Fire Safety Analysis:

Theory & technology

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2 Fire Resistance Analysis

2.1 Introduction

Fire resistance evaluation of building structures is by no means an easy task and often puts a strain even on skilled and experienced engineers. Where-as one was limited to simplified and approximating design rules used in the past, PowerFrame now enables the engineer to switch to more advanced analysis techniques and to evaluate structural strength globally taking into consideration the disastrous effects of fire hazards.

Over the last decades, extensive research work has been undertaken to investigate the response of building structures to fire hazards. Those research activities have allowed to develop more advanced design analysis techniques, which have been documented in a considerable number of research reports and which have found their way into new design codes. Those advanced techniques have in the meantime proven to be indispensable when it comes to assessing fire safety levels of building structures & building components with sufficiently high confidence levels.

Fire resistance of building structures can be obtained experimentally or numerically. The numerical approach to this problem is part of **parts 1-2 "Structural Fire Design"** of the several structural Eurocodes.

Where-as EN 1991-1-2 presents the fire loads and related accidental loads combinations, the other parts 1-2 will concentrate on specific analysis rules related to the individual construction materials.

2.2 Thermal loads

In order to calculate the thermal response to a fire hazard, the fire loads must be known. To describe such type of loads, a number of different fire models are mentioned by EN 1991-1-2. The development of such models is a complex task, depending on the number of parameters that are accounted for. More complex fire models will be more computationally intensive, but will a closer match with a natural fire. The table below gives an overview of the different types of possible fire models:

Nominal fire models	Standard fire *
	External fire *
	Hydrocarbon fire *
Basic fire models	Local fire
	Parametric fire *
Advanced fire models	1-zone model
	2-zone model
	Combined 1-zone & 2-zone model
	Field model

* fire models supported by the BuildSoft software

Nominal fire models and parametric fire models are described by a temperature-time curve describing the evolution of ambient temperature as a function of time. Those fire models describe a fully developed fire hazard and assume a uniform temperature at any instant of time within the entire compartment.

While the application of nominal fire curves requires only a limited knowledge on the behaviour of a fire within a compartment, the calculation of temperatures for basic and advanced fire models will also depend on

- fire load & caloric value of all present inflammable materials, expressed in MJ/m² floor surface;
- ventilation conditions;
- dimension and wall composition (including possible openings) of the compartment.

Those fire models consider the entire "life cycle" of a fire hazard, including the decay phase.

2.2.1 Nominal fire curves

Any method based on the use of nominal fire curves requires, thanks to the simplifying hypotheses, only a limited knowledge on the behaviour of a fire hazard within a compartment. It should however be recognized that the use of such methods will deliver fairly conservative designs when more stringent regulations are imposed (for instance, in case the prescribed fire resistance should amount to 60 or 120 minutes).

Eurocodes distinguish three nominal fire curves:

- standard curve (or ISO 834 curve)

$$\theta_g = 20 + 345\log_{10}(8t+1)$$

This model should be used when no extra information on the fire hazard is available. The standard fire has following characteristics:

the fire is assumed to be active within the whole compartment, independent of the actual size of this compartment;
the fire never decays, not even after all combustible materials have been exhausted;
it does not depend on the compartment's fire load nor on ventilation conditions.

Despite this very conservative approach, this model is widely used thanks to its ease of use.

- external fire

$$\theta_g = 20 + 660\left(1 - 0.686e^{-0.32t} - 0.313e^{-3.8t}\right)$$

This model is applicable to the external face of separating walls which are exposed to smoke coming from a fire developing within the compartment that is internal to the walls. Such a fire is characterised by less elevated temperatures and should therefore not be used for structural elements that are exterior to the compartment but that can still be exposed to higher temperatures (e.g. through openings). In this case, a different model should be used.

- hydrocarbon fire

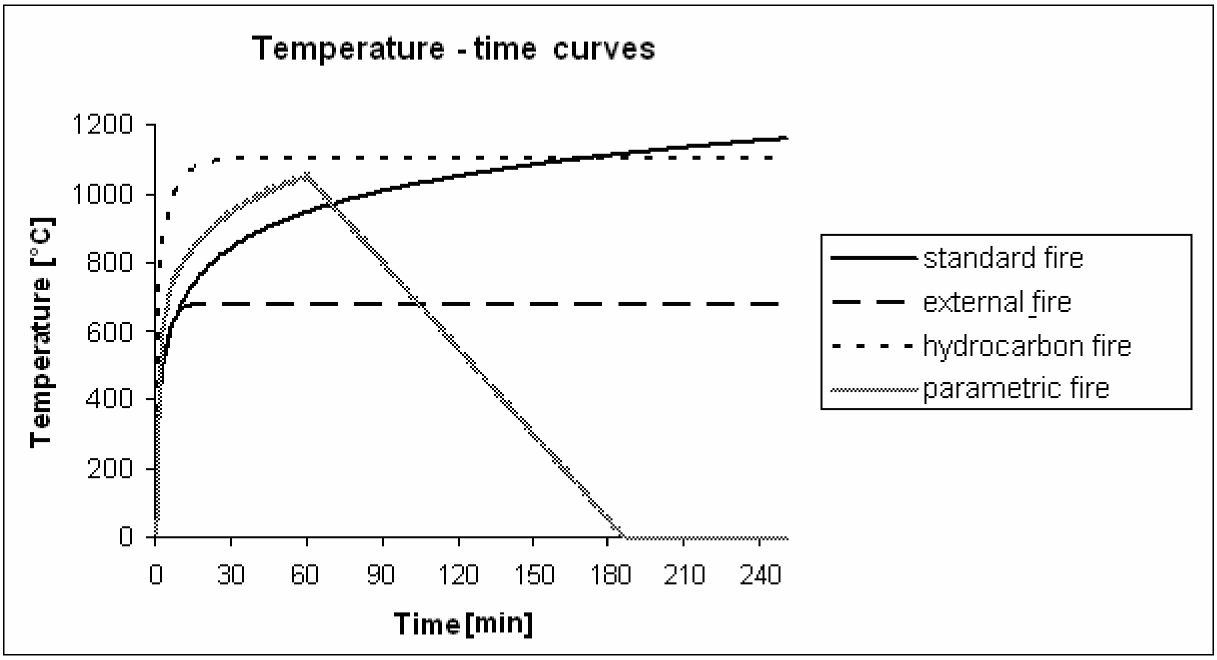
$$\theta_g = 20 + 1080\left(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}\right)$$

This model is applicable to fire hazards which are caused by the ignition of hydrocarbons (fuel, diesel,...) and is characterised by significantly elevated temperatures.

