PART 2 : Thermal Response

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1. INTRODUCTION

When exposed to fire conditions, a building construction is subjected to both mechanical and thermal actions. The mechanical actions follow from the dead weight and the superimposed loads, working on the structure at the moment of fire outbreak. The thermal actions follow from the increase of the gas temperature in the fire compartment and are governed by the heat transfer conditions at the surface of the construction elements. As a result of the thermal actions, the temperatures in the construction will increase. This is called “thermal response” and potentially leads to thermal elongation and deterioration of the mechanical properties in the heated parts of the construction. Depending on the situation, the thermal elongation may (partly) be restrained leading to thermal induced stresses. In combination with the mechanical actions, significant deformations may occur and - under circumstances - the building construction, or parts of it, may even collapse. This process is called “mechanical” response.

The above meant chain of events is schematically presented in the Fig. 1.

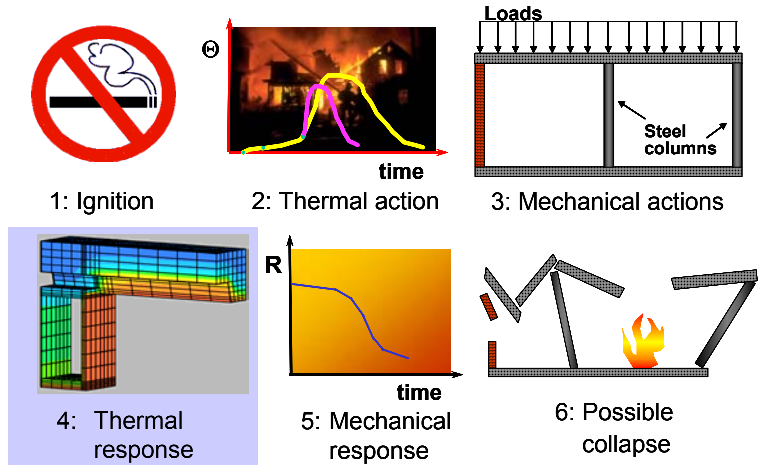


Fig. 1: Resistance to fire – Chain of events

A fire engineering approach into the relevant actions during a fire is presented in Part 1 of this course. For the mechanical response, reference is made to Part 3. The underlying Part 2 deals with the thermal response. The discussion is limited to steel and to composite steel concrete elements and follows the fire parts of the relevant Eurocodes [1], [2].

2. BASICS & ILLUSTRATIONS

The heat transfer in a building element is governed by the following differential equation (so called Fourier differential equation) in combination with the relevant boundary and initial conditions:

$$\frac{\partial(\rho\, c\theta)}{\partial t} + \frac{\partial(\lambda \frac{\partial\theta}{\partial x})}{\partial x} + \frac{\partial(\lambda \frac{\partial\theta}{\partial y})}{\partial y} + \frac{\partial(\lambda \frac{\partial\theta}{\partial z})}{\partial z} = 0 \qquad \ldots (1)$$

where:

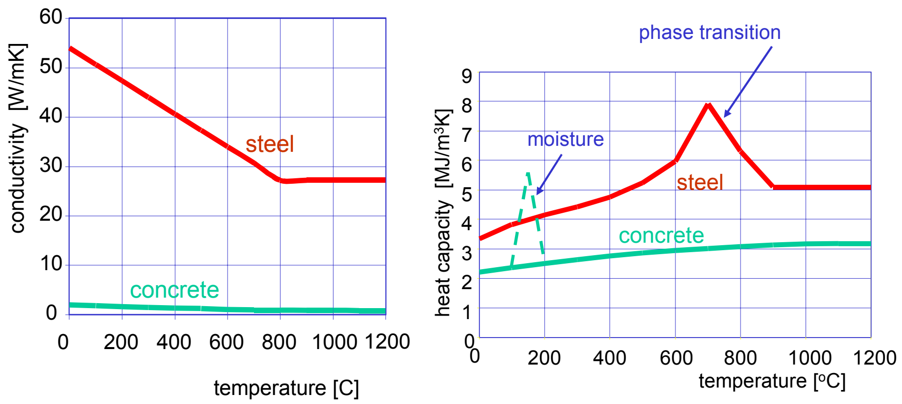
x, y, z	is	co-ordinates in m
Θ	is	temperature at x, y, z in ˚C
ρ	is	density in kg/m^3
c	is	specific heat in J/kg
λ	is	thermal conductivity in W/m ˚K

For a brief explanation of this basic equation, refer to Annex A.

From equ. 1, one can conclude that the following thermal material properties have an influence on the temperature development in building elements exposed to fire:

- thermal conductivity
- specific heat.

It is common to combine the specific heat with the density. Notation: "heat capacity", dimension: J/m^3. Both the thermal conductivity and the specific heat of most of the building materials are strongly dependent on the temperature. This is illustrated in Figs. 2^{a,b} for concrete and steel [1], [2].



(a) thermal conductivity

(b) thermal capacity

Fig. 2: Thermal properties of steel and concrete

The peak at appr. 730 ºC in the graph for the thermal capacity of steel is due to phase transition in steel; the peak in the graph for the heat capacity of concrete is to account for the effect of moisture vaporization.

Note that the thermal conductivity of steel is an order of magnitude larger than that of concrete. This why the temperature distribution in fire exposed steel elements is much more uniform than that in the concrete parts of composite elements. By way of simplification, it often even assumed that the temperature distribution in steel elements is uniform. See chapter 3.

If the thermal properties of the materials of which a building element is composed are known, the temperature development in such an element can - for given thermal actions – be calculated on the basis of equ. (1). However, only in exceptional (simple) cases, analytical solutions are available [3]. In practical situations, numerical methods (computer models) have to be used. At present, a variety of such models do exist. See Part 4.

In Fig. 3, 4 and 5 some illustrations of the abilities of thermal response models are presented.

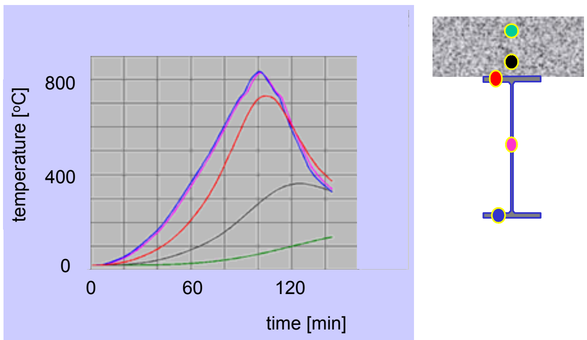
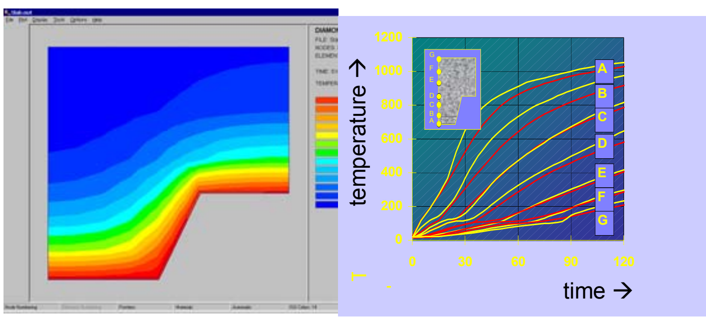


Fig. 3: Thermal response: steel beam/concrete slab (2D)

In Fig. 3 the thermal response of a bare steel beam with a concrete slab on top, exposed to natural fire conditions from below, is presented [4]. One can see that the temperature in the lower flange and in the web of the steel beam is practically identical. However, the temperature development in the upper flange lags behind. This is due to the heat losses to the relatively cold concrete slab on top of the upper flange. In the simple calculation models for steel according to EN 1993-1.2, a uniform temperature distribution is assumed, which is based on the temperature in the lower parts of the steel section. To account for the lower temperatures in the upper flange, a correction factor κ on the mechanical load is introduced. See Part 3.



computer simulation test vs. theory

Fig. 4: Thermal response: composite slab (2D)

In Fig. 4 the 2 D temperature distribution in a concrete slab with profiled steel sheet is presented after 120 minutes standard fire exposure, calculated by means of DIANA [5]. Also, a comparison is made between the calculated temperature fields and test results. There appears to be good agreement between test and theory, especially in the critical areas, at the upper side of the ribs (i.e. location D on the right hand side of Fig. 4). Note that the temperature distribution is significantly non-uniform. This is a result of the relatively small value of the thermal conductivity of concrete.

