PART 3: Mechanical Response

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1 INTRODUCTION TO ANALYSIS OF MECHANICAL RESPONSE OF STRUCTURES IN FIRE SITUATION

During a fire, the mechanical response of the structure can be considered as the last one among different events illustrated in figure 1. It is also one of the most important impacts created by fire to building structures.

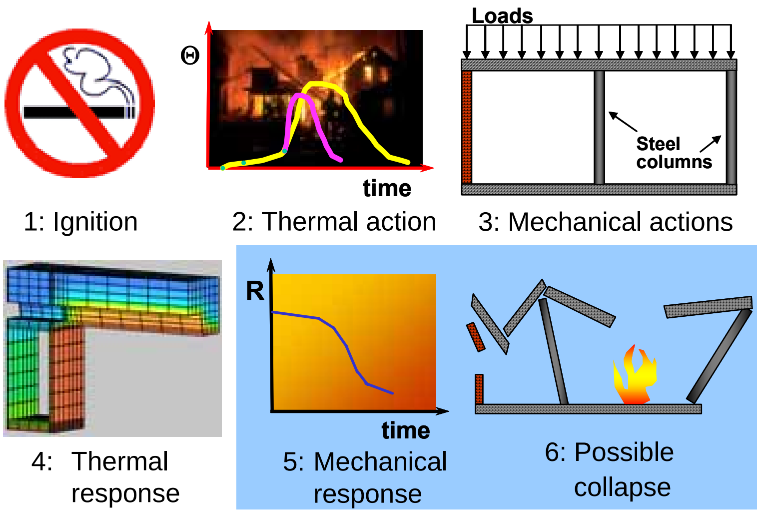


Figure 1 Resistance to fire – chain of event

It should be noted that the mechanical response of a structure under fire situation is directly related to how it behaves once subjected to fire. In general, the reaction of a structure to fire may be summarised as follows (see also figure 2):

- temperature rise called also as thermal response induced by the heat transfer from fire
- once the structure is heated, it will deform according to a thermal expansion coefficient which is usually positive
- at the same time, an important temperature rise will lead to a material softening so the loss of both stiffness and strength of the structure creating therefore additional structural deformation

- In some cases, the loss of strength and stiffness becomes so important that the structure is no longer capable of bearing the applied loads and collapse will occur consequently

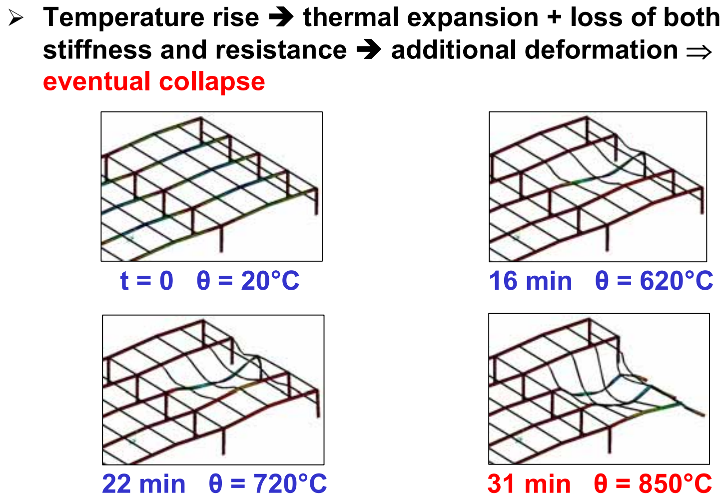


Figure 2 How does structure react to fire

It is not difficult to understand the general behaviour of a structure under fire situation. But it is extremely important for an engineer to be able to predict in an accurate way the structural behaviour of a building in order to know exactly its fire safety level. In actual fire safety engineering, there exist two major assessment approaches to evaluate the mechanical response of structures or structural members exposed to fire (see figure 3).