In the above equations, following parameters are used:

- θ_g: ambient (gas) temperature within the compartment or close to the separating wall, in °C;
- t: time, in minutes.

Nominal temperature-time curves are presented graphically in the figure below:



It is observed that both the external fire curve as well as the hydrocarbon fire curve quickly evolve to a maximum, at either 680°C or 1100°C. The standard fire curve continues to raise and delivers higher temperatures compared to the hydrocarbon curve from 180 minutes onwards.

2.2.2 Parametric fire curves

Parametric fires are also described by a temperature-time curve, which however depends on a considerable number of environmental parameters and therefore provides a more realistic approach to how a fire hazard develops and evolves. A parametric fire curve takes into account the compartment's ventilation conditions and thermal properties of its bounding walls. Parametric fire curves furthermore consider the fire's decay phase, thus allowing for a temperature decrease once the fire load has been exhausted. As temperatures are assumed to be uniformly distributed within the compartment, those fire models should in principle only be applied to compartments of a moderate size.

For more information on the derivation of parametric fire curves, reference is made to the (informative) annex A of EN1991-1-2.

2.3 Design actions

Design actions which should be applied to a building structure to account for fire hazards, are obtained by combining the different mechanical and thermal actions (including the fire load), using the appropriate partial safety factors and combination coefficients.

According to EC0, a fire hazard should be considered as an accidental design action, implying that only ultimate limit states must be evaluated.

Accidental combinations can be written as:

$$\sum_{j\geq 1} G_{k,j} + A_d + (\psi_{1,1} \text{of} \psi_{2,1})Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$

in which

…k: characteristic values

- G: permanent loads
- Q_1: the principal variable loads
- Q_i: the remaining variable loads
- ψ_1: combination factor for the frequent part of the variable loads
- ψ_2: combination factor for the quasi-permanent part of the variable loads

All partial safety factors are assigned a unity value. The choice between $\psi_{1,1}Q_{k,1}$ or $\psi_{2,1}Q_{k,1}$ may differ from one country to the other, and should be made as a function of the relevant accidental design combination. Eurocode prefers that the quasi-permanent factor ψ_2 be applied for all variable loads. In some countries (for example, in Belgium), the frequent combination factor ψ_1 should be used for the principal variable loads.

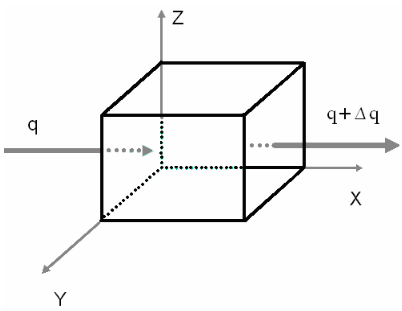
2.4 Thermal response

With increasing gas temperature θ_g, the building structure's temperature will raise as well. One will of course observe a time delay between gas and structural temperature variations. As steel cross-sections are characterised by high thermal conductivity and are relatively thin, such sections will warm up in a fairly uniform way. Concrete sections on the other hand are fairly massive and will usually experience a non-uniform temperature increase.

To describe heat transport within building materials, the laws of thermodynamics should be used. Depending on the material, further simplifications or assumptions can be introduced.

Generally speaking, heat transport within a construction material can be described by the Fourier differential equation:

$$\frac{\partial(\rho c \theta)}{\partial t} + \frac{\partial\left(\lambda \frac{\partial \theta}{\partial x}\right)}{\partial x} = 0 \quad \text{(along the X-direction)}$$



with

- θ temperature in point x [°C]
- ρ mass density [kg/m³]
- c specific heat [J/kgK]
- λ thermal conductivity coefficient [W/mK]

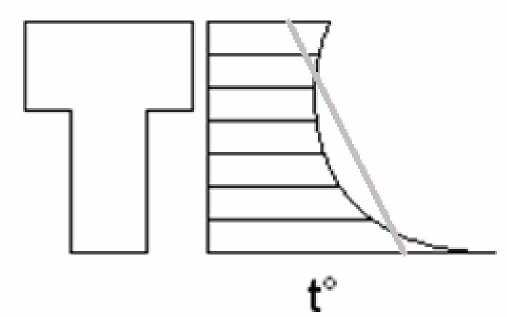
From the above equation, it can easily be understood that the temperature variation within a structural element subjected to a fire hazard is mostly influenced by following thermal properties:

- thermal conductivity λ
- specific heat c

Specific heat c is usually combined with mass density ρ, more commonly referred to as heat capacity $\rho.c$. Both heat capacity and thermal conductivity are strongly dependent on temperature for most common construction materials.

2.4.1 Heat transport in concrete

Given the fact that concrete sections are usually quite massive and that the material “concrete” has a fairly low thermal conductivity, the temperature variations within a concrete cross-section will evolve non-uniformly. It is therefore of the utmost importance to have access to a powerful analysis program, which implements the general Fourier differential equation to calculate temperature variations as a function of point coordinates within the cross-section. A solid knowledge and experience with thermodynamics is required, and is delivered by the thermodynamics specialist Physibel with whom BuildSoft has partnered to develop and deliver a product for fire safety analysis.