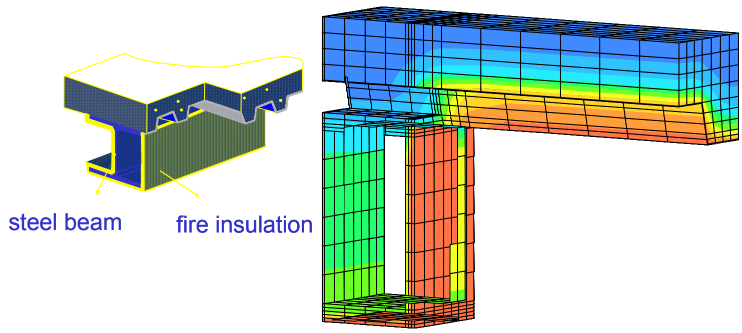


Fig. 5: Thermal response composite edge beam (3D)

In Fig. 5 the 3D thermal response of a composite edge beam is presented [6]. The steel section is at one side box protected and provided with a contour encasement at the other side. 3D calculations, the results of which are shown here, are rather cumbersome and not often used in practical design situations. The aim of showing the results here is merely to point out the potential of presently available calculation tools.

3 CALCULATION RULES FOR STEEL ELEMENTS

3.1 Scope

Main aim of a thermal response analysis of structural steelwork is to find the resistance to fire. Since steel elements normally do not have a separating function, only the fire resistance criterion regarding the load bearing capacity is relevant.

In EN 1993.1.2 the following options for calculating the behaviour of fire-exposed steelwork are given[1]:
- simple calculation models;
- advanced calculation models.

The thermal response in the advanced models is based on equ. (1), in combination with the relevant thermal actions. Such models are of a general validity. Basic assumption for the simple models is that the temperature distribution is uniform. This is approximately true because of the relatively high thermal conductivity of steel. See also the discussion in the previous chapter.

Under the assumption of a uniform steel temperature, the calculation of the fire resistance can schematically be reviewed as in Fig. 6.

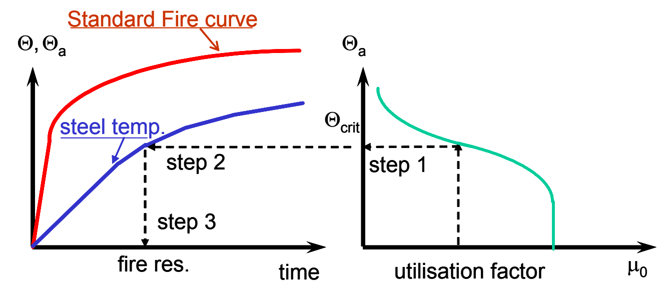


Fig. 6: Calculation procedure for fire resistance of steel elements, on the basis of the simple calculation method

The following three steps have to be made:
- step 1: determine the critical steel temperature (i.e. the temperature at which failure occurs); this temperature depends on the ratio between the actual load and the load bearing capacity at room temperature of the element under consideration and is the outcome of the mechanical response analysis dealt with in Part 3.
- step 2: determine the temperature development in the steel section; this is the outcome of the thermal response analysis of the steel element, dealt with in this part of the syllabus.
- step 3: determine the fire resistance of the steel element; this is simply the combination of step 1 and 2.

[1] Note that for steel, no tabulated data are given.

Hereafter, the various aspects of step 2 will be discussed. Assume an infinitive high value for the conductivity of steel. Hence, the steel temperature is uniformly distributed and equ. (1) reduces to (see Annex B):

$$\frac{d\theta}{dt} = \frac{A_m/V}{\rho_a c_a} \cdot \dot{h}_{net,tot} \qquad \ldots (2)$$

with:

θ_a	is	steel temperature in ºC (assumed to be uniform)
t	is	time in sec
ρ_a	is	density of steel in kg/m^3
c_a	is	specific heat of steel in J/kg
$\dot{h}_{net,tot}$	is	total net heat flux to the steel element in W/m^2
A_m	is	the fire exposed surface area of the steel member in m^2/m'
V	is	volume of the steel member in m^3/m'

In the right hand side of equ. (2) the following terms are distinguished:

- the term "$\dot{h}_{net,tot}$" represents the thermal action, depending on the relevant fire model (e.g. standard fire conditions, hydro-carbon fires, natural fire) and the protection (if any) of the steel member (see also Part 1)
- the term "ρ_a c_a" represents the effect of the thermal properties of steel
- the term "A_m/V" represents the effect of the geometry of the steel section and the way it is exposed to fire conditions (exposure on all sides, 3 sides etc.); this term is commonly referred to as "Section Factor".

Equ. (2) is the basis of the calculation rules for the steel temperature development, specified for the simple calculation model in the fire part of the Eurocode on steel structures [1] and can only be solved if the initial and boundary condition are known. A common assumption regarding the initial conditions is that prior to the occurrence of fire, room temperature conditions hold, i.e. 20 ºC. The boundary conditions are determined by the total net heat flow (= thermal action) from the fire environment to the steel element. This heat flux is due to radiation and convection. For some basic equations, refer to Fig. 7. See also Part 1.

- Radiative heat transfer: $\dot{h}_{net,r} = \Phi \cdot \varepsilon_m \cdot \sigma \cdot \left[(\Theta_r + 273)^4 - (\Theta_m + 273)^4 \right]$
- Convective heat transfer: $\dot{h}_{net,c} = \alpha_m \cdot (\Theta_g - \Theta_m)$

with:

Θ_{rad}	is radiation temperature [°C]	$\Rightarrow \Theta_{rad} \approx \Theta_g \rightarrow$ fire curve
Θ_m	is surface temperature [°C]	$\Rightarrow$ thermal response
ε_m	is surface emissivity [-]	$\Rightarrow$ safe: 0.8
α_c	is coefficient convection	$\Rightarrow$ 25 - 50 W/m^{2}K (depending on fire model)
Φ	is configuration factor [-]	$\leq 1.0 \Rightarrow$ safe: 1.0
ρ	is Stephan Boltzmann constant	= $5.67 \cdot 10^{-8}$ W/m^{2}K^4

Fig. 7: Heat transfer at the exposed side

The following observations hold:

The radiation law of Stephan Bolzmann gives the radiative heat transfer. According to this law, the so-called radiation temperature of the fire environment determines the maximum radiation to the steel element [3]. It can be shown that - by way of conservative approximation - the radiation temperature can be taken equal to the gas temperature and follows from the fire model taken into account. See Part 1. This is the basis for the equation for net radiative heat transfer specified in EN 1993.1.2 [1]. In this equation, the following physical factors play a role:

- Stephan Bolzmann constant ($\sigma = 5.67\ 10^{-8}\ W/m^2K^4$): this is a physical constant
- the surface emissivity of the member (ε_m): this depends on the material applied in the surface
- the configuration factor (Φ): a geometrical factor ≤ 1; for many practical cases (e.g. simulation of standard fire tests) this factor may be taken equal to unity[2].

Note that the value of the surface temperature (Θ_m) for a certain time step follows from the temperature in the preceding time step by solving equ. (1).

The net convective heat transfer may be approximated proportional to the temperature difference ($\Theta_g - \Theta_m$) and is characterized by the coefficient of convection (α_c); it varies in practice from 25 (standard fire conditions) to 50 W/m^2K (hydrocarbon conditions)[3]. See also [7].
Some practical implications of the above calculation rules will be discussed for bare and protected steel sections respectively

3.2 Bare steelwork

Calculation rules for the temperature development in bare (i.e. unprotected) steelwork specified in the ENV version of EC3-1.2 are based on conventional values for the coefficients of both radiative and convective heat transfer [8]. These values are chosen such that a reasonable agreement with test results is obtained, leading however to assumptions which are – from a physical point of view – not very convincing. This particularly holds for the radiative heat transfer: a value for the resultant emissivity as low as 0.5 ($= \varepsilon_f.\varepsilon_m$)[4] is necessary in order to achieve a reasonable match with test results. This problem has become even more explicit when introducing the so-called Plate Thermometer (in stead of the common thermocouples) as measuring device for controlling the gas temperature during standard fire resistance testing [1], [9].

With a view to arrive at more realistic and consistent calculation rules for the temperature development in bare steel members and also to stay in line with future standard fire resistance testing practice, in the EN version of EC3-1.2 [1] more realistic values for the emissivity coefficients have been specified: for the surface emissivity of steel (ϵ_a): 0.7 (being a low, but realistic value) and for the fire environment (ϵ_{fi}): 1.0 (as direct consequence of using the plate thermometer for furnace control) [9]).

