly chosen. A loading rate 0.7 for the fire situations is assumed. Meaning that, in case the other factors stay equal, the steel is only loaded with 70% of the maximum allowed stress at normal temperature (cold situation). Furthermore, at high temperatures, the steel can operates at a stress which is 1.15 times bigger than the corresponding stress in the cold situation. This follows from the different safety coefficient: 1.15 for the cold situation and 1.00 for the fire situation. Thus, in order for the steel to support the load, the residual strength fraction required when hot is given:

$$k_{s,\text{required}} = \frac{\eta_{\text{fs}}}{\gamma_s / \gamma_{s,\text{fi}}} = 0.7 / (1.15 / 1.0) = 0.609$$  \hspace{1cm} (6.43)

In the Eurocode, a figure is included, giving reference curves for the critical steel temperature $\theta_{cr}$ corresponding to the reduction factor $k_{s}(\theta_{cr}) = \sigma_{s,\text{fi}} / f_{yk}(20^\circ\text{C})$ or $k_{p}(\theta_{cr}) = \sigma_{p,\text{fi}} / f_{pk}(20^\circ\text{C})$, which is reproduced in Figure 6.25. This figure is meant to be used with the tables given in the Eurocode. The curves are slightly different than the ones given in 6.8.2, since they are based on tests carried out according to different procedures. Now, it can be read from the graph that the value of $k_{s,\text{required}} = 0.609$ corresponds to a reinforcing steel temperature of 500°C. The same is done for prestressed steel, which gives a temperature of 400°C for prestressed bars (curve 2) and 350°C for wires and strands (curve 2).

These differences in critical temperatures may be expressed as an increase of the axis distance. This way, the values in the tables that only apply to reinforcing steel can easily be conversed for prestressing steel. This can be explained using Figure 6.26, which gives the temperature distribution inside a slab after 90 minutes of fire exposure. The temperature at a distance of 30 mm from the exposed surface equals 500°C. At that same time, the temperature a point, laying 15 mm deeper, equals 350°C which corresponds to the critical temperature for wires and strands. Thus the axis distance found in the tables may be increased with 10 mm or 15 mm for prestressed bars or wires and strands respectively.
Figure 6.25: Reference curves for the critical steel temperature $\theta_{cr}$ corresponding to the reduction factor $k_s(\theta_{cr}) = \frac{\sigma_{s,fi}}{f_{ys}(20^\circ C)}$ or $k_p(\theta_{cr}) = \frac{\sigma_{p,fi}}{f_{pk}(20^\circ C)}$. Redrawn from EN 1992-1-2.

Figure 6.26: Temperature profile of a slab (height $h = 200$) for R90. $x$ is the distance from the exposed surface. Based on a figure from EN 1992-1-2 (2004) and Denoël (2007).
6.9.5 Additional points of attention

These additional points of attention should be checked or incorporated in the structural fire design of concrete structures. The rules are given as they are stated in the Eurocode. These serve mostly for simplified and advanced calculation methods, since several aspects are already included in the tabulated data.

**Shear, torsion and anchorage**

Shear, torsion and anchorage may be examined according to a calculation method that is supported by test information. An example of a simplified calculation method may be found in Annex C of EN 1992-1-2. This check must not be made when designing with tabulated data.

**Explosive spalling**

Explosive spalling must either be avoided or its effects on the performance requirements (R and/or EI) must be taken into account. The Eurocode (EN 1992-1-2, 2004) states one simple rule: explosive spalling is unlikely to occur if the value of the moisture content of the concrete is lower than $k$ % by weight. The value of $k$ may be found in the National Annex of the respective Member State, but the recommended value is $k = 3$, which must also be used according to National Annexes of Sweden and Belgium. When the moisture content exceeds this value, a more accurate assessment of moisture content, type of aggregate, permeability of concrete and heating rate should be considered.

For beams, slabs and tensile members where the moisture content of the concrete is more than $k$ % by weight, the following procedure may be followed. An examination of the effect of the explosive spalling on the load-bearing function (R) can be done by assuming local loss of cover to one reinforcing bar or bundle of bars in the cross-section and then checking the reduced load-bearing capacity of the section. The temperature of the other, unexposed, reinforcing bars may be taken as the temperature prior to the event of spalling. Another way of checking the behaviour of an element with relation to spalling behaviour is experimentally or by applying a complementary protection which is verified by testing.

No further checks must be made when designing with tabulated data. Also, for a member that is designed for exposure class X0 and XC1 (practically meaning: concrete inside buildings with low air humidity, or concrete permanently submerged in water, see EN 1992-1-1) spalling must not be checked, since the moisture content is assumed to be between 2.5 and 3 %, less than $k$ % by weight.
**Falling off of concrete**
Falling off of concrete must either be avoided or its effects on the performance requirements (R and/or EI) must be taken into account. When the axis distance to the reinforcement is less than 70 mm, falling off of concrete must not be feared. When this is not the case, either tests must be carried out to show that falling off doesn’t occur or an extra surface reinforcement must be provided. This reinforcement should have a mesh with a spacing not greater than 100 mm, and a diameter not less than 4 mm.

**Joints**
Joints must be designed based on an overall assessment of the structure and its structural behaviour. The detailing of a joint must be done in compliance with the R and EI criteria that are required for the different members that the joint connects. Stability of the total structure must also be ensured. The fire design rules for joint components of structural steel may be found in EN 1993-1-2. The Eurocode also determines the size of gaps in joints based on the insulation (I) criterion. The width of a gap should be smaller than 20 mm and the gap should not be deeper than half of the thickness d of the actual separating component. When a gap is deeper than this limit and, if necessary, with an additional sealing product, the fire resistance should be documented in the basis of an appropriate test procedure.

**Protective layers**
Protective layers may be applied to achieve the required fire resistance. The material used for this layer and its properties and performance should be checked with an appropriate test procedure.

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**6.10 High strength concrete**
Chapter 6 of EN 1992-1-2 (2004) gives additional rules for high strength concrete that are summarized here. The given properties and recommendations only apply to a fire exposure that corresponds to a standard fire.