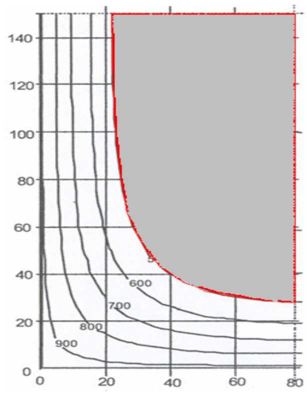
Once the temperature distribution is known, a uniform temperature increase and a (linear) temperature gradient along both principal axes of the section is calculated. Uniform temperature increase and temperature gradients will cause extra deformations in case those deformations are not restrained externally, or will cause internal forces and stresses in the opposite case. Temperature gradients may in particular give raise to significant bending moments.

The thermal properties of concrete materials are evaluated on the basis of the average temperature increase:

- specific heat c_c increases only moderately with increasing temperatures, but can approximately be assumed to have a constant value of 1000 J/kgK.

- thermal conductivity λ_c decreases with increasing temperatures, but can approximately be taken equal to 1,60 W/mK (siliceous granulates), 1,30 W/mK (calcareous granulates) or 0,80 W/mK (light-weight granulates).

It is mentioned here that in case concrete temperatures exceed 500℃, the concrete material is conventionally considered to contribute no longer to cross-section resistance. This conventional approach is justified by the fact that at higher temperatures lumps of concrete material can almost explosively



disconnect, and will therefore reduce the concrete section that remains effectively available during the fire hazard. This reduced cross-section will be used to evaluate reinforcement quantities. With this conventional approach, the part of the cross-section for which concrete temperatures have not exceeded 500°C will be assumed to have the same mechanical properties as at room temperature.

2.4.2 Heat transport in steel

Given the fact that thermal conductivity of steel is an order of magnitude higher than the thermal conductivity of concrete, the temperature distribution within a steel cross-section will be much more uniform than within a concrete cross-section. Therefore, it is mostly assumed from the start that temperature is uniformly distributed within a steel cross-section.

As a constant temperature is assumed over the entire cross-section, constrained deformations will be limited to longitudinal deformations and analytic formulae can be used to derive steel temperatures.

Indeed, the general Fourier differential equation is reduced to following equations for unprotected and protected steel sections:

$$\Delta\theta_a = k_{sh}\frac{1}{\rho_a \cdot c_a}\cdot\frac{A_m}{V}\cdot\dot{h}_{net}\cdot\Delta t \qquad (1)$$

and

$$\Delta\theta_a = \frac{\lambda_p}{d_p}\cdot\frac{1}{\rho_a \cdot c_a}\cdot\frac{A_p}{V}\left(\frac{1}{1+\phi/3}\right)(\theta_g - \theta_a)\Delta t - \left(e^{\phi/10}-1\right)\Delta\theta_g \qquad (2)$$

The above finite difference equations describe temperature variation within a steel cross-section during a time step Δt and can numerically be solved if initial conditions and boundary conditions are known. In both cases, the ambient temperature is assumed to be 20°C.

Equation (2) accounts for heat storage in the protection layer and assumes the temperature of the protection layer's external surface to be equal to the ambient gas temperature.

Let us now look in more detail into the meaning of the terms appearing in the above equation.

2.4.2.1 Net heat flux h_{net}

The heat flux $\dot{h}_{net}$ depends on the selected fire model and on the thermal properties of the protection layer.

a) net heat flux for unprotected steel sections

The heat flux for unprotected steel is determined by:

radiation: $\dot{h}_{net,r} = 5{,}67.10^{-8} \cdot \Phi \cdot \varepsilon_{res} \cdot \left((\theta_r + 273)^4 - (\theta_a + 273)^4\right)$
convection: $\dot{h}_{net,c} = \alpha_c \left(\theta_g - \theta_a\right)$

in which:

Φ view factor
ε_{res} residual emissivity coefficient
α_c convection coefficient
θ_a steel temperature
θ_r radiation temperature
θ_g ambient gas temperature

Some further clarifications:

- the view factor Φ is a geometric factor ≤ 1; for most practical cases, this factor can be assumed equal to 1.
- the residual emissivity coefficient ε_{res} is the product of the emissivity coefficient of steel ($\varepsilon_m = 0.7$) and of the enviroment ($\varepsilon_f = 1$).
- the radiation temperature θ_r can be taken equal to the ambient gas temperature θ_g during the fire hazard. This temperature is given by the fire curve used for the analysis.
- the value of the steel temperature θ_a is equal to the temperature from the previous calculation step.
- the convection coefficient α_c varies in practice between 25 (standard fire & external fire) and 50 W/m²K (hydrocarbon fire). For natural fire conditions, $\alpha_c = 35 W/m^2K$ may be assumed.

b) net heat flux for protected steel sections

During the calculation of temperatures for protected steel sections, the effect of the fire protection materials must be accounted for. The total net heat flux $\dot{h}_{net}$ for protected steel sections thus depends also on the thickness of the protection layer (d_p) and on the thermal properties of the protection material (λ_p, ρ_p, c_p).

Those properties are introduced into the finite difference equation (2) through the factor ϕ.

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

ϕ is called the participation factor of the protection layer.