The "uplifting effect" in terms of calculated temperatures of these modifications is – by and large - compensated by taking into consideration the so-called "shadow effect", which is not explicitly taken into account in the ENV rules. Assuming fully embedded members (as in the case of simple calculation models), the shadow effect is caused by local shielding of the radiation, due the shape of the steel profile.

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2 In case a steel element is subjected to a localized fire, $\Phi < 1$ applies.
3 For natural fire conditions, $\alpha_c = 35\ W/m^2K$ applies.
4 The emissivity of the fire environment is noted as: ϵ_f

It plays a role for profiles with a concave shape, such as I-sections; for profiles with a convex shape, such as tubes, it does not exist (no local shielding).

The increase of temperature $\Delta\theta_{a,t}$ in an unprotected steel member during a time interval Δt may then be determined from:

$$\Delta\theta_{a,t} = k_{sh} \frac{A_m / V}{c_a \rho_a} \dot{h}_{net,d} \Delta t \qquad \text{… (3)}$$

with:

k_{sh} is correction factor for the shadow effect

$\dot{h}_{net,d}$ is the design value of the net heat flux per unit area calculated for bare steel, i.e. with $\epsilon_a = 0.7$ and $\epsilon_{fi} = 1.0$ [W/m²].

New in the expression – compared to the ENV version of EC3-1.2 – is the correction factor k_{sh} for the shadow effect[5]. It can be shown that for I sections under nominal fire actions the shadow effect is reasonably well described by taking: [9]

$$k_{sh} = 0.9\, [A_m/V]_{box}/[A_m/V] \qquad \text{... (4}^{a}\text{)}$$

with

$[A_m/V]_{box}$ is box value of the section factor[6]

In all other cases the value of k_{sh} shall be taken as:

$$k_{sh} = [A_m/V]_{box}/[A_m/V] \qquad \text{… (4}^{b}\text{)}$$

It follows from the above definitions of k_{sh} that for tube profiles, the shadow effect is not activated, since $[A_m/V] = [A_m/V]_{box}$

Refer to Fig. 8 for a summary of the basic equations for the temperature rise in bare steel elements

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5 The correction factor makes no distinction between radiative and convective heat flux. It is clear that convective heat transfer is less affected by the shadow effect than radiative heat transfer; this effect is ignored because convection plays only a minor role under fire conditions.

6 The box value of the section factor of a steel section is defined as the ratio between the exposed surface area of a notional bounding box to the section and the volume of steel.

$$\frac{d\Theta_a}{dt} = k_{sh}\frac{A_m/V}{\rho_a c_a}\cdot \dot{h}_{net,tot} \qquad \dots (1)$$

$$\Delta\Theta_a = k_{sh}\frac{K_{bare}}{\rho_a c_a}\cdot\frac{A_m}{V}\cdot(\Theta_g - \Theta_a)\cdot\Delta t \qquad \dots (2)$$

Legend:
$\Delta\Theta_a$: increase steel temp.
Δt : time step
A_m/V section factor
K_{bare}: heat transfer coef.
k_{sh} : corr. coef. shadow effect

with

$$K_{bare} = \alpha_c + \frac{\varepsilon_m \sigma\left[(\Theta_g+273)^4 - (\Theta_a+273)^4\right]}{\Theta_g - \Theta_a} \qquad \dots (3)$$

Fig. 8: Temperature rise in bare steelwork

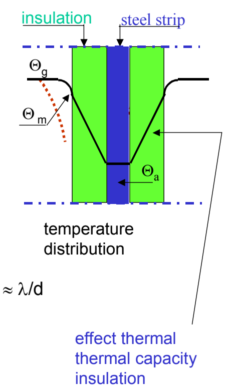
3.3 *Insulated steelwork*

The equation for calculating the temperature development in insulated steelwork is similar to equ. (3). However in this case the effect of the insulation has to be taken into account when calculating the net heat flux. In practical situations, the temperature drop over the insulation is relatively large. Consequently, the surface temperature of the insulation is close to the gas temperature. Hence, the effect of the radiative heat transfer is small and normally can be ignored. This means that the shadow effect is not important; hence there is no need to introduce a correction factor k_{sh} as for bare steel sections. See also [1]. The above is visualised in Fig. 9. Also the basic equations for insulated steel sections are presented in this Fig. As for bare steel, an overall heat transfer coefficient can be defined (notation: K_{ins}). Apparently, K_{ins} is a function of the thickness of the insulation (d_p) and of the thermal properties of both steel (ρ_a, c_a) and the insulation material (λ_p, ρ_p, c_p). See also [1], where equations are presented which take into account the above-mentioned effects. If the thermal capacity of the insulation is small compared to the thermal capacity of the steel, K_{ins} may be approximated by $K_{ins} \approx \lambda_p/d_p$, since under such circumstances a linear temperature distribution over the insulation may be assumed. This is also indicated in Fig. 9. The Section Factor for insulated steel elements is denoted as A_p/V. See under 3.4.

$$\Delta\theta_a = \frac{K_{ins}}{\rho_a c_a}\cdot\frac{A_m}{V}\cdot(\Theta_g - \Theta_a)\cdot\Delta t \qquad \dots (a)$$

with

$$K_{ins} = K_{ins}\left(\frac{\lambda}{d}, \rho_p, c_p, \rho_a, c_a\right) \qquad \dots (b)$$



Notes: (!) $\Theta_g - \Theta_m$ <<<< $\Theta_m - \Theta_a$
(2) for light weight insulation: $K_{ins} \approx \lambda/d$

Fig. 9: Temperature rise insulated steel work: basic

3.4 *Design parameters for the temperature development*

3.4.1 General

The temperature development in a steel element depends – for given fire conditions – on two design parameters:

- the section factor A_m/V, A_p/V (for bare and insulated (protected) members respectively)
- the insulation characteristics d_p, λ_p, ρ_p, c_p (for insulated members only).

In the following paragraphs each of these parameters will briefly be discussed. Emhasis is on standard fire conditions, because in practice such conditions are most widely used. The discussion will be complemented with comments on the potential use of applying the Natural Fire Safety Concept on bare and insulated steel members. See paragraph 3.4.4.

3.4.2 Effect and determination of the of the Section Factor

In Fig. 10 the effect of the Section Factor on the temperature development in bare steel sections under standard fire conditions is dispatched. A practical range of Section Factors is chosen, i.e. between 50 and 400 m^{-1}. For a Section Factor of 100 m^{-1}, also the possible effect of fire insulation is presented.

Similar information, however in a more generalized way, is presented in Fig. $11^{a,b}$. Fig. 11^a refers to bare steel members; each curve holds for a certain time of standard fire duration. Fig. 11^b refers to steel members provided with a (practical) insulation system and exposed to standard fire conditions during 90 minutes; each curve holds for a certain insulation thickness.
One may conclude from these Figures that the Section Factor has a significant influence on the development of the steel temperature, especially if the Section Factor is low and for small values of the insulation thickness.