- As it is well known, the fire tests are always the available way to obtain the mechanical response of structures or structural members. Whatever the cost of them is, they will remain as a very useful tool to investigate the mechanical behaviour of structures exposed to fire
- On the other hand, it is more and more common for engineers to predict the mechanical performance of structures or structural members exposed to fire by means of design rules which are also the main aim of actual presentation

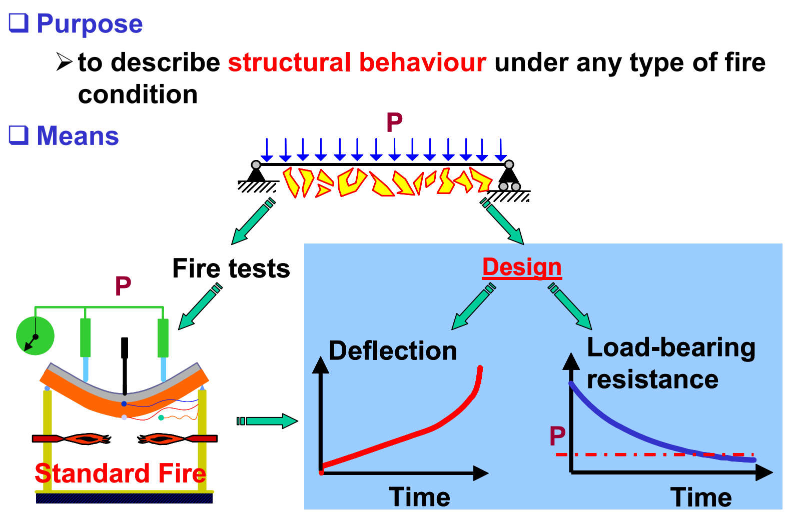


Figure 3 Assessment of mechanical response of structures in fire

2 GENERAL APPLICATION PRINCIPLES OF FIRE DESIGN TO STEEL AND COMPOSITE STRUCTURES

2.1 *Basic application features related to fire assessment of mechanical response of steel and composite structures*

As far as steel and composite structures are concerned, the assessment of their mechanical response under fire situation by means of design according to Eurocodes, needs to have a good knowledge of following features:

- First of all, the determination of the relevant mechanical loading that a structure is subjected to under fire situation
- Secondly, the appropriate temperature dependant material properties, such as stress-strain relationships, young's modulus, yield strength at elevated temperatures
- Thirdly, different design possibilities and their application domains with both simple calculation rules and advanced fire safety engineering tools
- Finally, specific points, such as special construction details, connection components of different structural members, which are not taken into account directly in normal fire design rules but extremely important to ensure an enough fire safety level

2.2 *Mechanical loading - Combination according to Eurocodes*

Under fire situation, the applied loads to structures can be obtained according to following formula (see relation 6.11b of EN1990):

$$\sum_{i\geq 1} G_{k,j} + (\Psi_{1,1} \text{ or } \Psi_{2,1})Q_{k,1} + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i}$$

where:

$G_{k,j}$: characteristic values of permanent actions
$Q_{k,1}$: characteristic leading variable action
$Q_{k,i}$: characteristic values of accompanying variable actions
$\psi_{1,1}$: factor for frequent value of a variable action
$\psi_{2,i}$: factor for quasi-permanent values of variable actions

The recommended values of ψ_1 and ψ_2 are given in table A1.1 of EN1990 but could be modified in National Annex.

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Another important notation largely used in fire design methods of Eurocodes is the load level for the fire situation $\eta_{fi,t}$ which is defined as $\eta_{fi,t} = \frac{E_{d,fi}}{E_d}$ with E_d and $E_{d,fi}$ respectively design effect of actions at room temperature design and design effect of actions for the fire situation. It can be alternatively determined by:

$$\eta_{fi,t} = \frac{G_k + \psi_{fi,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

where $\gamma_{Q,1}$ is the partial factor for leading variable action 1.

In fact, the load level η_{fi} depends strongly on the factor $\psi_{1,1}$ which varies as function of building categories. In EN1993-1-2 (fire part for steel structures) and EN1994-1-2 (fire part for composite structures), following figure (figure 4) is provided to show clearly the influence of both load ratio $Q_{k,1} / G_k$ and the factor $\psi_{1,1}$ on load level.