In case the thermal capacity of the protection layer is low compared to the thermal capacity of steel ($\phi \approx 0$), then the next heat flux can be approximated as follows:

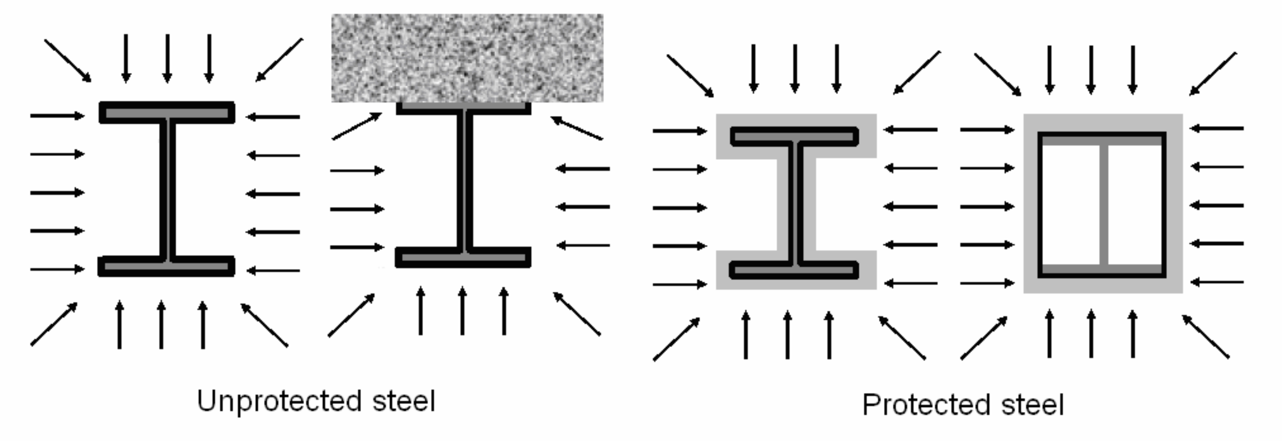
$$\dot{h}_{net} \approx \lambda_p / d_p \cdot (\theta_g - \theta_m)$$

Thus eq. (1) can approximately also be applied to protected steel sections.

2.4.2.2 Profile factor A_m/V

The profile factor A_m/V [m^{-1}] reflects geometric aspects of the steel cross-section and the way it is exposed to fire. The profile factor is defined as the ratio of the section’s circumference along which heat is injected into the section, to the section’s surface. Cross-sections with a high profile factor will respond faster to a thermal load and will warm up more quickly. A uniform temperature distribution is then mostly justified. Steel cross-sections with a low profile factor will be more massive and have a non-neglectable thermal inertia. Temperature variations within the section itself can no longer be ignored.

The concept of the profile factor is illustrated in the figures below.



Some remarks with respect to those figures:

- for steel elements below a concrete floor, the heat transfer from steel to concrete is neglected.
- for elements with a boxed protection layer, the steel surface is taken equal to the inner surface of the protection layer box

The profile factor for protected steel elements is referred to as A_p/V. Formulae for A_p/V are given in Table 4.3 of EN 1993-1-2.

It is mentioned that the profile factor for both unprotected and protected steel elements is usually included in product catalogues.

2.4.2.3 Specific heat c_a of steel

The specific heat of steel c_a [J/kgK] is determined as follows:

- for $20°C \leq \theta_a < 600°C$

$$c_a = 425 + 7{,}73 \cdot 10^{-1}\theta_a - 1{,}69 \cdot 10^{-3}\theta_a^2 + 2{,}22 \cdot 10^{-6}\theta_a^3$$

- for $600°C \leq \theta_a < 735°C$

$$c_a = 666 + \frac{13002}{738 - \theta_a}$$

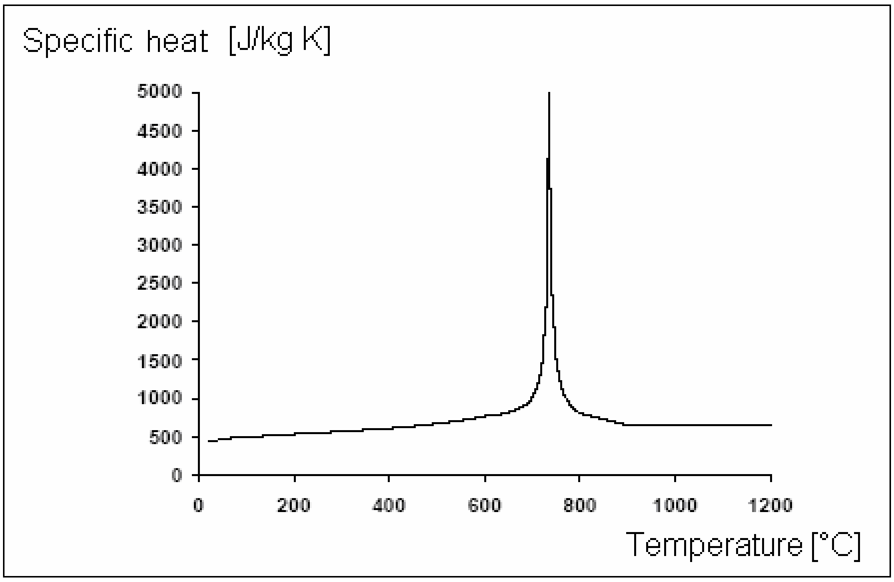
- for $735°C \leq \theta_a < 900°C$

$$c_a = 545 + \frac{17820}{\theta_a - 731}$$

- for $900°C \le \theta_a$

$$c_a = 650$$

The graph below illustrates the dependency of specific heat on temperature:



This graph clearly illustrates the strong dependency of specific heat on steel temperature. A peak value of 5000 J/kg K is reached at a steel temperature of 735℃.

2.4.2.4 Shadow coefficient k_{sh}

The shadow coefficient k_{sh} is a reduction factor that can be used with the calculation of temperatures for unprotected steel sections exposed to a standard fire.

The shadow effect is caused by the fact that parts of the section are locally hidden from the heat source, and it is clearly related to the shape of the steel section. It is only applicable to open type of sections (e.g. I-sections), while it is irrelevant for closed type of sections (e.g. hollow tubular sections).