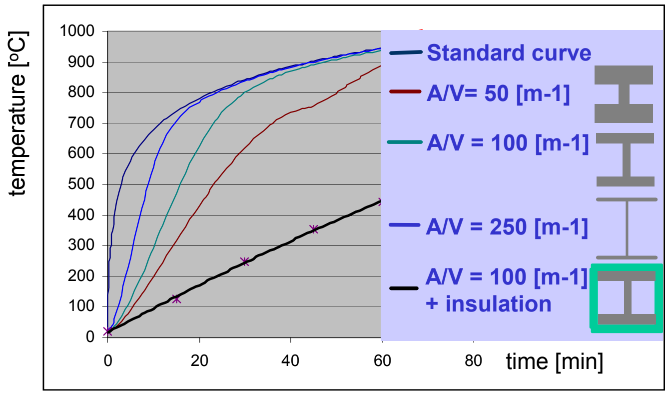


Fig. 10: Temperature development in steel profiles as function of time

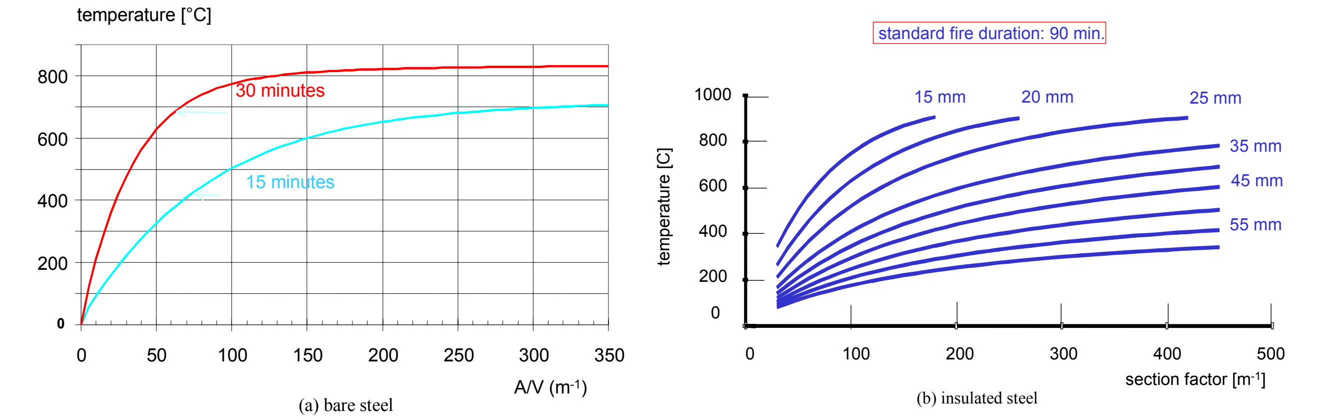
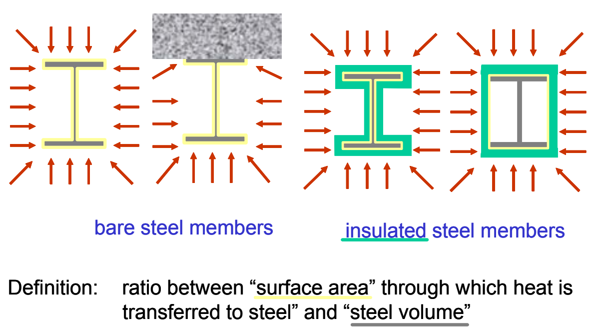


Fig. 11: Steel temperatures as function of the section factor

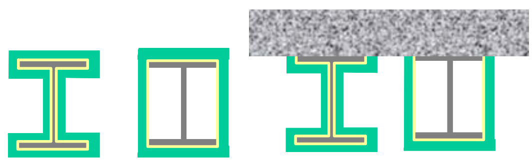
As mentioned earlier, the Section Factor is defined as the ratio between the "surface area through which heat is transferred to steel" and the "steel volume". In addition, the following (conventional) rules apply:

- for box protection, the steel perimeter is taken equal to the bounding box to the steel profile
- for steel sections under a concrete slab, the heat exchange between steel and concrete is ignored.

The concept is illustrated in Fig 12. In this Figure, also some quantitative examples are given. For a more comprehensive overview, refer to [1].



(a) concept



IPE100	387	300	334	247
HE280A	165	113	136	84
HE320B	110	77	91	58

Note: range: ≈ 50 - 400 [m^{-1}]

(b) quantification

Fig. 12: Section Factor steel profile

3.4.3 The characteristics of fire insulation on structural steelwork

In paragraph 3.4.1 the following characteristics of the fire insulation have been mentioned:
- thermal conductivity (λ_p)
- specific heat (c_p)
- density (ρ_p)
- thickness (d_p)

The first three characteristics are physical properties. One has to realize however that their actual values depend on changes, which take place in the insulation during fire exposure, such as cracks, delaminating, migration of moisture etc. This holds especially for the thermal conductivity. Moreover, the thermal conductivity of the materials commonly used as fire insulation, significantly increases as function of the temperature. This is why λ_p-values as given in handbooks for room temperature applications should not be used in the fire design.

For the determination of λ_p, a special semi-empirical approach has been developed [10]. In this approach, two different types of tests are foreseen:
(a) tests on loaded and unloaded beams
(b) tests on unloaded, short columns.

ad. a: Aim of these tests is to verify whether the insulation system "remains coherent and cohesive to their supports throughout the relevant fire exposure"[7] as required in 3.4 of [1]. To this end, two pairs of beams with the same cross section are exposed to standard fire conditions in a furnace. The beams of one pair are provided with the maximum thickness of the insulation system under consideration; the beams of the other pair with the minimum thickness. Differences in thermal response between the loaded and the unloaded beam of one pair are assumed to be due to stress induced deformation of the loaded beam. Where appropriate, correction factors are introduced to take such effects into account.

ad. b: Aim of these tests (commonly referred to a "exploratory tests) is to find λ_p-values which are representative under fire conditions. To this end, a series of 10 unloaded, short columns (typical height: 1 m) is exposed to standard fire conditions. The thickness of the insulation as well as the Section Factor are systematically varied. The measured steel temperatures are, where necessary, corrected on the basis of the beam tests. The results are subject to an assessment, which leads to design graphs, such as presented in Fig. 11[b], although also other formats are in use. Computer programmes are available, by which such an analysis can be performed as well as programmes which can use the obtained information for fire design purposes.

In Fig. 13[a,b] photos are dispatched of a loaded beam before and after the fire test. Note the significant deformations, which may lead to "stickability" problems.

[7] This is commonly referred to as "stickability".

(a) before fire testing (b) after fire testing

Fig. 13: Deformation of loaded beams as result of fire exposure

The test & assessment method referred to above is used for insulation systems, enclosing the steel element. These are on the market in different forms:

- sprays
- boards
- intuemescents.

Other types of fire protection means are screens. They can be applied vertically (to fire protect steel studs in partitions) or horizontally (as ceiling membranes, to protect steel beams in floor or roof constructions). European standards are available, based on similar principles as described above, to assess the contribution of such protection systems to the fire resistance of structural steelwork [11], [12]. Discussion of these standards is outside the scope of this syllabus.

It will be clear from the above discussion that the use of insulation characteristics, obtained under conditions which are representative for what could happen during a fire, are recommended. Under circumstances it may be useful to have the possibilities for a "fast & easy" approximation of the temperature rise in fire exposed steelwork. With this in mind, the European Convention for Constructional Steelwork (ECCS) has developed so-called "Euro-monograms" [13]. An illustration of these monograms is given in Fig. 14. For a given time of standard fire exposure, the temperature of a bare steel member can be found as function of the Section Factor A_m/V. For an insulated member, the following factor is used as input parameter (see also Fig. 9):
$(\lambda_p / d_p) . (A_p/V)$.

Note that the Euronomograms are determined on the basis of the ENV version EC3-1.2. Also for this reason they should be used with some reluctancy.

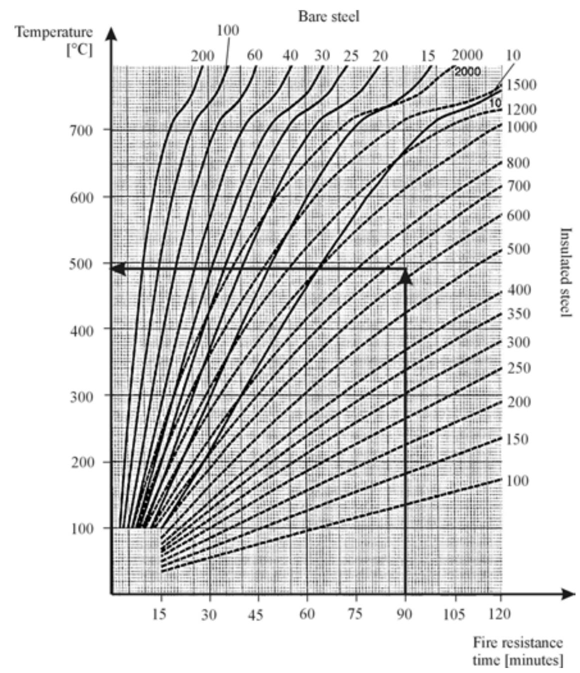


Fig. 14: Euronomograms

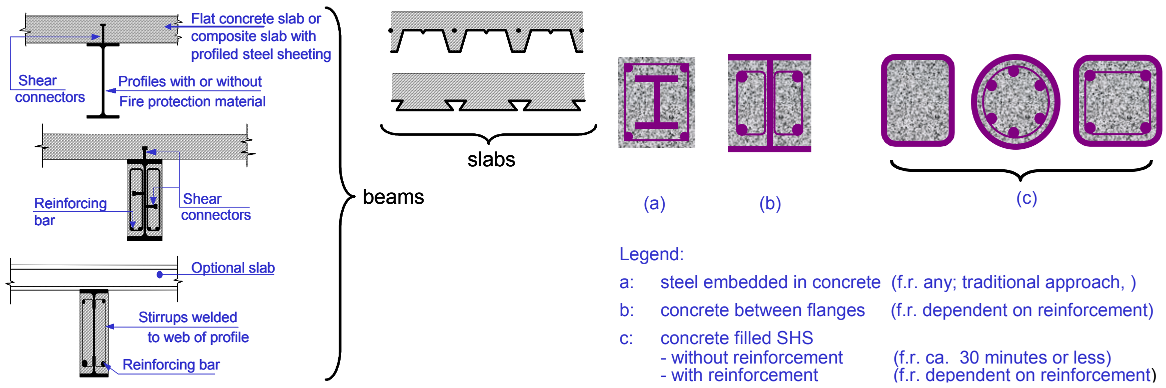
3.4.4 The use of non standard fire conditions