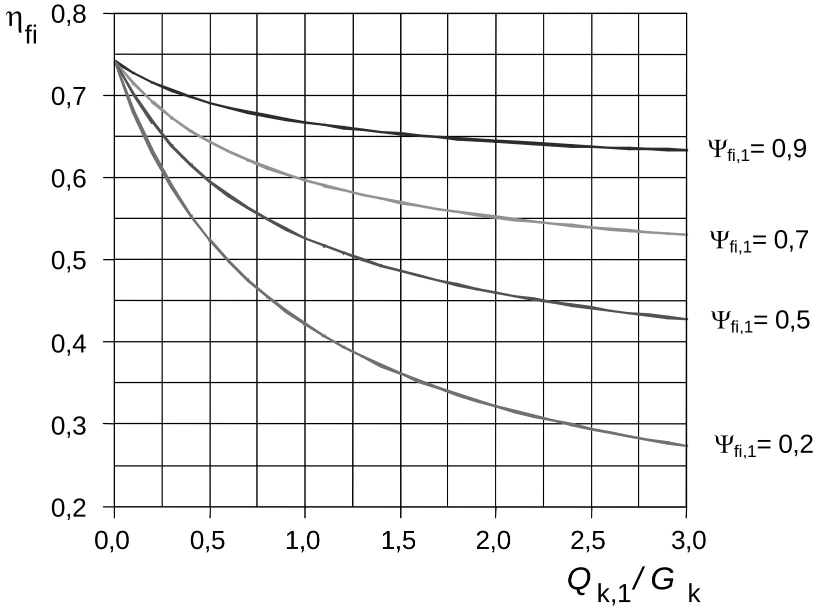


Figure 4 Variation of the reduction factor η_{fi} with the load ratio $Q_{k,1} / G_k$

In addition to above method for calculating the load level $\eta_{fi,t}$, another more realistic and more practical way of determining it is:

$$\eta_{fi,t} = \frac{E_{d,fi}}{R_d}$$

where R_d is the loadbearing capacity in room temperature design and there is certainly $E_d \leq R_d$.

The load level obtained from above relation is in general less important than with design load at room temperature design, as a consequence, leading to more economic fire design.

2.3 *Basic material mechanical properties of steel and composite structures at elevated temperatures*

2.3.1 *Stress-strain relationships of steel at elevated temperatures*

The two basic materials used for steel and composite structures are steel and concrete. It is therefore necessary to have their mechanical properties at elevated temperatures. EN1993-1-2 and EN1994-1-2 have provided detailed information related to these two materials. Concerning structural steel, its strength as function of temperature as well as its stress-strain relationships at elevated temperatures is illustrated in figure 5. One can find that the steel starts to lose strength and stiffness significantly from 400 °C. At 600 °C, its stiffness could be reduced by about 70% and its strength reduced by about 50%

The detailed steel's mechanical properties at elevated temperatures can be obtained using the data given in table 3.1 and figure 3.1 of EN1993-1-2.

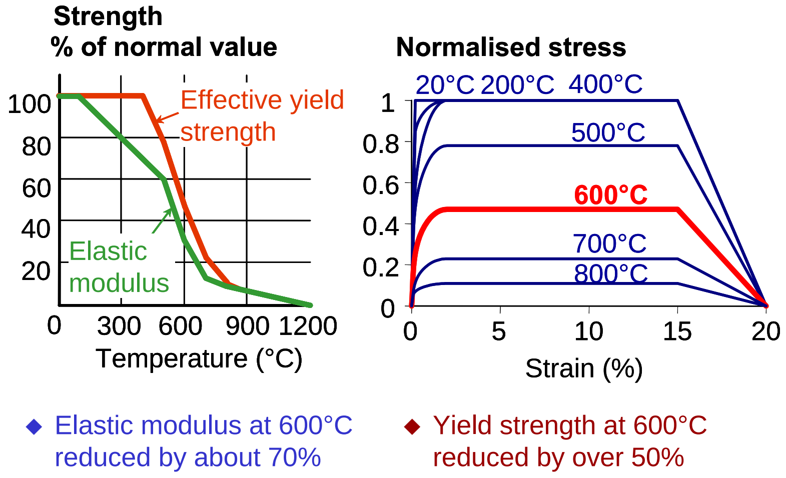
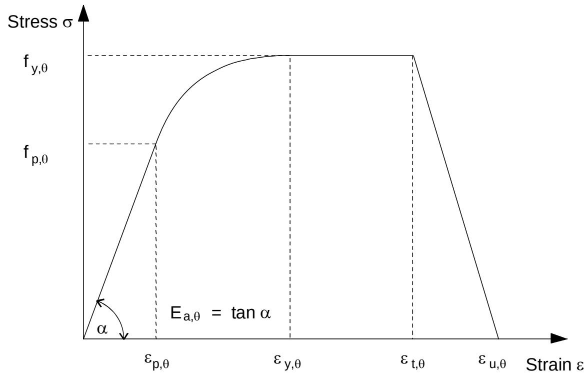