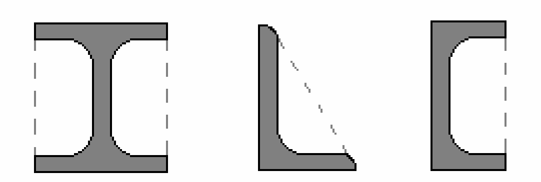
It can be shown that for I-sections the shadow effect can be accounted for accurately through the following relationship:

$$k_{sh} = 0.9[A_m / V]_{box} / [A_m / V]$$

In all other cases, the following relationship can be used to assess the value of k_{sh}:

$$k_{sh} = [A_m / V]_{box} / [A_m / V]$$

$[A_m / V]_{box}$ is the contour value of the profile factor, and it is defined as the ratio of the imaginary circumference encompassing the steel section, to the steel section area.



For protected steel elements, the shadow coefficient is relatively unimportant and may be taken equal to unity.

2.5 Mechanical response

The "mechanical response" of a building structure exposed to a fire hazard, is understood to cover the complete set of mechanical actions to which the building is subjected during the entire duration of the hazard.

Indeed, as a result of temperature increase

- a building structure will expand. However, this will only rarely result in a uniform expansion of the structure.
- strength and stiffness properties of construction materials will deteriorate, thus invoking additional deformations.
- strength and stiffness properties will deteriorate such that the structure may not be capable any more to resist the design actions.

Next to the use of expensive and destructive fire tests in laboratory conditions to determine the mechanical response of a structure or of structural elements, it becomes more and more usual to derive mechanical response characteristics using analysis techniques.

For steel structures in particular, 3 design methods are distinguished:

- each individual structural element of the entire structure is considered separately, respecting dedicated boundary conditions for the individual elements;
- part of the entire structure is investigated, simulating the interaction with the remainder of structure through appropriate boundary conditions for the part under investigation;
- the entire structure is considered as a whole.

It is obvious that the last mentioned design method will lead to a more realistic insight into the mechanical response of the structure when subjected to a fire hazard. Indeed, the loads transfer between the elements directly exposed to fire and the elements which are not exposed to fire, can be modelled accurately without any simplifying assumptions.

Furthermore, Eurocode presents three techniques to calculate mechanical response:

- through the use of **tables**; this option is applicable only to mixed steel-concrete type of structures;
- through **simplified analysis models**, including amongst other the "Critical temperature"-method;
- through **advanced analysis models,** which are numerical models based on the finite element method or the finite difference method.

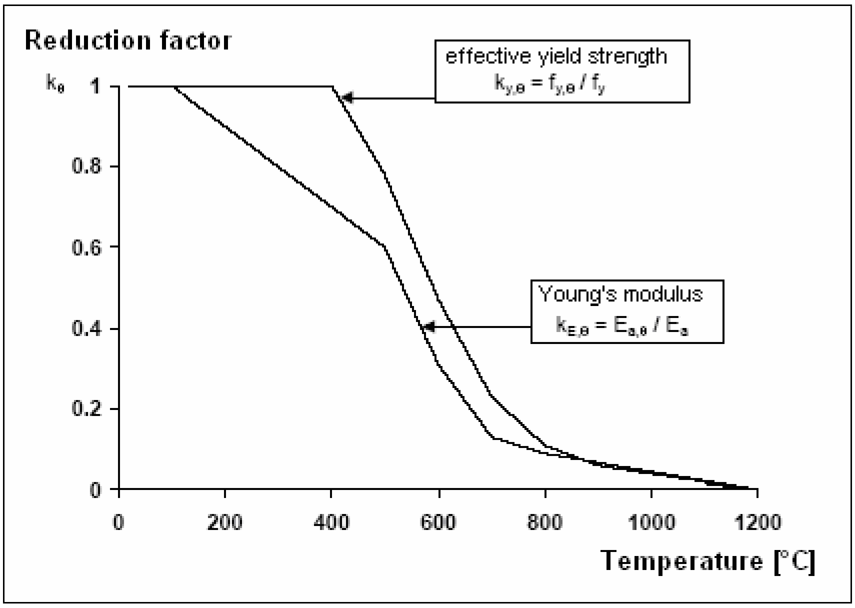
The last mentioned technique can be applied to all type of building structures and is used more and more as part of a modern Fire Safety Engineering approach.

Extensive information on the analysis method used by PowerFrame is found in the relevant chapters of the PowerFrame manual on Fire Resistance Analysis. Here, the further discussion is limited to the impact of temperature increase on the mechanical properties of steel & concrete.

2.5.1 Reduced yield strength and Young's modulus for steel elements

As mentioned already, mechanical properties will deteriorate as temperature increases. This will more in particular be the case for yield strength and Young's modulus of carbon steels, showing a strong decrease with increasing temperatures.