**6.10.1 Material properties**
As with normal strength concrete, the strength of high strength concrete diminishes with increasing temperature. The values of the strength reduction \( f_{c,d}/f_{c,ck} \) may be given in the National Annex. The Eurocode also provides values for the reduction in strength for three classes of high strength concrete, but notes that
these values are based on a limited number of test results. The recommended division of classes is: class 1 for C55/67 and C60/75, class 2 for C70/85 and C80/95, and class 3 for C90/105. An alternative selection of the classes and/or a limit of use may be specified in the National Annex. Both Sweden and Belgium made the use of the classes as given here normative. These values of the strength reduction are given in Table 6.5 and illustrated in Figure 6.27. Note, that the higher the strength of the concrete, the bigger the reduction at elevated temperatures. Thus, it is necessary to check if the real strength will equal the strength specified in the design. If there is a chance that the real strength will be higher, a bigger strength reduction must be used in the fire design.

Table 6.5: Strength reduction \(f_{c,\theta}/f_{ck}\) of high strength concrete at elevated temperatures. From EN 1992-1-2.

<table>
<thead>
<tr>
<th>Concrete temp. (\theta^\circ\text{C})</th>
<th>(f_{c,\theta}/f_{ck})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class 1</td>
</tr>
<tr>
<td>20</td>
<td>1.00</td>
</tr>
<tr>
<td>50</td>
<td>1.00</td>
</tr>
<tr>
<td>100</td>
<td>0.90</td>
</tr>
<tr>
<td>200</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>0.90</td>
</tr>
<tr>
<td>300</td>
<td>0.85</td>
</tr>
<tr>
<td>400</td>
<td>0.75</td>
</tr>
<tr>
<td>500</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td></td>
</tr>
<tr>
<td>700</td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>0.15</td>
</tr>
<tr>
<td>900</td>
<td>0.08</td>
</tr>
<tr>
<td>1000</td>
<td>0.04</td>
</tr>
<tr>
<td>1100</td>
<td>0.01</td>
</tr>
<tr>
<td>1200</td>
<td>0.00</td>
</tr>
</tbody>
</table>
The thermal properties given for normal strength concrete (see Section 6.8.1) may be used for high strength concrete. Separate rules may apply for the thermal conductivity which only given by an upper and lower limit in the Eurocode. These rules must then be stated in the National Annex. The Swedish National Annex specifies the same choice as for normal strength concrete, the lower limit, unless another model can be proved by testing. In Belgium the choice of the thermal conductivity is free, as long as the chosen values are higher than the ones specified for normal strength concrete.

6.10.2 Spalling

The design rules for spalling of normal strength concrete, given in Section 6.9.5 may be used for concrete grades C55/67 to C80/95, provided that the maximum content of silica fume is less than 6% by weight of cement. For higher contents of silica fume, or for stronger concrete, up to C90/105, the Eurocode states that spalling can occur in any situation where this type of concrete is exposed to fire. More drastic measurements are thus required. Four methods are provided and at least one should be applied:

- Method A: A reinforcement mesh with a nominal cover of 15 mm. This mesh should have wires with a diameter $\geq 2$ mm with a pitch $\leq 50 \times 50$ mm. The nominal cover to the main reinforcement should be $\geq 40$ mm.
- Method B: A type of concrete for which it has been demonstrated (by local experience or by testing) that no spalling of concrete occurs under fire exposure.

- Method C: Protective layers for which it is demonstrated that no spalling of concrete occurs under fire exposure.

- Method D: Include in the concrete mix more than 2 kg/m\(^3\) of monofilament propylene fibres.

The selection of methods may be specified in the National Annex of the Member State. For example, in Sweden, method A is not allowed. In Belgium, method A is also discouraged. A reinforcement mesh may only be used as a method against spalling if it is experimentally confirmed. The reason for this is that it is difficult to keep the reinforcement mesh in its place during the pouring of the concrete (Denoël, 2007).

6.10.3 Structural design

Calculation of the load bearing capacity of high strength concrete structures must consider the following: the thermal exposure and the consequent temperature distribution in the members, reduction of the strength due to elevated temperatures, effects of restraint forces due to thermal expansion and second order effects. To achieve this, the Eurocode allows two calculation methods: a global analysis which is based on verified information or a simplified member calculation. The latter may be either a simplified calculation method based on cross-section reduction according to the methods given in Annex B of EN 1992-1-2, i.e. 500°C isotherm method or zone method, or it may be tabulated data. However, some modifications should be made when using these methods due to the fact that high strength concrete loses its strength much quicker than normal strength concrete. Thus for example, when applying the 500°C-isotherm method, the isotherm temperature is actually lower than 500°C. Eurocode specifies 460°C for Class 1 and 400°C for Class 2. For Class 3 more accurate methods are recommended. These values can also be specified differently in the National Annex, however in Belgium and Sweden the recommendations should be used. Design may be based on the original design charts, tables, etc. for normal strength concrete, after which the results simply need to be multiplied by a certain factor, as given in the Eurocode.
Chapter 7. Examples of design

This Chapter shows some calculations to help demonstrate the design methods of the Eurocode. Some individual components are designed using the simplified calculation method with zero strength for concrete above 500°C. More information about the design according to this method can be found in Annex B of EN 1992-1-2. However, with the knowledge of normal temperature design, these examples can be easily followed. Further assumptions have been made to simplify the calculations. It is assumed that concrete has no tensile strength. The parabolic compressive stress block in the concrete is approximated by an equivalent rectangle, calculated with a characteristic strength that is assumed to be 85% of the crushing strength of concrete. Compression reinforcement for beams and slabs are conservatively ignored. Note also that the bending moment diagrams are plotted on the tension side of the flexural members. A moment that causes tension on the underside of the beam is called a positive moment whereas a negative moment causes tension on the top of the beam. All the following examples are based on examples from Buchanan (2002), but the strategy, values and symbols have been modified to comply with the Eurocode.