Figure 5 Mechanical properties of structural steel at elevated temperatures

Table 3.1: Reduction factors for stress-strain relationship of carbon steel at elevated temperatures

Steel temperature θ_a	Reduction factors at temperature θ_a relative to the value of f_y or E_a at 20 °C		
	Reduction factor (relative to f_y) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$	Reduction factor (relative to f_y) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$	Reduction factor (relative to E_a) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$
20 °C	1,000	1,000	1,000
100 °C	1,000	1,000	1,000
200 °C	1,000	0,807	0,900
300 °C	1,000	0,613	0,800
400 °C	1,000	0,420	0,700
500 °C	0,780	0,360	0,600
600 °C	0,470	0,180	0,310
700 °C	0,230	0,075	0,130
800 °C	0,110	0,050	0,090
900 °C	0,060	0,0375	0,0675
1000 °C	0,040	0,0250	0,0450
1100 °C	0,020	0,0125	0,0225
1200 °C	0,000	0,0000	0,0000
NOTE: For intermediate values of the steel temperature, linear interpolation may be used.			

Strain range	Stress σ	Tangent modulus
$\varepsilon \le \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$	$E_{a,\theta}$
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + (b/a)\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}$	$\dfrac{b(\varepsilon_{y,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0,5}}$
$\varepsilon_{y,\theta} \le \varepsilon \le \varepsilon_{t,\theta}$	$f_{y,\theta}$	0
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta}\left[1 - (\varepsilon - \varepsilon_{t,\theta})/(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})\right]$	-
$\varepsilon = \varepsilon_{u,\theta}$	0,00	-
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta}/E_{a,\theta}$ $\varepsilon_{y,\theta} = 0,02$	$\varepsilon_{t,\theta} = 0,15$ $\varepsilon_{u,\theta} = 0,20$
Functions	$a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c/E_{a,\theta})$ $b^2 = c\,(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2$ $c = \dfrac{(f_{y,\theta} - f_{p,\theta})^2}{(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})}$	



Key:
- $f_{y,\theta}$ effective yield strength;
- $f_{p,\theta}$ proportional limit;
- $E_{a,\theta}$ slope of the linear elastic range;
- $\varepsilon_{p,\theta}$ strain at the proportional limit;
- $\varepsilon_{y,\theta}$ yield strain;
- $\varepsilon_{t,\theta}$ limiting strain for yield strength;
- $\varepsilon_{u,\theta}$ ultimate strain.

Figure 3.1: Stress-strain relationship for carbon steel at elevated temperatures

2.3.2 *Stress-strain relationships of concrete at elevated temperatures*

Similarly, the mechanical properties of concrete at elevated temperatures can be obtained from EN1994-1-2 (see figure 6). If more attention is paid to the compressive strength of concrete at elevated temperatures, one can find easily that it falls gradually to about 50% of its room temperature strength at 600°C so quite similar to structural steel.

The detailed information to establish concrete's mechanical properties at elevated temperatures is given in table 3.1 and figure 3.1 of EN1994-1-2.