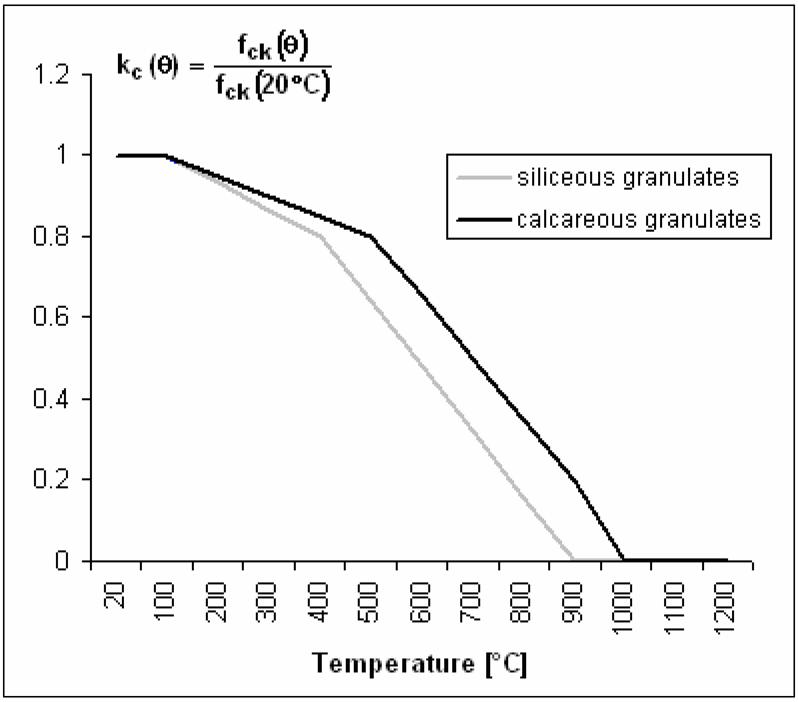
This relationship can be described analytically and is represented graphically in the figures given below. The factors $k_{y,\theta}$ and $k_{E,\theta}$ are reduction factors representing the ratio of yield strength / Young's modulus at temperature θ with respect to yield strength / Young's modulus at room temperature.



It can be seen that carbon steel strength & stiffness will only decrease significantly for temperatures exceeding 400°C. At 600°C, stiffness is reduced to 31% of the original value while strength has been reduced to approximately 47%.

2.5.2 Reduced compressive strength for concrete elements

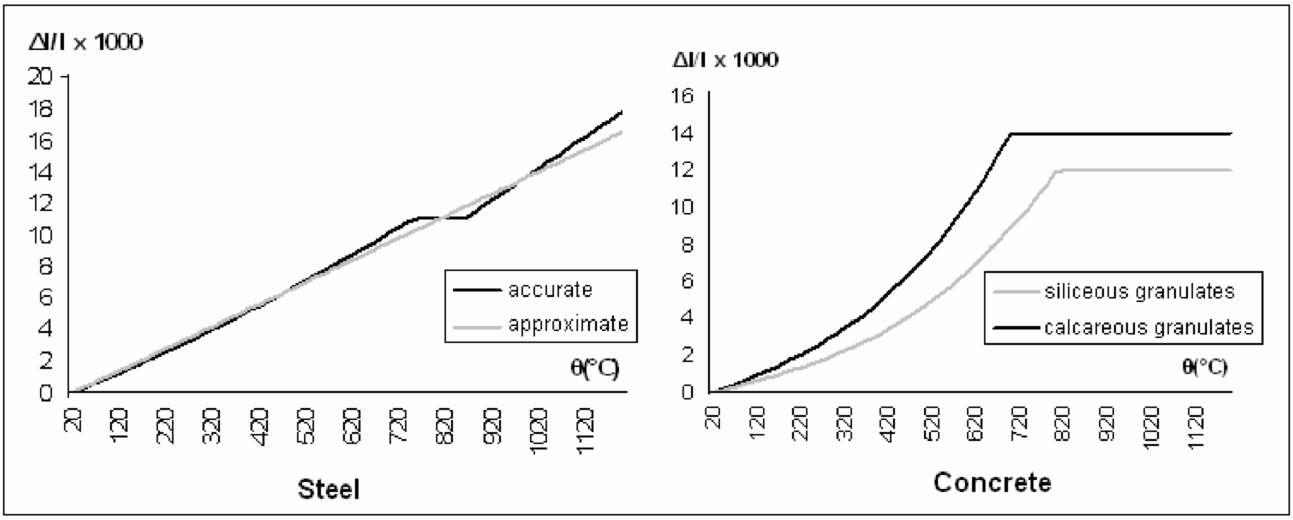
The variation of compressive strength f_{ck} as a function of temperature is documented in the graph below.



The slower decrease of compressive strength for siliceous concrete is due to the lower thermal conductivity of the granulates.
Note that the strength of such types of concrete is reduced to 50% at a temperature of 600℃. It will only happen rarely however that concrete temperatures will rise to this level.

2.5.3 Thermal expansion of steel and concrete

The more advanced analysis models require, next to the mechanical properties, the thermal expansion properties to be known. Thermal expansion coefficients of steel and concrete are presented graphically below. For numerical data, reference is made to EN1992-1-2 and EN1993-1-2.



For carbon steel, the thermal expansion coefficient can be considered constant over the entire temperature range. For concrete, the thermal expansion coefficient first increases with temperature, after which it remains constant for higher temperatures.

2.6 Fire Resistance Analysis scenario

The first step to be taken when a fire resistance analysis is performed with PowerFrame, consists in the selection or definition of a fire curve. The user either chooses one of the three predefined nominal fire curves (standard fire, external fire or hydrocarbon fire), or defines himself/herself a temperature-time curve (parametric fire). The requested fire resistance must also be specified, in other words the time window during which the building structure must maintain its function during a fire hazard.

Based on those parameters, the temperature variation within the compartment during the imposed time window is known. The **thermal loads** have thus been derived.

Following the ambient gas temperature increase, the temperature of the building structure will also raise albeit with a time delay. The next step will therefore imply the calculation of temperatures within the structural building elements during the time period in which they are exposed to a fire hazard, or in other words the calculation of the **building structure's thermal response**.

Depending on the type of material, a number of simplifying hypotheses can be introduced. For steel elements, only a minor error is introduced by considering the temperature to be uniform over the entire section. This allows the general differential equation to be reduced to a fairly straightforward set of finite difference equations. Starting from an initial temperature of 20℃, those equations then allow to assess steel temperature step by step.

For concrete building elements, a different approach will be required. Concrete cross-sections are typically more solid than steel sections and will have a considerably higher thermal inertia, implying temperature variations over the entire section. In addition to a global temperature increase of the section, the aforementioned temperature variation will also result in a (linear) temperature gradient with respect to both principal axes of inertia.