A small recapitulation of the basic rules from the Eurocode, used in these examples:

Combination of actions for ultimate limit state design (main combination):

\[ \gamma_G G_k \oplus \gamma_{Q,i} Q_{k,i} \oplus \sum_{i=1}^{n} \gamma_{Q,j} \psi_{0,i} Q_{k,j} \]  
(7.1)

Combination of actions for fire situation:

\[ G_k \oplus \psi_{2,i} Q_{k,i} \oplus \sum_{i=1}^{n} \psi_{2,j} Q_{k,j} \]  
(7.2)

where the recommended value of \( \psi_{2,1} Q_{k,1} \) has been chosen (see Section 6.7.2). The partial safety factor for permanent actions \( \gamma_G \) equals 1 when the action is favourable and 1,35 when it is unfavourable. In case of variable actions, \( \gamma_Q = 0 \) or 1,50 if the action has a favourable or unfavourable effect respectively. The factors \( \psi \) were chosen for domestic, residential spaces in building: \( \psi_0 = 0,7; \psi_1 = 0,5 \) and \( \psi_2 = 0,3 \).
The design values of the strength properties of concrete and reinforcing steel for normal temperature design:

\[ f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} \]  

(7.3)

\[ f_{yd} = \frac{f_{yk}}{\gamma_s} \]  

(7.4)

with the partial safety factors \( \gamma_c = 1.5 \) and \( \gamma_s = 1.15 \). The recommended value \( \alpha_{cc} = 1 \) is used.

The mechanical material properties for the fire situation (see Section 6.8.4) are:

\[ X_{d,fi} = \frac{k_x X_k}{\gamma_{M,fi}} \]  

(7.5)

where \( \gamma_{M,fi} = 1.0 \) (recommended value). The reduction factor \( k_x \) can be found in 6.8 and 6.9.2.

In all examples, concrete with siliceous aggregates and cold worked reinforcing steel has been used. The recommended class N for reinforcing steel has been chosen.

### 7.1 Example 1: Simply supported reinforced concrete slab

A simply supported reinforced concrete slab is exposed to a standard fire. The span, load, geometry and reinforcing are known. Reference is made to Figure 7.1. Figure 7.1: Simply reinforced slab exposed to fire of Example 1. Check the flexural capacity after 60 minutes fire exposure. Use Wickström’s formula (see Section 6.9.2 and Eq. (6.35) till (6.39)) to calculate the reinforcing temperature.

**Given information**

- Slab span \( l = 7.0 \text{m} \)
- Slab thickness \( h = 200\text{mm} \)
- Concrete density \( \rho = 24\text{kN/m}^2 \)
- Concrete strength \( f_{ck} = 30\text{MPa} \)
- Yield stress \( f_{yk} = 300\text{MPa} \)
- Bar diameter \( \varphi = 16\text{mm} \)
- Bar spacing \( s = 125\text{mm} \)
- Bottom cover \( c = 15\text{mm} \)
Chapter 7. Examples of design

Dead load (exl. self weight) \( G_1 = 0.5 \text{kN/m} \)
Live load \( Q = 2.5 \text{kN/m} \)

![Diagram of a simply reinforced slab exposed to fire]

Figure 7.1: Simply reinforced slab exposed to fire of Example 1.

**Solution**

Design a 1 m wide strip \( b = 1000 \text{mm} \)
Self weight \( G_2 = \rho b h = 4.8 \text{kN/m} \)
Total dead load \( G = G_1 + G_2 = 0.5 + 4.8 = 5.3 \text{kN/m} \)
Steel Area \( A_s = \pi r^2 b / s = 1608 \text{m}^2 \)
Effective depth \( d = h - c \cdot \varphi / 2 = 177 \text{mm} \)
Effective cover \( c_e = c + \varphi / 2 = 23 \text{mm} = 0.023 \text{m} \)

**PART 1: COLD CALCULATIONS (for a 1 m wide strip)**

- **Calculate the design loads and bending moment**

  Design load \( w_{\text{cold}} = 1.35 G + 1.50 Q = 11 \text{kN/m} \)
  Bending moment \( M_{E,\text{cold}} = w_{\text{cold}} l^2 / 8 = 11 \times 7^2 / 8 = 67 \text{kN/m} \)

- **Calculate the bending strength**

  Design concrete strength \( f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_c = 1 \times 30 / 1.5 = 20 \text{MPa} \)
  Design yield stress \( f_{yd} = f_{yk} / \gamma_s = 300 / 1.15 = 261 \text{MPa} \)
  Stress block depth \( a = A_s f_{yd} / (0.85 f_{cd} b) \)
  \[ a = 1608 \times 261 / (0.85 \times 20 \times 1000) = 25 \text{mm} \]
  Internal lever arm \( z = d - a / 2 = 177 - 25 / 2 = 165 \text{mm} \)
  Bending strength \( M_{R,\text{cold}} = A_s f_{yd} z = 1608 \times 261 \times 165 / 10^6 = 69 \text{kNm} \)
  \( M_{R,\text{cold}} > M_{E,\text{cold}} \) so design is OK.
PART 2: FIRE CALCULATIONS

- Calculate the design loads and bending moment

Design load  
\[ w_{fi} = G + 0,3Q = 6\text{kN/m} \]

Bending moment  
\[ M_{E,fi} = w_{fi} l^2 / 8 = 6 \times 7^2 / 8 = 37\text{kNm} \]

- Calculate the bending strength

Partial safety factor  
\[ \gamma_{M,fi} = 1,0 \]

60 minutes of fire exposure  
\[ t = 60\text{min} \quad (t_h = 1,0\text{ hour}) \]

Fire temperature  
\[ \Theta_g = 20 + 345 \log(8t + 1) = 945\text{°C} \]

Surface temperature  
\[ \Theta_m = \left[ 1 - 0,0616 \times t_h^{0,88} \right] \Theta_g = \left[ 1 - 0,0616 \times 1,0^{0,88} \right] \times 945 = 887\text{°C} \]

Concrete temperature at \( x = c_c \)  
\[ \Theta_c = \left[ 0,18 \ln(t_h / c_c^2) - 0,81 \right] \Theta_m = \left[ 0,18 \ln(1,0 / 0,023^2) - 0,81 \right] \times 887 = 486\text{°C} \]