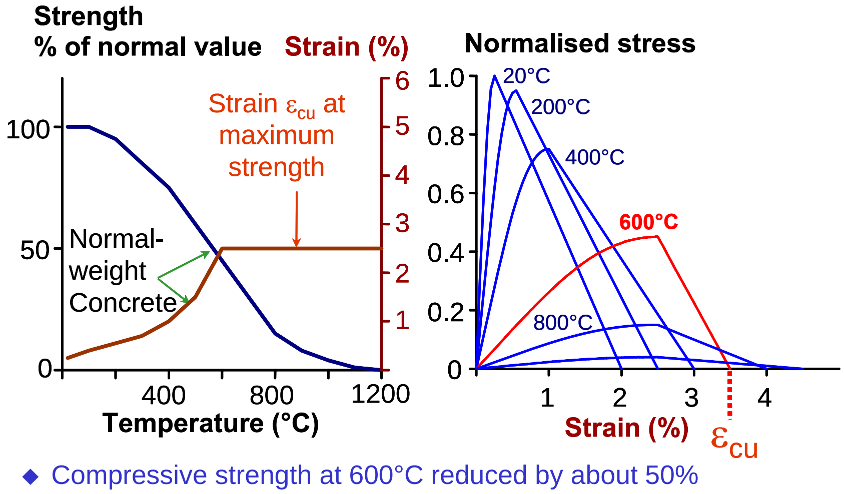
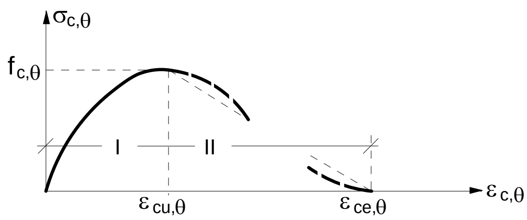


Figure 6 Mechanical properties of normal weight concrete at elevated temperatures



<u>RANGE I:</u>

$$\sigma_{c,\theta} = f_{c,\theta}\left[3\left(\frac{\varepsilon_{c,\theta}}{\varepsilon_{cu,\theta}}\right)\Bigg/\left\{2+\left(\frac{\varepsilon_{c,\theta}}{\varepsilon_{cu,\theta}}\right)^{3}\right\}\right]$$

$$\left.\begin{array}{l} k_{c,\theta} = \dfrac{f_{c,\theta}}{f_c} \\ \text{and } \varepsilon_{cu,\theta} \end{array}\right\} \text{ to be chosen according to the values of Table 3.3}$$

<u>RANGE II:</u>

For numerical purposes a descending branch should be adopted

Figure 3.2: Mathematical model for stress-strain relationships of concrete under compression at elevated temperatures

Table 3.3: Values for the two main parameters of the stress-strain relationships of normal weight concrete (NC) and light weight concrete (LC) at elevated temperatures

Concrete Temperature θ_c [°C]	$k_{c,\theta} = f_{c,\theta}/f_c$		$\varepsilon_{cu,\theta} \cdot 10^3$
	NC	LC	NC
20	1	1	2,5
100	1	1	4,0
200	0,95	1	5,5
300	0,85	1	7,0
400	0,75	0,88	10,0
500	0,60	0,76	15,0
600	0,45	0,64	25,0
700	0,30	0,52	25,0
800	0,15	0,40	25,0
900	0,08	0,28	25,0
1000	0,04	0,16	25,0
1100	0,01	0,04	25,0
1200	0	0	-

2.3.3 *Thermal expansion of steel and concrete*

Parallel to mechanical properties, the thermal expansion behaviour needs to be taken into account in a lot of fire safety engineering application cases, in particular with advanced calculation models.

Concerning this feature, EN1993-1-2 and EN1994-1-2 recommend to use the expansion curves given in figure 7 respectively for steel and concrete. The mathematic expression of these curves is given in figure 7.