In Part 1, emphasis was on an approach, based on the Natural Fire Safety Concept. Such an approach can directly be applied to bare steel, since the thermal (and mechanical) properties of steel are valid also under non-standard fire conditions. This is not straightforward for the relevant thermal characteristics of the insulation systems used to protect steel. As has been explained in the previous section, such characteristics are determined under standard fire conditions. Strictly speaking, they refer to these conditions and no systematic information is available how their values are affected if the fire conditions are significantly different from the standard fire conditions. The use of the Natural Fire Safety Concept for the design of protected steel structures should therefore be handled with some caution. On the other hand one has to realise that at present the thermal characteristics are accepted without any concern that in reality the fire conditions may be far from standard. For this reason one could argue that the characteristics may be used in a natural fire safety approach as well.

4. CALCULATION RULES FOR COMPOSITE ELEMENTS

4.1 Scope

EN 1994.1.2 covers a variety of composite elements. For a review, refer to Fig. 15.



(a) beams and slabs (b) columns

Fig. 15: Options for composite element according to EN 1994.1.2

Depending on their nature, composite steel-concrete elements may have a load bearing and a separating function. Hence, all 3 fire resistance criteria may be relevant:
- load bearing capacity
- insulation
- integrity[8].

Concrete is an integral part of the cross section of any composite member. The assumption of a uniform temperature over the cross section (as in the simple models for steel elements) is not realistic for composite members. This complicates the calculation procedure for composite elements significantly. That is why in EN 1994.1.2 not only options for simple and advanced calculation models are given (as for steel), but also for tabulated data [2].

The role of thermal response analysis in the tabulated data on composite elements will not be discussed in this syllabus. This is because in the tabulated data on load bearing capacity, no reference is made to assumptions regarding thermal response, whilst the data that are given are mainly based on experience from standard fire tests. As far the tabulated data on insulation are concerned: for beams with flat concrete slabs on top, reference is made to generally accepted rules for flat concrete slabs; for steel slabs with profiled steel sheet, no tabulated data are given.

For the simple calculation models, the thermal response is often - but not always – based on the advanced heat transfer model explained in chapter 2. Basically, three options are used here:

[8] For verification of the integrity criterion, no calculation models exist so far and this criterion is therefore outside the scope of this course. For some possible solutions to meet the integrity criterion, refer to [2].

- the simple calculation model is based on semi-empirical rules, based on conventional assumptions; this approach is e.g. followed for composite columns with concrete between the flanges (see Fig. 15[b])
- the results of systematic calculations on basis of the advanced model are used in a parameter study in order to achieve simple calculation rules; this approach is followed for composite slabs (see Fig. 15[a])
- the advanced model as such is used in the simple model (which in fact is not that simple anymore) and the simplifications refer to the mechanical response; this approach has been used for the concrete filled SHS columns (see Fig. 15[b])

For a review of the various options available in EN 1994.1.2 with regard to the tabulated data and the thermal response analysis in simple models, refer to Annex C.

The basis for advanced thermal response models is (and should be!) the equation for heat transfer, as discussed in chapter 2. Refer to Figs. 3, 4, 5 for some examples. The procedures needed to solve this equation for composite elements are similar to those needed for concrete and steel, when using advanced thermal response models.

In the next sections, applications of the above principles will be discussed:
- calculation rules for the thermal response of composite columns with partially encased steel sections (i.e. steel columns with concrete between the flanges)
- simple calculation rules for verifying the thermal insulation criterion for concrete slabs with profiled steel sheet
- simple calculation rules for assessing the temperature in the positive (sagging moment) reinforcement in concrete slabs with profiled steel sheet
- the use of an advanced thermal response model in the "simple" model for calculating the fire resistance of concrete filled SHS columns.

4.2 Simple rules for the thermal response of composite columns with partially encased steel sections

To account for the thermal response of composite columns with a partially encased steel section, the cross section is divided into four components:
- the flanges of the steel profile
- the web of the steel profile
- the concrete contained by the steel profile
- the reinforcing bars.

Each component is evaluated on the basis of a reduced strength and stiffness (depending on the average temperature). For the concrete and the web of the steel section, also a reduced cross section is taken into account. See Fig. 16.

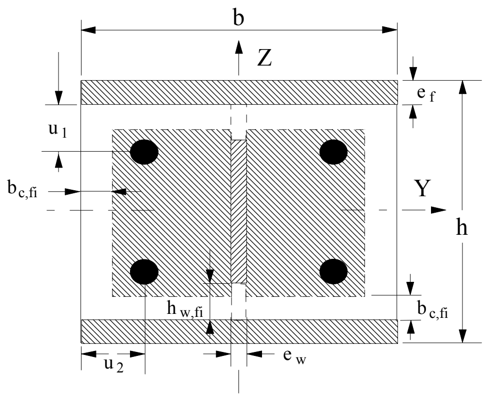


Fig. 16: Reduced cross section of composite column with partially encased steel column

The simple calculation rules apply only for standard fire conditions and four side exposure.

The average temperature and the reduction of the concrete cross section depend on the time of standard fire exposure (i.e. 30, 60, 90 and 120 minutes) and on the geometry of the cross section. The latter is represented by the Section Factor, which takes (in this case) the form (see also Fig. 16):

$$A_m / V = \frac{2(b+h)}{b.h} \quad \dots (5)$$

The necessary relationships are of a semi-empirical nature and are derived from standard fire tests. For a comprehensive review, see [2]. Some backgrounds are given in [20].

4.3 Calculation rules for verifying the thermal insulation criterion for composite concrete slabs with profiled steel sheet

In simple calculation models, the criterion for thermal insulation is identical to the one applied in standard fire testing, i.e.: the temperature increase at the non-directly exposed side of the elements shall not be more than 140 ˚C on the average or not more the 180 ˚C at any point [14], whichever is decisive. In case of composite concrete slabs with profiled steel sheet, the temperature at the non-directly exposed surface varies as function of the position of the measuring point, due to the profiled shape of the cross section. See Fig. 17.

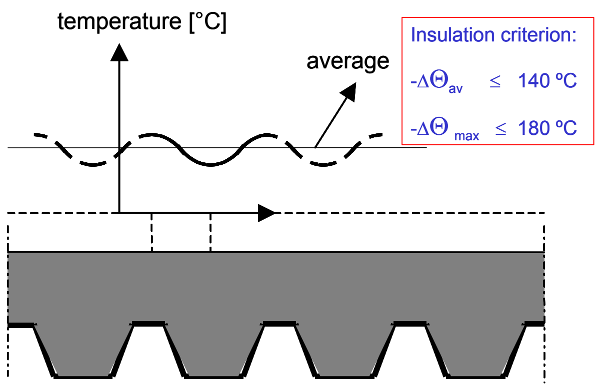


Fig. 17: Typical temperature distribution at the unexposed side of fire exposed slabs

In the simple calculation model for evaluating the insulation criterion, this effect is taken into account. To this end, systematic thermal response calculations have been performed with steel deckings currently available in Europe, including both trapezoidal and re-entrant profiles. For a review, refer to Table 1.