The temperature increase and gradients will next be applied as thermal loads on the structural elements exposed to the heat source. This accidental load is then combined with other mechanical loads to define design load combinations.

It should be noted however that PowerFrame always performs a fire safety analysis at element level. In this particular situation, it is specified by the relevant standards that the thermal loads should not be considered during elastic analysis. Nevertheless, PowerFrame does allow you to consider temperature increase & gradients. It is however recommended to consider the effects of temperature increase & gradients only partially, otherwise the internal forces will rise to extremely high levels. The standards do not provide any particular indications with respect to the amount in which temperature increase & gradients should be accounted for. This choice thus requires some experience and responsibility from the user.

Let's now have a closer look into the **mechanical response of the building structure** to the above temperature changes:

- A number of material properties, among which Young's modulus and the thermal expansion coefficient, will decrease with increasing temperature. The global elastic analysis, which is performed for the accidental fire load combinations, will therefore need to take those reduced material characteristics into account.

- As a result of the external deformation constraints, internal forces and stresses will come into existence within the building structure. In case of a steel structure, constrained deformations will be limited to longitudinal deformations, for concrete structures the temperature increase & gradients will give rise to longitudinal & bending deformations.

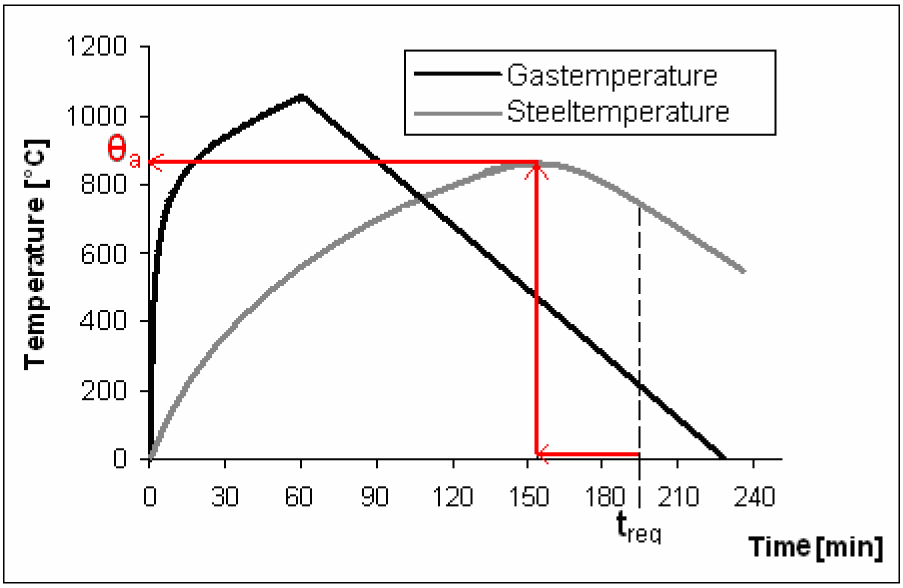
- Once the global elastic analysis has been completed, the internal forces will serve as a basis for the verification of steel sections or for the calculation of reinforcement quantities.

- Resistance and buckling verification of steel members will be conforming to EN 1993-1-2 (par 4.2), and will be entirely analogous to a verification at room temperature conditions. One should of course not overlook that yield strength is strongly dependent on temperature, so that high temperature levels will have a major impact on the verification results.

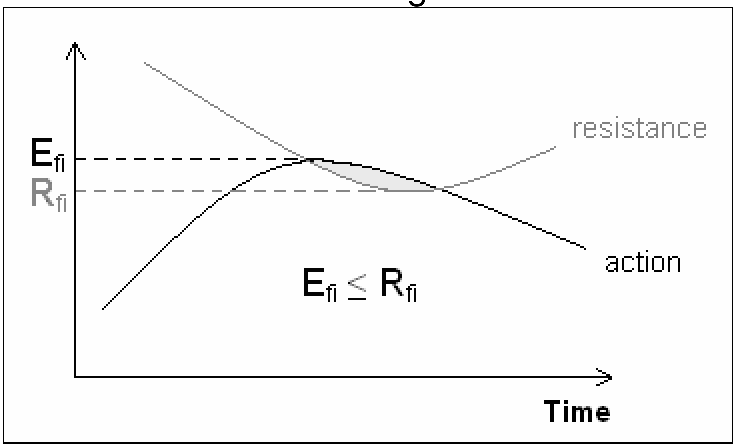
- For concrete sections, reinforcement quantities are calculated on the basis of a reduced concrete section and deteriorated steel characteristics. All other properties will be taken identical to the ones at room temperature (20℃). For both concrete and reinforcement steel, the partial safety factor is taken equal to 1.

- Remark: the design temperature for a particular material is not necessarily equal to the temperature at the time instant corresponding to the required fire resistance. Indeed, in case of a parametric fire the environment temperature may have significantly decreased at that time instant.

- For steel elements, one only needs to look at the maximum steel temperature within the entire time interval. With decreasing temperature, the mechanical properties of constructive steel grades will again evolve to their original values while the thermal load will decrease again.