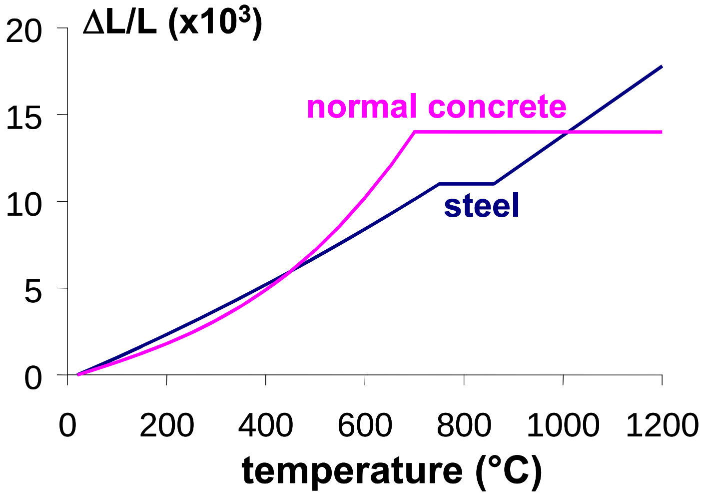


Figure 7 Thermal expansion of steel and concrete (EN1992-1-2, EN1993-1-2 and EN1994-1-2)

The detailed equation to establish above curve is given below.

Steel	$\Delta\ell / \ell = -2{,}416 \,.\, 10^{-4} + 1{,}2 \,.\, 10^{-5}\ \theta_a + 0{,}4 \,.\, 10^{-8} \theta_a^2$	for 20 °C < θ_a ≤ 750 °C
	$\Delta\ell / \ell = 11.10^{-3}$	for 750 °C < θ_a ≤ 860 °C
	$\Delta\ell / \ell = -6{,}2 \,.\, 10^{-3} + 2 \,.\, 10^{-5}\ \theta_a$	for 860 °C < θ_a ≤ 1200 °C
	Or in simple way: $\Delta\ell / \ell = 14 \,.\, 10^{-6}\ (\theta_a - 20)$	
Concrete	$\Delta\ell / \ell = -1{,}8 \,.\, 10^{-4} + 9 \,.\, 10^{-6}\ \theta_c + 2{,}3 \,.\, 10^{-11}\ \theta_c^3$	for 20 °C ≤ θ_c ≤ 700 °C
	$\Delta\ell / \ell = 14 \,.\, 10^{-3}$	for 700 °C < θ_c ≤ 1200 °C
	Or in simple way: $\Delta\ell / \ell = 18 \,.\, 10^{-6}\ (\theta_c - 20)$	
where:	ℓ is the length at 20°C of the steel or concrete member $\Delta\ell$ is the temperature induced elongation of the steel or concrete member θ_a and θ_c are respectively the steel or concrete temperature	

2.4 *Design approach for mechanical response of structures in fire situation*

Concerning the design of mechanical response of structures exposed to fire, it can be reached by following three approaches (see also figure 8):

- Member analysis, in which each member of the structure will be assessed by considering them fully separated from other members and the connection condition with other members will be replaced by appropriate boundary conditions
- Analysis of parts of the structure, in which a part of the structure will be directly taken into account in the assessment by using appropriate boundary conditions to reflect its links with other parts of the structure
- Global structural analysis, in which the whole structure will be used in the assessment

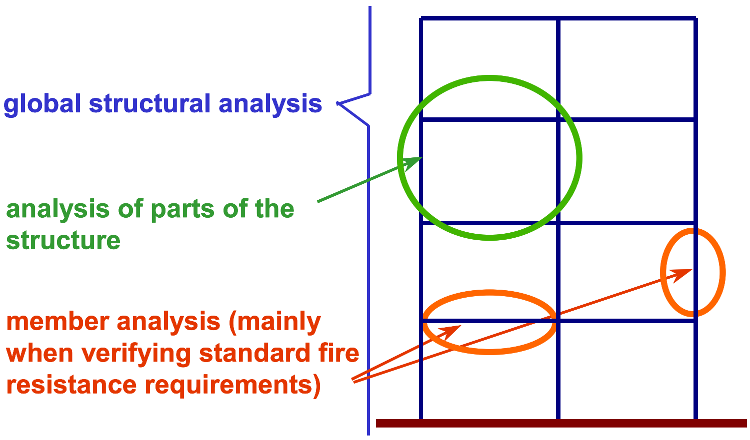


Figure 8 Different design approaches for mechanical response of structures in fire

Regarding above design procedures to assess the mechanical response of structures in fire, following remarks may be made:

- The member analysis will be applied to isolated structure element (element by element) so easy to use in particular with simplified calculation methods and therefore largely used under nominal fire condition (for example: ISO-834 standard fire)
- The analysis of parts of the structure or global structural analysis will consider at least several structural members together so that the interaction effect between them will be directly dealt with; load redistribution from heated parts (weakened parts inside fire

compartment) to cold parts (stronger parts outside fire compartment) can be taken into account in accurate way and the global behaviour of structures will be analysed providing therefore more realistic situation of mechanical response of structures in fire.