Steel temperature  
\[ \Theta_s = \Theta_c = 486\text{°C} \]

Reduction factor  
\[ k_s(\Theta_s) = 0,7078 \]

(from Table 6.3)

Design reduced yield stress  
\[ f_{yd,fi} = k_s f_{yk} / \gamma_{M,fi} = 212\text{MPa} \]

Design concrete strength  
\[ f_{cd} = f_{ck} / \gamma_{M,fi} = 30\text{MPa} \]

Stress block depth  
\[ a = A_s f_{yd,fi} / (0,85 f_{ct} b) = 1608 \times 212 / (0,85 \times 30 \times 1000) = 13\text{mm} \]

Internal lever arm  
\[ z = d - a / 2 = 177 - 13 / 2 = 170\text{mm} \]

Bending strength  
\[ M_{R,fi} = A_s f_{yd,fi} z = 1608 \times 271 \times 170 / 10^6 = 58\text{kNm} \]

\[ M_{R,fi} > M_{E,fi} \text{ so design is OK.} \]

Note here, in the case of a simply supported slab, that the temperature of the concrete compressive zone is assumed to be low enough, i.e. under 500°C, so that no reduction is necessary. This is a reasonable assumption since only one side of the slab, i.e. the bottom, is exposed to the fire and the compressive stress block is situated in the top of the slab.
7.2 Example 2: Reinforced concrete beam

A simply supported reinforced concrete beam is exposed to a standard fire. The span, load, geometry and reinforcing are known. Reference is made to Figure 7.2. Check the flexural capacity after 90 minutes fire exposure.

**Given information**

- Beam span $l = 15.0\text{m}$
- Beam width $b = 500\text{mm}$
- Beam depth $h = 800\text{mm}$
- Concrete density $\rho = 24\text{kN/m}^2$
- Concrete strength $f_{ck} = 30\text{MPa}$
- Yield stress $f_{yk} = 300\text{MPa}$
- Bar diameter $\phi = 32\text{mm}$
- Number of bars $n = 8$
  (2 rows of 4 bars)
- Bottom cover $c = 25\text{mm}$
- Dead load (exl. self weight) $G_l = 4.0\text{kN/m}$
- Live load $Q = 10.0\text{kN/m}$

**Solution**

- Area of one bar $A_{s1} = \pi r^2 = 804\text{mm}^2$
- Total steel area $A_s = n\pi r^2 = 6434\text{mm}^2$
- Effective depth $d = h - c - 1.5\phi = 800 - 25 - 48 = 727\text{mm}$
- Self weight $G_s = \rho bh = 24 \times 0.5 \times 0.8 = 9.6\text{kN/m}$
- Total dead load $G = G_l + G_s = 4.0 + 9.6 = 13.6\text{kN/m}$

---

**Figure 7.2: Reinforced concrete beam exposed to fire of Example 2.**
PART 1: COLD CALCULATIONS

- **Calculate the design loads and bending moment**

  Design load \[ w_{\text{cold}} = 1,35G + 1,50Q = 1,35 \times 13,6 + 1,50 \times 10 = 33,4 \text{kN/m} \]
  
  Bending moment \[ M_{R,\text{cold}} = w_{\text{cold}} L^2 / 8 = 33,4 \times 15^2 / 8 = 938 \text{kNm} \]

- **Calculate the bending strength**

  Design concrete strength \[ f_{\text{cd}} = \alpha_{cc} \times f_{\text{ck}} / \gamma_{c} = 1 \times 30 / 1,5 = 20 \text{MPa} \]
  
  Design yield stress \[ f_{\text{yd}} = f_{\text{yk}} / \gamma_{s} = 300 / 1,15 = 261 \text{MPa} \]
  
  Stress block depth \[ a = A_{s} f_{\text{yd}} / (0,85 f_{\text{cd}} b) \]
  \[ = 6434 \times 261 / (0,85 \times 20 \times 500) = 198 \text{mm} \]
  
  Internal lever arm \[ z = d - a / 2 = 727 - 198 / 2 = 628 \text{mm} \]
  
  Bending strength \[ M_{R,\text{cold}} = A_{s} f_{\text{yd}} z = 6434 \times 261 \times 628 / 10^6 \]
  \[ = 1055 \text{kNm} \]
  
  \( M_{R,\text{cold}} > M_{E,\text{cold}} \) so design is OK.

PART 2: FIRE CALCULATIONS

![Diagram](image)

*Figure 7.3: Stress distribution for reinforced concrete beam of Example 2.*

- **Calculate the design loads and bending moment**

  Design load \[ w_{\text{fi}} = G + 0,3Q = 13,6 + 0,3 \times 10 = 16,6 \text{kN/m} \]
  
  Bending moment \[ M_{E,\text{fi}} = w_{\text{fi}} L^2 / 8 = 16,6 \times 15^2 / 8 = 467 \text{kNm} \]
Example of design

- **Calculate the bending strength**

Fire duration \( t = 90 \text{ min} \)

Now, the steel and concrete temperatures are determined using the design chart of EN 1992-1-2, Annex A, Figure A.9 (a), that is, the temperature profile \( R_{90} \) for a beam \( h \times b = 800 \times 500 \). The values have been linearly interpolated.

Depth of 500°C isotherm \( c_f = 30 \text{ mm} \)

(assuming one-dimensional heat transfer)

Reduced width \( b_f = b - 2c_f = 500 - 2 \times 30 = 440 \text{ mm} \)

We assume that the concrete with temperature above 500°C has no compressive strength and concrete below 500°C has full compressive strength.