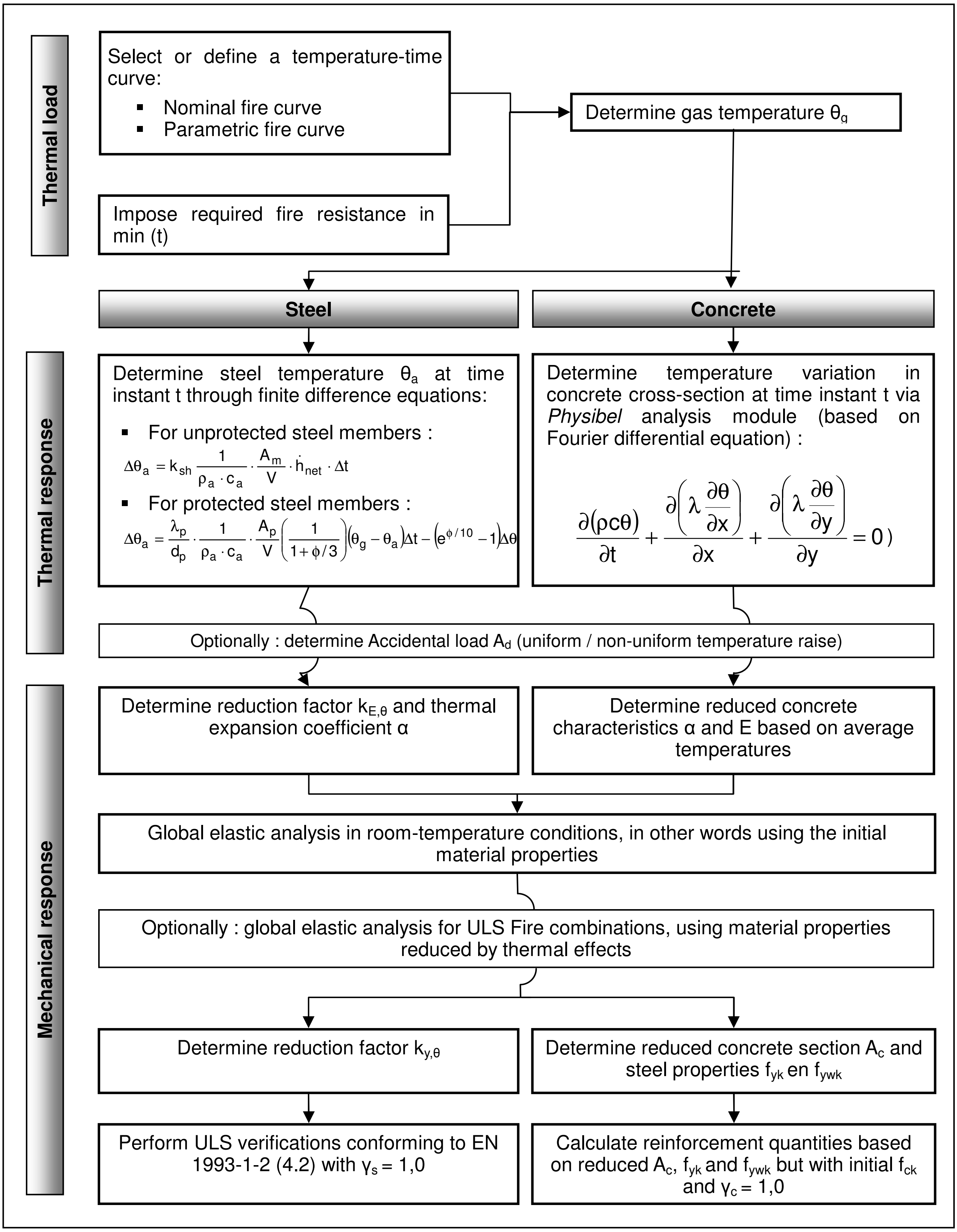


For concrete elements, this is somehow different. The material properties do not necessarily reach a minimum at the time instant at which the thermal action has reached its highest level.



While the global elastic analysis is performed for the time instant at which the thermal action is at its highest level, reinforcement quantities are calculated taking into account the temperature at which concrete resistance is at its lowest.

For a nominal fire curve, this doesn't pose any particular problems as temperature never decreases over time. The lowest resistance will necessarily be found when the highest temperatures are reached, in other words at the time instant corresponding to the required fire resistance.


Thermal load
Select or define a temperature-time curve:
Nominal fire curve
Parametric fire curve
Impose required fire resistance in min (t)
Determine gas temperature θ_g
Steel
Concrete
Thermal response
Determine steel temperature θ_a at time instant t through finite difference equations:
For unprotected steel members :
$\Delta\theta_a = k_{sh} \frac{1}{\rho_a \cdot c_a} \cdot \frac{A_m}{V} \cdot \dot{h}_{net} \cdot \Delta t$
For protected steel members :
$\Delta\theta_a = \frac{\lambda_p}{d_p} \cdot \frac{1}{\rho_a \cdot c_a} \cdot \frac{A_p}{V} \left(\frac{1}{1+\phi/3}\right)(\theta_g - \theta_a)\Delta t - \left(e^{\phi/10} - 1\right)\Delta\theta$
Determine temperature variation in concrete cross-section at time instant t via *Physibel* analysis module (based on Fourier differential equation) :
$\frac{\partial(\rho c \theta)}{\partial t} + \frac{\partial\left(\lambda \frac{\partial\theta}{\partial x}\right)}{\partial x} + \frac{\partial\left(\lambda \frac{\partial\theta}{\partial y}\right)}{\partial y} = 0$)
Optionally : determine Accidental load A_d (uniform / non-uniform temperature raise)
Mechanical response
Determine reduction factor $k_{E,\theta}$ and thermal expansion coefficient α
Determine reduced concrete characteristics α and E based on average temperatures
Global elastic analysis in room-temperature conditions, in other words using the initial material properties
Optionally : global elastic analysis for ULS Fire combinations, using material properties reduced by thermal effects
Determine reduction factor $k_{y,\theta}$
Determine reduced concrete section A_c and steel properties f_{yk} en f_{ywk}
Perform ULS verifications conforming to EN 1993-1-2 (4.2) with $\gamma_s = 1,0$
Calculate reinforcement quantities based on reduced A_c, f_{yk} and f_{ywk} but with initial f_{ck} and $\gamma_c = 1,0$