Table 1: Thermal calculations with currently available deckings

Decking type	Concrete depth H_B [mm]	Concrete type
re-entrant (6x)	50, 60, 70, 80,	NCW and LWC
trapezoidal (49x)	90, 100, 110, 120	Eurocode 4, 1994

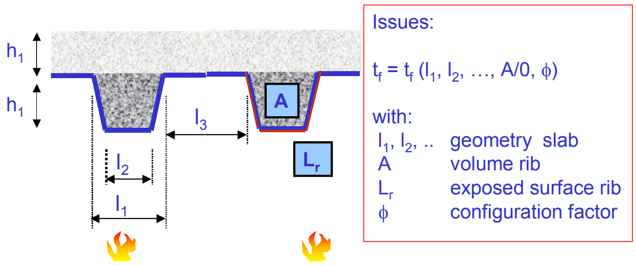
The calculations are based on equ. (1) and are carried out for both normal weight and light weight concrete. The following is assumed:

- standard fire conditions apply at the directly exposed side (i.e. the underside)
- heat transfer conditions at the exposed side (i.e. convection and radiation) account for the profiled shape of the slabs and the effect of the zinc lay-layer; for details refer to [15]
- the thermal conductivity (λ_c) and the thermal capacity ($\rho_c c_c$) of concrete are taken in accordance with Eurocode assumptions
- average moisture contents of 4% for normal weight concrete and 5% for light weight concrete (by dry weight).

For each of the analysed cases (see Table 1), the time to meet the above insulation criterion (= $t_{f,i}$) is calculated. The results are analysed by means of linear regression, using the following parameters:

- rib geometry factor (A/L_r)
- view factor of the upper flange (Φ)
- height of the concrete slab (h_1)
- width of the upper flange (l_3).

The regression constants were determined with simple linear regression techniques available in standard spread sheet computer programmes. The procedure is reviewed in Fig. 18. For a more detailed description, refer to Annex D, taken from [2]. Complete backgrounds are presented in [15].



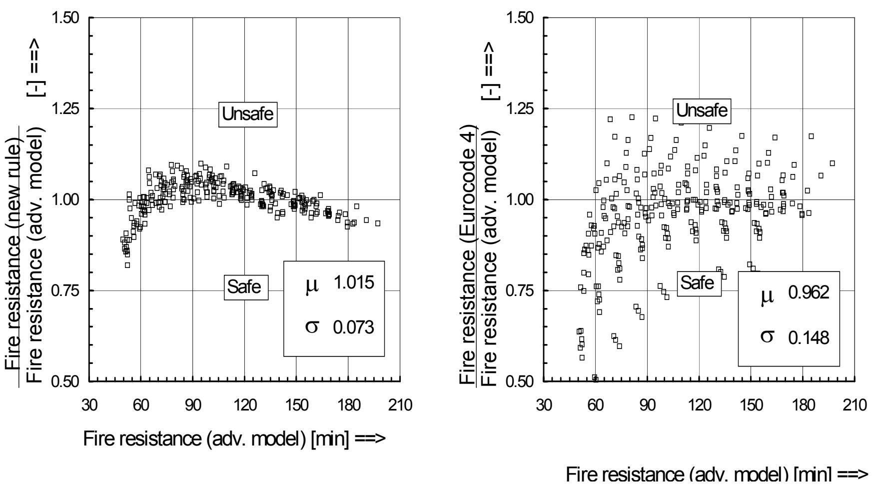
$t_f = a_0 + a_1 \cdot h_1 + a_2 \cdot \phi + a_3 \cdot A/L_r + a_4 \cdot 1/L_3 + a_5 \cdot A/L_r \cdot 1/l_3$ [min]

with:

a_i coefficients, depending on duration of s.f.c. exposure

Fig. 18: Thermal insulation composite slabs

The resulting equation for the resistance to fire with regard to the insulation criterion is specified in EN 1994-1.2 and replaces the corresponding equation given in the ENV version. In Fig. 19[a] a comparison is made between the outcomes of the simplified and the advanced model. For orientation, a similar comparison, however based on the rules given in the ENV version of the fire part of the Eurocode on composite structures, is presented in Fig. 19[b]. Conclusion is that applying the new rules, results in a more accurate assessment of the fire resistance with regard to insulation.



(a) assessment to new rule ((b) assessment to ENV

Fig. 19: Comparison of the rules for the insulation criterion to the outcomes of the advanced model

For a review of the corresponding regression coefficients, refer to Table 2.

Table 2: Coefficients for determination of the fire resistance with respect to thermal insulation

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

4.4 Calculation rules for the positive (sagging moment) reinforcement of composite slabs with profiled steel sheet

Information on the temperature distribution in the cross section of a composite slab is necessary in order to calculate the plastic moment capacity. Distinction is made between sagging moment capacity (often at mid span) and the hogging moment capacity (at the support, if appropriate). In this section focus is on the temperature in the additional reinforcement, applied to affect the sagging moment capacity.

The temperature in the additional reinforcement (if any), commonly placed in the centre line of the ribs, is of particular importance for the sagging moment capacity[9]. The temperature of such rebars is strongly influenced by fire exposure.

In a similar way as described in 4.3for the insulation criterion, regression formulae have been developed, giving the temperature of the additional reinforcement (Θ_r) as function of the main parameters, i.e.:
- distance to the lower flange (u_3)
- position of the rebar in the rib (z), according to Annex E
- angle of the web (α)

Based on systematic calculations, the following equation is found:

$$\theta_s = c_0 + \left(c_1 \cdot \frac{u_3}{h_2} \right) + \left(c_2 \cdot z \right) + \left(c_3 \cdot \frac{A}{L_r} \right) + \left(c_4 \cdot \alpha \right) + \left(c_5 \cdot \frac{1}{\ell_3} \right) \qquad \dots (6)$$

The procedure is reviewed in Fig. 20. For a detailed description, refer to Annex E. Complete backgrounds are presented in [15].

[9] The temperatures in the upper part of the cross section (compressive zone!) are low. That is why in calculations of the sagging moment capacity, room temperature values for the concrete strength are assumed.

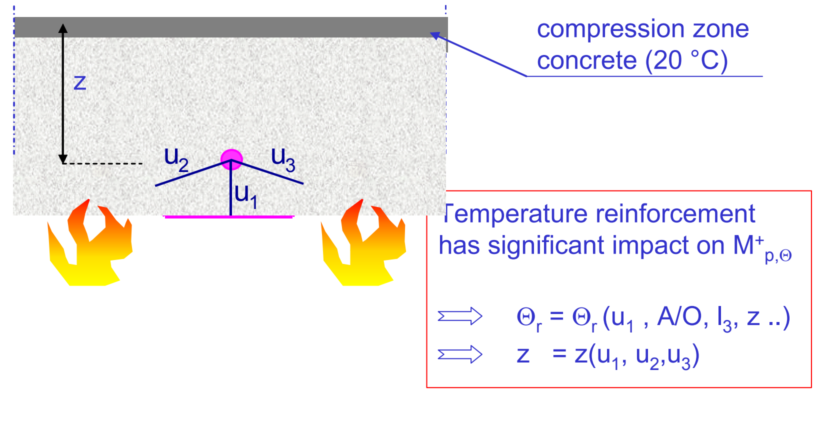


Fig. 20: Calculation rules for the temperature in the positive

The regression constants depend on the time of standard fire exposure and are –both for normal weight and lightweight concrete – given in Table 3. In Fig. 21^{a}, a comparison is made between the outcomes of the simplified and the advanced model. For orientation, a similar comparison, however based on the rules given in the ENV version of the fire part of the Eurocode on composite structures [16], is presented in Fig. 21^{b}. Conclusion is that applying the new rules, results in a more accurate assessment of the temperature of the additional reinforcement.