Steel temperatures:
- Bar group (1): < 440°C
- Bar group (2): 100°C
- Bar group (3): 620°C

Reduced yield strength of reinforcing bars at elevated temperatures (from Table 6.3, linear interpolation):

\[
\begin{align*}
      k_{\phi,1} &= 0,832 \\
      k_{\phi,2} &= 1,00 \\
      k_{\phi,3} &= 0,344
\end{align*}
\]

Design reduced yield stress
\[
\begin{align*}
      f_{yd,fi,1} &= k_{\phi,1} f_{y\bar{k}} / \gamma_{M,fi} = 250 \text{ MPa} \\
      f_{yd,fi,2} &= k_{\phi,2} f_{y\bar{k}} / \gamma_{M,fi} = 300 \text{ MPa} \\
      f_{yd,fi,3} &= k_{\phi,3} f_{y\bar{k}} / \gamma_{M,fi} = 103 \text{ MPa}
\end{align*}
\]

\[
f_{yd,fi} = \frac{4 \times A_{s1} \times f_{yd,fi,1} + 2 \times A_{s1} \times f_{yd,fi,2} + 2 \times A_{s1} \times f_{yd,fi,3}}{A_s}
\]

\[= 226 \text{ MPa}\]

Design concrete strength \( f_{cd} = f_{ck} / \gamma_{M,fi} = 30 \text{ MPa} \)

Stress block depth \( a = A_s f_{yd,fi} / (0,85 f_{cd} b_f) \)

\[= 6434 \times 226 / (0,85 \times 30 \times 440) = 129 \text{ mm} \]

Internal lever arm \( z_f = d - a / 2 = 727 - 129 / 2 = 662 \text{ mm} \)

Bending strength \( M_{R,fi} = A_s f_{yd,fi} z_f \)

\[= 6434 \times 226 \times 662 / 10^6 = 963 \text{kNm} \]

\( M_{R,fi} > M_{R,fi} \) so design is OK.
7.3 Example 3: Reinforced concrete T-beam

A reinforced concrete T-beam is continuous over three supports (two spans) and is exposed to a standard fire. It is one of a series of beams 400 mm wide by 800 mm deep, at 4,0 m centres, supporting a 150 mm thick concrete slab, as shown in Figure 7.4 and Figure 7.5. Check the structural adequacy of the beam before and after 3 hours fire exposure. The contribution of compressive reinforcing may be ignored.

**Given information**

<table>
<thead>
<tr>
<th>Property</th>
<th>Bottom</th>
<th>Top</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam span</td>
<td>$l = 13,0m$</td>
<td></td>
</tr>
<tr>
<td>Beam depth</td>
<td>$h = 800mm$</td>
<td></td>
</tr>
<tr>
<td>Web width</td>
<td>$b_w = 300mm$</td>
<td></td>
</tr>
<tr>
<td>Tributary width (for self weight)</td>
<td>$b_t = 4,0m$</td>
<td></td>
</tr>
<tr>
<td>Effective flange width (for positive moment)</td>
<td>$b_e = 2,0m$</td>
<td></td>
</tr>
<tr>
<td>Concrete density</td>
<td>$\rho = 24kN/m^2$</td>
<td></td>
</tr>
<tr>
<td>Concrete strength</td>
<td>$f_{ek} = 30MPa$</td>
<td></td>
</tr>
<tr>
<td>Yield stress</td>
<td>$f_{yk} = 300MPa$</td>
<td></td>
</tr>
<tr>
<td>Live load</td>
<td>$Q = 2,5kN/m^2$</td>
<td></td>
</tr>
</tbody>
</table>

**Reinforcing**

<table>
<thead>
<tr>
<th>Property</th>
<th>Bottom</th>
<th>Top</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars</td>
<td>$n_b = 5$</td>
<td>$n_t = 18$</td>
</tr>
<tr>
<td>Bar diameter</td>
<td>$\phi_b = 28mm$</td>
<td>$\phi_t = 20mm$</td>
</tr>
<tr>
<td>Area of one bar</td>
<td>$A_{sb1} = \pi r^2 = 616mm^2$</td>
<td>$A_{st1} = \pi r^2 = 314mm^2$</td>
</tr>
<tr>
<td>Total steel area</td>
<td>$A_{sb} = n_b A_{sb1} = 3079mm^2$</td>
<td>$A_{st} = n_t A_{st1} = 5655mm^2$</td>
</tr>
<tr>
<td>Cover</td>
<td>$c = 25mm$</td>
<td>$c = 25mm$</td>
</tr>
<tr>
<td>Effective depth</td>
<td>$d_b = h - c - \phi_b / 2 = 761mm$</td>
<td>$d_t = h - c - \phi_t / 2 = 765mm$</td>
</tr>
</tbody>
</table>

*Figure 7.4: Two-span reinforced concrete T-beam of Example 3.*
Chapter 7. Examples of design

Figure 7.5: Cross-section of the reinforced concrete T-beam of Example 3.

Solution

Calculate the design loads for both cold and fire calculations:

Dead load (self weight)\[ G = (0,15 \times 4,0 + 0,65 \times 0,3) \times 24 = 19,1kN/m \]
Live load \[ Q = 2,5 \times 4,0 = 10,0kN/m \]
Load combination for cold conditions \[ w_{\text{cold}} = 1,35G + 1,50Q = 40,8kN/m \]
Load combination for fire conditions \[ w_{\text{fi}} = G + 0,3Q = 22,1kN/m \]
**PART 1: COLD CALCULATIONS**

Design concrete strength  
\[ f_{cd} = f_{ck} / \gamma_c = 1 \times 30 / 1.5 = 20 \text{MPa} \]

Design yield stress  
\[ f_{yd} = f_{yk} / \gamma_s = 300 / 1.15 = 261 \text{MPa} \]

**NEAR MID-SPAN**  
(positive moment)

Elastic bending moment  
\[ M_{E,m} = 9w_{cold}l^2 / 128 \]
\[ = 485 \text{kNm} \]

Stress block  
\[ a_m = A_{sb}f_{yd} / (0.85f_{cd}b_e) \]
\[ = 3079 \times 261 / 0.85 \times 20 \times 2000 \]
\[ = 24 \text{mm} \]

Internal lever  
\[ z_b = d_b - a_m / 2 \]
\[ = 761 - 24 / 2 \]
\[ = 749 \text{mm} \]

Design flexural strength  
\[ M_{R,m} = A_{sb}f_{yd}z_b \]
\[ = 3079 \times 261 \times 749 / 10^6 \]
\[ = 602 \text{kNm} \]

**AT SUPPORT**  
(negative moment)

Elastic bending moment  
\[ M_{E,s} = w_{cold}l^2 / 8 \]
\[ = 862 \text{kNm} \]

Stress block  
\[ a_s = A_{st}f_{yd} / (0.85f_{cd}b_w) \]
\[ = 5655 \times 261 / 0.85 \times 20 \times 300 \]
\[ = 289 \text{mm} \]

Internal lever  
\[ z_t = d_t - a_s / 2 \]
\[ = 765 - 289 / 2 \]
\[ = 620 \text{mm} \]

Design flexural strength  
\[ M_{R,s} = A_{st}f_{yd}z_t \]
\[ = 5655 \times 261 \times 620 / 10^6 \]
\[ = 915 \text{kNm} \]

\[ M_{E,m} < M_{R,m} \text{ and } M_{E,s} < M_{R,s} \text{ so design is OK.} \]

**PART 2: FIRE CALCULATIONS**

- **NEAR MID-SPAN**

\[ N_{cd} = 0.85f_{cd}a_m b_e \]
\[ N_{sd} = A_{sb}f_{yd}f_t \]

*Figure 7.7: Stress distribution near mid-span of the reinforced concrete T-beam of Example 3.*
Chapter 7. Examples of design

Elastic bending moment

\[ M_{E,m,fi} = 9w_{f1}f^2 / 128 = 263\text{ kNm} \]

Effective cover to bottom bars

\[ c_e = c + \varphi_b / 2 = 25 + 28 / 2 = 39\text{ mm} \]

Steel bar temperatures from the design chart EN 1992-1-2, Annex A, Figure A.8.

- Bar group (1) (corner bars) 710°C
- Bar group (2) (interior bars) <550°C

Reduced yield strength (from Table 6.3, linear interpolation)

\[ k_{\sigma,1} = 0,119 \]
\[ k_{\sigma,2} = 0,535 \]
\[ f_{yd,fi,1} = k_{\sigma,1}f_{ytk} / \gamma_{M,fi} = 36\text{ MPa} \]
\[ f_{yd,fi,2} = k_{\sigma,2}f_{ytk} / \gamma_{M,fi} = 161\text{ MPa} \]
\[ f_{yd,fi} = \frac{2 \times A_{sb1} \times f_{yd,fi,1} + 3 \times A_{sb1} \times f_{yd,fi,2}}{A_{sb}} = 111\text{ MPa} \]

Design concrete strength

\[ f_{cd} = f_{ck} / \gamma_{M,fi} = 30\text{ MPa} \]

Stress block depth

\[ a_m = A_{sb}f_{yd,fi} \times (0,85f_{cd}b_e) = 3079 \times 111 \times (0,85 \times 30 \times 2000) = 7\text{ mm} \]

Note that no reduction has been made for the concrete strength. The concrete compressive zone starts 150 – 7 = 143 mm from the exposed surface, i.e. the bottom of the concrete slab. From the appropriate design chart from Annex A of EN 1992-1-2, it is read that the temperature of the zone is under 100°C, well below the 500°C-isotherm.

Internal lever arm

\[ z_b = d_b - a_m / 2 = 761 - 7 / 2 = 757\text{ mm} \]

Bending strength

\[ M_{R,m,fi} = A_{sb}f_{yd,fi}z_b = 3079 \times 111 \times 757 / 10^6 = 259\text{ kNm} \]

\[ M_{R,m,fi} < M_{k,m,fi} \] so the cross-section fails.
AT THE SUPPORT

Figure 7.8: Stress distribution at the support of the reinforced concrete T-beam of Example 3.

Elastic bending moment \( M_{E,s,fi} = w_{\text{fire}} \frac{L^2}{8} = 467 \text{kNm} \)

The top reinforcing bars lay at a depth of \( 150 - 25 - 20/2 = 115 \text{ mm} \). Their temperature is much less than 300°C, thus no reduction in strength.

We assume that the concrete with temperature above 500°C has no compressive strength and concrete below 500°C has full compressive strength. For this, the temperature profiles of Annex A of EN 1992-1-2 (2004) were used.

Depth of 500°C isotherm \( c_{fi} = 37 \text{mm} \) (assuming one-dimensional heat transfer)

Reduced width of stress block \( b_{w,fi} = b_w - 2c_{fi} = 300 - 2 \times 37 = 226 \text{mm} \)
Reduced effective depth \( d_{t,fi} = d_t - c_{fi} = 765 - 37 = 728 \text{mm} \)

Design reduced yield stress \( f_{yd,fi} = f_{yk} / \gamma_{M,fi} = 300 \text{MPa} \)
Design concrete strength \( f_{cd} = f_{ck} / \gamma_{M,fi} = 30 \text{MPa} \)
Stress block depth \( a_s = A_{si} f_{yd} / (0.85 f_{cd} b_{w,fi}) \)
\[ = 5655 \times 300 / (0.85 \times 30 \times 226) \]
\[ = 294 \text{mm} \]

Internal lever arm \( z_t = d_{t,fi} - a_s / 2 = 728 - 294 / 2 = 581 \text{mm} \)
Bending strength \( M_{R,s,fi} = A_{si} f_{yd,fi} z_t \)
\[ = 5655 \times 300 \times 581 / 10^6 = 986 \text{kNm} \]
These calculations show that the fire has caused the flexural capacity at mid-span to drop below the elastic bending moment which would cause failure, if this beam would be simply supported. Due to this beam’s continuity, moment redistribution is possible. This will now be checked.