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

Concrete	Fire resistance [min]	c_0 [°C]	c_1 [°C]	c_2 [°C/mm$^{0.5}$]	c_3 [°C/mm]	c_4 [°C/°]	c_5 [°Cmm]
Normal weight concrete	60	1191	-250	-240	-5.01	1.04	-925
	90	1342	-256	-235	-5.30	1.39	-1267
	120	1387	-238	-227	-4.79	1.68	-1326
Light weight concrete	30	809	-135	-243	-0.70	0.48	-315
	60	1336	-242	-292	-6.11	1.63	-900
	90	1381	-240	-269	-5.46	2.24	-918
	120	1397	-230	-253	-4.44	2.47	-906

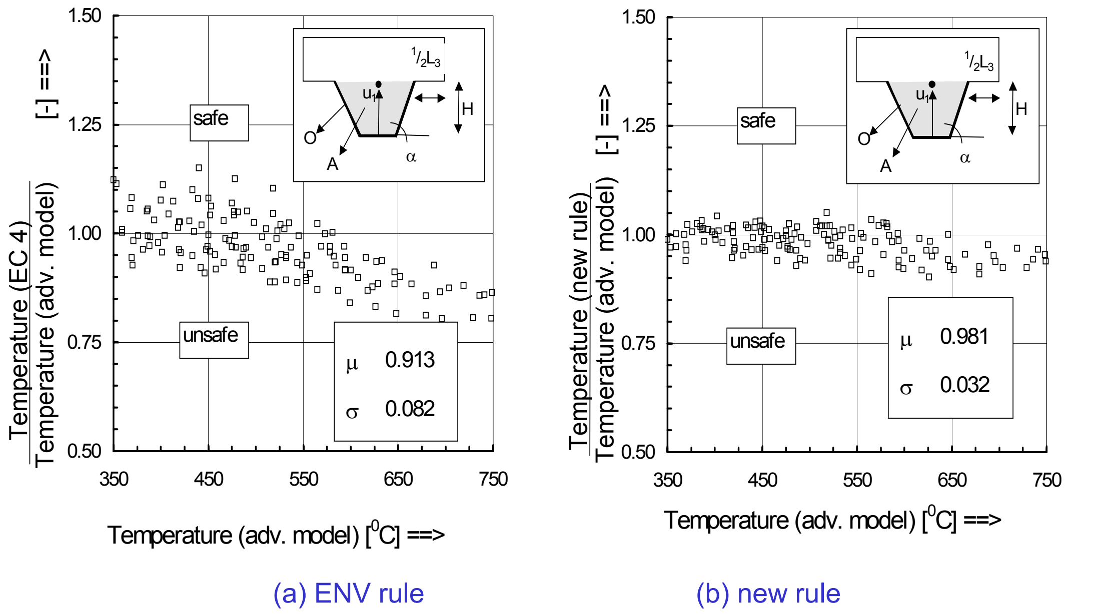


Fig. 21: Comparison of the simple rules for the temperature of the additional reinforcement with outcomes of the advanced model

The above approach is used in EN 1994.1.2. Note in this respect that also the steel decking may significantly contribute to the sagging moment capacity. That is why in EN 1994-1.2 also simple calculation rules for the temperature development in the various parts of the steel sheet are given. The nature of these rules is similar to the one described here for the temperature in the additional reinforcement.

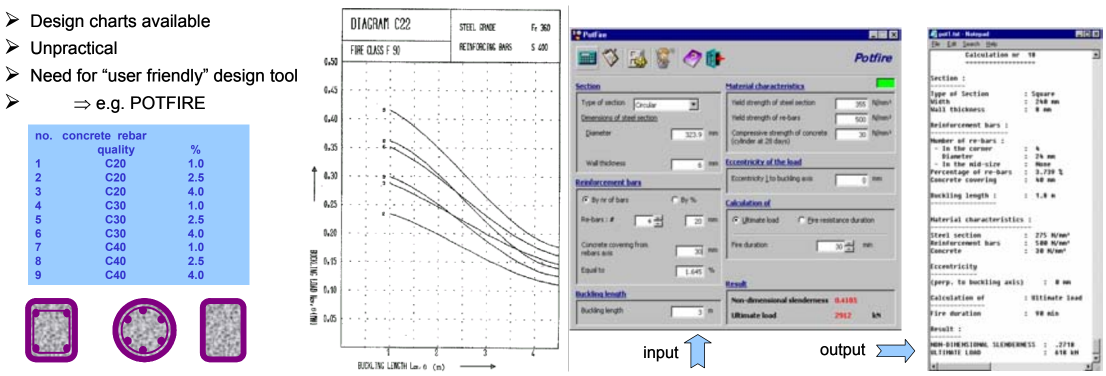
4.5 The thermal response model used for the calculation of the fire resistance of concrete filled SHS columns

The simple rules for concrete filled SHS columns are – as far as the thermal response is concerned – based on the advanced calculation model according to equ. (1), while simplifications are in the mechanical response model. The thermal response is further based on standard fire conditions in combination with heat transfer conditions specified in [7]. Main parameters are[10]:

- time of (standard) fire exposure
- cross sectional dimensions of the SHS column.

Such an approach implies that that no simple analytical calculation rules for the fire resistance of concrete filled SHS columns exist. Hence, a large number of graphs would be necessary to provide the user with straightforward design information. See for example the CIDECT Design Guide for SHS Columns Exposed to Fire, in which such design information – based on the ENV version of EN1994.1.2 - is presented, including more than 40 diagrams [17]. For illustration, refer to Fig. 22[a].

[10] Relevant mechanical response parameters are: steel grade, concrete quality, % of reinforcement.



(a) traditional approach (b) POTFIRE approach

Fig. 22: Fire design of concrete filled HSS columns

This is why a special purpose, user friendly computer programme has been developed for the determination of the fire resistance of concrete filled SHS columns under standard fire conditions: POTFIRE [18]. This programme is fully in line with the Eurocode assumptions. In Fig. 22^b the input and output screens are dispatched. The programme is extensively verified against the outcomes of (standard) fire tests [19]. By way of example, refer to Fig. 23 in which the results of temperature measurement in various points of the cross section of a (standard) fire exposed SHS column are compared to the outcomes of calculations by POTFIRE [19]. The agreement is satisfying.

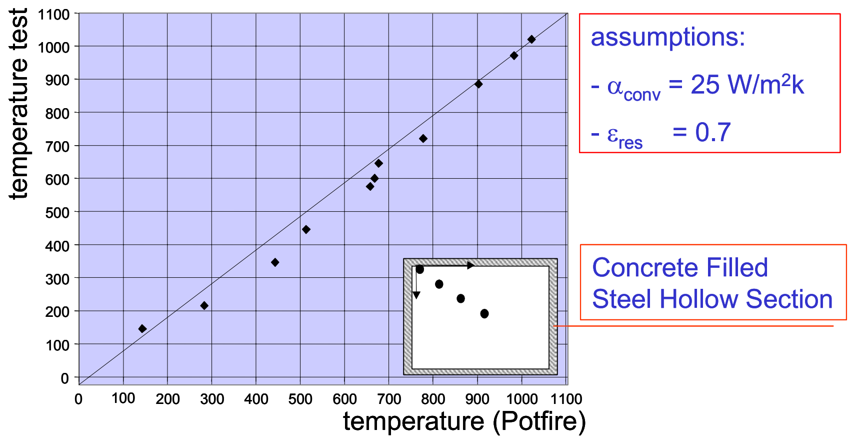


Fig. 23: Verification Potfire model

Note: it is important to realize that even not so “simple” models like POTFIRE are associated with a field of application. See Table 4. It will be clear from Fig. 23 that this is rather due to uncertainties in the mechanical response model than due to uncertainties in the thermal response model.

Table 4: Field of application for fire design of concrete filled SHS columns according to EN1994.1.2

lower limit	aspect	upper limit
0	Buckling length	13.5 m
230 mm	Height of cross section	1100 mm
230 mm	Width of cross section	500 mm
0 %	Percentage of reinforcing steel	6%
0 min	Standard fir resistance	120 min

4.6 Evaluation

The calculation of the thermal response of composite concrete steel elements is – compared to the analysis of the thermal response of bare and insulated steel elements – complicated. This is due to the fact that the temperature distribution in such elements generally is strongly non-uniform. To cope with this complication, EN 1994.1.2 offers the following tools:
- tabulated data
- simple calculation models.

Tabulated data are based on the experience, obtained from of standard fire test results.

The background of the simple calculation models varies significantly: sometimes they are based on direct interpretation of standard fire test results, taking on board some basic theoretical concepts. An example is the composite column with concrete between the flanges. In some applications, the simple rules follow from the generalisation of systematic calculations on basis of advanced calculations. For example: composite slabs with profiled steel sheet. In other cases, an advanced model is used (e.g. concrete filled SHS columns). A general feature of the simple models for thermal response analysis is that their use is practically limited standard fire conditions.

The Natural Fire Safety Concept (NFSC) approach is feasible for the thermal response of composite structures, but requires advanced modelling. A variety of necessary tools (i.e. computer codes) is available at present. See Part 4.