**PART 3: MOMENT REDISTRIBUTION**

It can be shown (Gustaferro and Martin, 1988) that for the end bay of a multi-span beam of length \( l \) (m) with uniformly distributed load \( w \) (kN/m), if the positive plastic moment capacity \( M_p^+ \) (kNm) is known, the negative moment \( M^- \) (kNm) at the interior support is given by:

\[
M^- = w l^2 / 2 - w l^2 \sqrt{2M_p^+ / w l^2}
\]

This is also illustrated in Figure 7.9 for a two-span beam. Now Eq. (7.6) can be used to calculate the redistributed moment at the support when the mid-span moment is taken equal to the flexural strength in fire conditions.

\[
M^+ = M_{E,m,red} = M_{R,m,fi} = 259\text{kNm}
\]

\[
M_{E,s,red} = M^- = w l^2 / 2 - w l^2 \sqrt{2M_p^+ / (w l^2)} = 22.1 \times 13^2 / 2 - 22.1 \times 13^2 \times \frac{2 \times 259}{22.1 \times 13^2} = 477\text{kNm}
\]

\[
M_{E,s,red} < M_{R,s,fi}
\]

This shows that the design is OK, because the bending moment can be redistributed to the line shown by \( M_{\text{fire,red}} \) in Figure 7.6. The bending moment near mid-span is decreased, and consequently, the bending moment at the support is increased. The maximum negative moment is now 477 kNm, still less than the flexural capacity of 986 kNm.

The location \( x \) (m) of the maximum mid-span moment can also be calculated (Gustaferro and Martin, 1988) from the end support (see Figure 7.9):

\[
x = \sqrt{2M_p^+ / l} = \sqrt{2 \times 259 / 13} = 6.3\text{m}
\]
The termination of the bottom reinforcing bars must be checked to determine if it is possible to develop full flexural strength at this location.

Figure 7.9: Bending moment diagram for a two-span beam.
8 Discussion, conclusions and further research

Here, the knowledge gained through the previous Chapters is used to take a critical look at how concrete structures are designed for the fire situation today. Additionally, the status of the research is discussed and suggestions are made as to where more work is needed, in order to eventually create a concrete model that can realistically represent both the material as the structural behaviour. The ideas described here were formed during the process of writing this work and studying the many books and papers. However Fletcher et al. (2007) and fib Bulletin 46 (2008) served as an additional inspiration in writing this Chapter

8.1 Fire design

The Eurocode provides a number of procedures in order to design concrete structures for the fire situation, both prescriptive as performance based. However, of the latter, only the basic principles are given and several gaps still need to be filled through research. Thus in practical design, the most common used procedures are either tabulated data or a simplified calculation method of members with a reduced cross-section and temperature profiles based on a nominal heating curve.

The use of these nominal fires certainly has its advantages. They are easy to adopt due to their simple temperature-time relationship, and the standard fire provides a base for the many empirical developed design methods and models that are widely used. Material properties, tabulated data, charts with temperature distributions, etc., are all given in the Eurocode and developed through standard fire testing. However, the question now arises: can realistic fires be properly represented by the standard fire? The answer is no. So many factors play a role in realistic fire loads like the quantity of the fuel, the dimensions of the compartment, the conditions of ventilation, and many more, whereas the standard fire curve only depends on time. Granted, designers can choose another nominal fire curve, but the three choices given in the Eurocode do not make up a wide range. Nominal fires also assume a uniform temperature field inside the compartment, thus representing a fully-developed fire. This post-flashover fire is certainly very important in assessing the fire behaviour of a structure, however,
pre-flashover behaviour is of the essence when considering life safety. After all, this stage is the only stage where evacuation is possible. Remember, flashover can only occur in an enclosed compartment and even if so, in large compartments a significant amount of time may elapse before occurrence. Localised fires are thus more likely to cause damage in for example big halls or open structures. Another shortcoming of nominal fires is that they increase monotonously and they do not model the cooling down of the fire. The behaviour of the concrete when it cools down, slowly or abruptly due to extinguishing, is thus not included in the design, whereas it is well known that additional damage may occur in this stage.

Furthermore, the Eurocode greatly focuses on the fire resistance of single concrete members and more specifically the bending collapse of members due to fire. More attention should be paid to the whole structure and its behaviour. The most important phenomenon is, without a doubt, the thermal expansion of the members and all the effects that go with it. When axial expansion of a member is restrained, it can greatly enhance or diminish the strength of that member. On the other hand, it may as well cause a great amount of damage when the surrounding structure cannot cope with the expansion and the resulting restraint forces. When thermal expansion may occur free, it can cause important second order effects. This also illustrates that all types of failure modes must be accounted for in the design of concrete structures. Other examples are shear and anchorage failure, loss of bond strength or spalling, which are only very limitedly covered in the Eurocode. This does not always mean that design according to prescriptive methods of the Eurocode may be unsafe, but could also produce results that are too safe and not economic.

From this wide variety of failure modes, it becomes clear that concrete cannot anymore only be identified by its strength when it comes to fire safety. Properties as ductility, thermal expansion, transient strain, etc. also play a significant role. This becomes even clearer when looking at spalling. All of a sudden properties as permeability or moisture transport become highly important to the limit state design of concrete structures. Many of these properties are also influenced by such basic factors as for example concrete mix-design, production process, curing conditions or application, illustrating that the term ‘concrete’ is not appropriate anymore when it comes to fire, but the term ‘concrete type’ should be used. Consequently, an ideal structural fire design should include separate design rules for each concrete type.
More and more, it becomes clear that the design rules that are used today by practising engineers do not sufficiently capture the true behaviour of a concrete structure in a real fire. Either a performance based approach is necessary or new prescriptive rules should be developed. In any way, more research is needed and some areas of further research are discussed below.

### 8.2 Materials

An understanding of the several physical and chemical transformations within heated concrete and the resulting temperature dependency of mechanical as well as thermal properties is key to a good design. Over the years many research has been performed but still more work needs to be done. All these effects and characteristics have been studied through extensive fire testing. However, most tests were performed according to certain predetermined hearmore, the test furnaces in which these tests are performed focus on developing a uniform heating environment. The effect of spatial non-uniformity on the concrete material and, as a consequence, on the concrete structure is thus poorly known. However, this non-uniform heating can cause additional thermal stresses in the members and could play a role in the phenomenon of spalling.

Furthermore, most specimens in these tests are heated unloaded. Studies however showed that both compressive strength and elastic modulus reduce far less with increasing temperature when the concrete was loaded before and during heating (Khoury, 2000). More tests are required to find out how structural effects, e.g. restraint and loading, influence the performance of concrete in fire. This illustrates also the importance of a proper structural analysis which is discussed later. Structural effects are, furthermore, particularly of interest for the study of explosive spalling.

### 8.3 Explosive spalling

Explosive spalling is discussed in Chapter 3 of this work and this discussion is here summarized. Since decades, many studies have been performed, however, the underlying mechanism of explosive spalling is still not known. The multitude of factors that influences explosive spalling and their complex coupling contributes to the intricacy of the subject. Furthermore some of these factors are particularly difficult to measure, as for example the transport of moisture or the pore pressure within the concrete. At this moment, a high amount of research is devoted to the field of spalling, illustrating that it is a ‘hot’ topic. The reason for this
is that there is a real-time demand from the industry for the development of protective measurements, particularly with respect to tunnels that make use of high strength concrete. The best known solution is polypropylene fibres. Its positive effects have been well established and many researchers are now developing guideline for the use of PP-fibres. However, as long as its mechanisms are not fully understood this won’t be much more than simple guesswork and extensive fire testing. Accordingly, the design rules given in the Eurocode are highly simplistic and ignore many factors that are known to have an influence on explosive spalling. The protective measurements against spalling provide very little guidance. For example, only one sentence covers the use of PP-fibres.

8.4 New types of concrete

Every day, researchers are working on the development of new types of concrete or new cementitious materials. Some very well known examples are high strength concrete and self compacting concrete, but also the highly innovative cementitious composites. Other types of reinforcement are examined as well, e.g. glass or carbon fibre reinforcement rather than steel. It should not be forgotten to study these new development with respect to their behaviour in the case of fire. The number of constituents in the mix is rising, and their behaviour on each other should be explored as well as their effect on the micro-structure and thermo-mechanical properties of the concrete.

8.5 Concrete modelling

In Chapter 6, constitutive models were studied and it was concluded that any structural analysis of heated concrete must include transient strain, or its variants transient creep strain or load induced thermal strain. If not, it will be completely inappropriate and will yield erroneous results, particularly for columns exposed to fire, since these are largely in a state of compression. Also the effect of load appeared to be of great influence. Nevertheless, the material model of concrete as presented in the Eurocode and shown in Figure 6.7 ignores these two effects. However, in some cases, the calculated failure times according to these models have shown to predict experimental results fairly well. This is because elements as beams and slabs are exposed to fire from below. The heat will thus only affect the tension zones of the members whereas transient strain solely acts in compression. Furthermore, very large strains on the descending branch were introduced to compensate for the transient strain (Anderberg, 2004). The strains $\varepsilon_c, \theta$ and $\varepsilon_{cu}, \theta$ are much larger than their respective values that can be found in the
literature. This explains the conclusion drawn from Figure 6.19 that a concrete specimen, unloaded yet restrained against thermal expansion, will not rupture, although this is really a consequence of the thermal strain.

A lot of progress has been made on the modelling level, like for example the discovery of transient strain, however, many problems are still awaiting an answer, particularly with respect to detailed behaviours as spalling. Previous modelling studies have often over-simplified the thermal-mechanical behaviour of concrete and thus did not yield acceptable results. Many extended isothermal models to incorporate thermal dependency. Thermal effects were included by adding a temperature dependency to the material properties, as strength and stiffness, of the concrete. However, this fails to capture the complex coupled thermo-mechanical behaviour of the concrete. Furthermore, this approach does not account for the irreversible changes these material properties undergo. To realistically simulate structural concrete, a fully coupled model should be used, combining mechanical, thermal and hydral effects and their influence on each other, as well as a sophisticated structural analysis, including transient strain.

8.6 Conclusions and recommendations

From the previous discussion some general conclusion can be drawn. In many cases, the design methods used in practice do not completely capture the true behaviour of concrete structures in fire. A proper design should incorporate a design fire that is realistic and based upon physical parameters and the structure must be studied as a whole, taking in account all the interactions between the elements. However, a lot of work is still needed in order to incorporate these elements in a practical design tool. Firstly, a systematic study is required of the effects of more realistic thermal exposures on the concrete material. This also includes the behaviour during the cooling down phase of the fire. An understanding of the complex phenomenon of spalling is required in order to be able to correctly predict its occurrence and thus produce a safe design. Therefore still a great amount of experimental data is required, obtained by proper test methods. It is of the author’s believe then only then, protective measurements as polypropylene fibre can be fully optimised. Furthermore, the design rules in the Eurocode concerning explosive spalling are too simplistic. Developing additional rules at this stage is, however, difficult since the underlying mechanism is not known and these rules must thus be empirically developed. However, the Eurocode could also specify a certain test procedure, from which can follow, for example, what amount of PP-fibres is needed to prevent spalling of a specific concrete type. Fur-
thermore, it should not be forgotten to study the fire behaviour of newly developed concrete types.

Besides a better understanding of the material, there is also the need for a proper structural analysis. More research is needed in investigating the different failure modes during, but also, after a fire. Furthermore, the effects of restrained thermal expansion should be incorporated in the design codes. The same applies to thermal transient stress, of which its effects on the structural behaviour still require further in-depth study. In order to understand the behaviour of concrete structures as a whole and to study the impact of small-scale phenomena at full scale, large-scale building tests are necessary, which are invasive and expensive. Another great source of information can be found in real fire incidents. However, better documentation is required for the latter.

![Figure 8.1: Large-scale fire test of an industrial hall, performed at the University of Ghent in 1974.](image)

Finally, the goal is to develop a constitutive model that reflects the true behaviour of concrete structures when exposed to fire, both from a material as a structural point of view. This means that the better the previously discussed elements are understood, the better concrete models can approximate reality. These models can then be used together with finite-element analysis to fully investigate and
predict the behaviour of concrete structures and provide new design codes and methods. To realistically simulate structural concrete, a fully coupled model should be used, combining mechanical, thermal and hydral effects and their influence on each other, as well as a sophisticated structural analysis, including transient strain.
References


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