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ANNEX A: FOURIER'S DIFFERENTIAL EQUATION

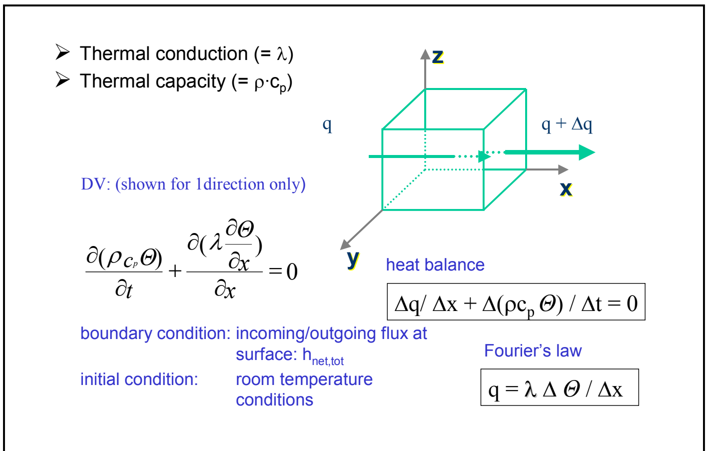


Fig. A.1: Thermal response: basics

Consider heat flow (q [W/m^2]) to volume element with ρ [kg/m^3], c_p [J/kg], λ [W/mK] and dimensions Δx, Δy, Δz [m] in x direction. Θ is temperature [°C]; t is time [s].

Heat balance: (no heat produced in volume element!)

$$\Delta q.\ \Delta y.\ \Delta z. + \Delta(\rho.\ c_p.\ \Theta).\Delta x.\ \Delta y.\Delta z = 0$$

$$\rightarrow \quad \Delta q / \Delta x + \Delta(\rho.\ c_p.\ \Theta) / \Delta t = 0$$

Fourier's law: (only in x-direction)

$$q = \lambda\ \Delta\Theta / \Delta x$$

Hence:

$$\Delta(\lambda\ \Delta\Theta / \Delta x) / \Delta x + \Delta(\rho.\ c_p.\ \Theta) / \Delta t = 0$$

For Δ → 0, the above differential equation (see Fig. A.1), which can easily be extended for y- and z-directions, results. This equation can (numerically!) be solved for known boundary conditions (→ thermal actions) and initial conditions (→ room temperature).

ANNEX B: THERMAL RESPONSE OF STEEL ELEMENTS, ASSUMING A UNIFORM STEEL TEMPERATURE

$$\left.\begin{array}{l} \dfrac{\partial(\rho c_p \Theta)}{\partial t} + \dfrac{\partial(\lambda \frac{\partial \Theta}{\partial x})}{\partial x} = 0 \\ \text{boundary \& initial conditions} \end{array}\right\} \Longrightarrow \frac{d\Theta_a}{dt} = \frac{\dot{h}_{tot} A_m}{\rho_a c_a V}$$

with
A_m is exposed surface area member [m²/m]
V is volume member [m³/m]

Note: key is uniform temperature distribution

Fig. B.1: Thermal response steel profiles (uniform steel temperature)

Assume uniform temperature distribution in cross section ($\lambda \rightarrow \infty$). Hence: $\partial \theta / \partial x \rightarrow 0$, $\partial \theta / \partial y \rightarrow 0$, $\partial \theta / \partial z \rightarrow 0$.

Assume a steel block (volume: V; exposed surface area: A_m), fully engulfed. The net heat flow entering the block during a time interval dt follows from:

$$\left(\int_A \dot{h}_{tot}\, dt\right) \bullet dt = \dot{h}_{tot} A\, dt \qquad \text{in [J]} \qquad \dots \text{(B.1)}$$

The increase of the heat content of the steel volume over time interval dt follows from (uniform temperature distribution!):

$$\rho_a c_a V\, d\,\Theta_a \qquad \text{in [J]} \qquad \dots \text{(B.2)}$$

Heat balance requires that the increase of the heat content of steel block equals the heat flow into the block. From equ. (B.1) and (B.2), after some rearrangement:

$$\frac{d\,\Theta_a}{d\,t} = \frac{A_m / V}{\rho_a c_a} \cdot \dot{h}_{net,tot} \qquad \dots \text{(B.3)}$$

with:

A_m/V	is	profile factor of the steel profile [m^{-1}]
$c_a\rho_a$	is	heat capacity of steel [J/m^3C]

This ordinary differential equation can be solved numerically for given initial and boundary conditions.

ANNEX C: TABULATED DATA AND SIMPLE MODELS ACCORDING TO EN 1994.1.2

Table C1: Overview tabulated data & thermal response analysis in simple models (horizontal elements)

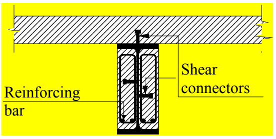
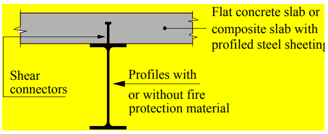
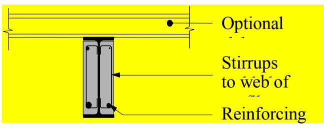
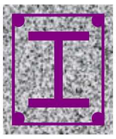
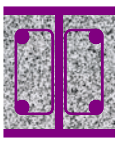
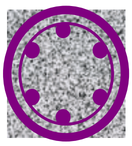
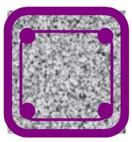
type of element	**tabulated data available**	**thermal response analysis used in simple model**
	yes	no
	no	semi-empirical approach
	no	semi-empirical approach
	no	generalisation of the outcomes of advanced calculation model

Table C2: Overview tabulated data & thermal response analysis in simple models (vertical elements)

type of element	**tabulated data available**	**thermal response analysis used in simple model**
	yes	no simple model
	yes	semi-empirical approach
  	yes	direct application of advanced thermal model

ANNEX D: EC RULES FOR FIRE RESISTANCE WITH RESPECT TO THERMAL INSULATION OF COMPOSITE SLABS WITH PROFILED STEEL SHEET

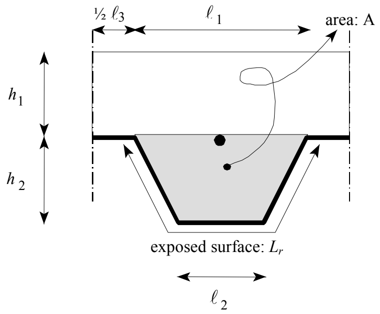
(1) The decisive fire resistance with respect to both the average temperature rise (=140°C) and the maximum temperature rise (=180°C), criterion "I", follows from the following equation:

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

where:

t_i	the fire resistance with respect to thermal insulation	[min];
A	concrete volume of the rib per m rib length	[mm^3/m];
L_r	exposed area of the rib per m rib length	[mm^2/m];
A/L_r	the rib geometry factor	[mm];
Φ	the view factor of the upper flange	[-];
l_3	the width of the upper flange (see Fig. D.1.1)	[mm].

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



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

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

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

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

ANNEX E: EC RULES FOR THE SAGGING REINFORCEMENT IN COMPOSITE SLABS WITH PROFILED STEEL SHEET

(1) Determine the temperature of the reinforcement bars in the rib, if any according to Fig. E.2.1, as follows:

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

where:

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

The factors c_i , for different values of the fire resistance and for both normal and light weight concrete, are given in table 3 of the main text. For intermediate values, linear interpolation is allowed.

(2) Determine the z-factor, which indicates the position of the reinforcement bar:

$$\frac{1}{Z} = \frac{1}{\sqrt{u_1}} + \frac{1}{\sqrt{u_2}} + \frac{1}{\sqrt{u_3}} \qquad \text{… (E.2)}$$

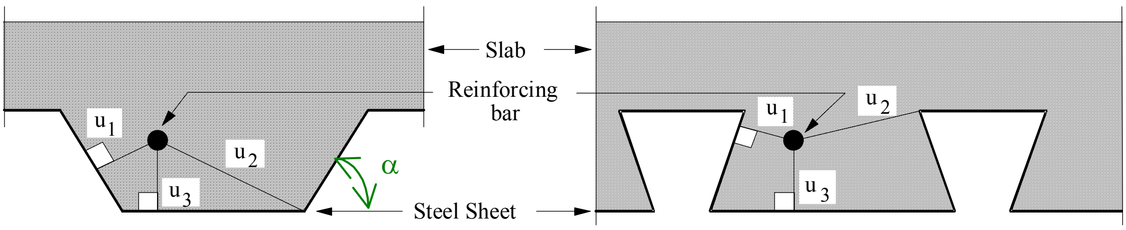


Fig. E.1: Position of the reinforcement

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

- u_1, u_2: shortest distance of the centre of the reinforcement bar to any point of the webs of the steel sheet;
- u_3: distance of the centre of the reinforcement bar to the lower flange of the steel sheet.