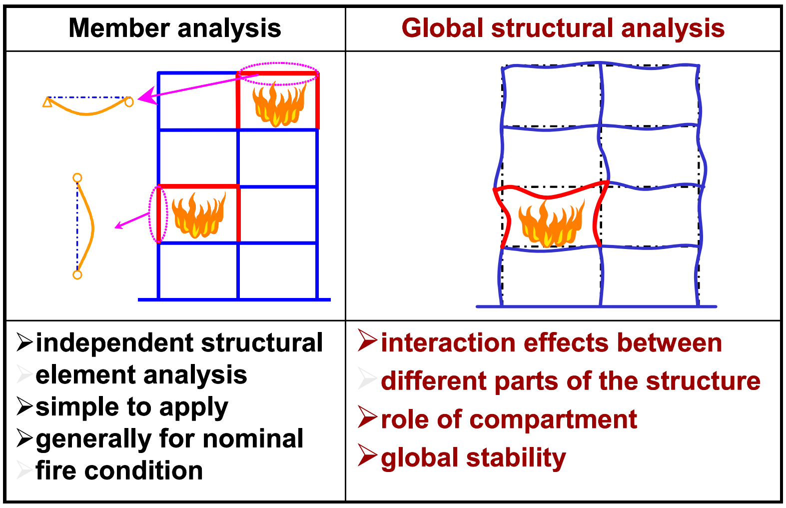


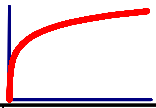
Figure 9 Comparison of different design approaches for mechanical response of structures in fire

According to current Eurocodes, three types of design methods can be used to assess the mechanical behaviour of structures under fire situation in different design approaches explained above. One can find notably:

- Simple calculation method based on predefined tabulated data, this method is only applicable to steel and concrete composite structures
- Simple calculation models, this type of design method can be divided into two different families, the first one is the famous critical temperature method widely applied to steel structural member analysis and the second one is all the simple mechanical models developed for both steel and composite structural member analysis.
- Advanced calculation models, this kind of design tools can be applied to all types of structures and they are in general based on either finite element method or finite difference method. In modern fire safety engineering, it becomes more and more employed design approach due to the numerous advantages that it can provide.

Before going into the detailed application of all above design methods, it is extremely important to get a good idea about the application domain of these design methods. The table given in figure 10 shows clearly the different application possibilities of three fire assessment methods under nominal (standard) fire condition. One can find easily that for member analysis, all three assessment methods may be applied. In very few cases, the simple calculation method can be also applied to the analysis of the mechanical resistance of a part of the structure subjected to fire, for example, simple steel portal frames. Therefore, the simple calculation methods are practically limited only to member analysis. Even under nominal fire situations, the fire design with complicated structures should be realised in general by using advanced calculation models.

❑Thermal action defined with nominal fires



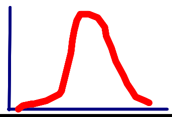
Type of analysis	Tabulated data	Simple calculation models	Advanced calculation models
Member analysis	Yes ISO-834 standard fire	Yes	Yes
Analysis of a part of the structure	Not applicable	Yes (if available)	Yes
Global structural analysis	Not applicable	Not applicable	Yes

Figure 10 Application domain of different design methods under standard fire situation

Under natural fire conditions, the application of simple calculation methods is largely limited since the heating behaviour of the member is fully different from that under standard fire condition. That's the reason why the table given in figure 11 shows a majority of negative applicable situation with simple calculation methods. The only example in which they can be used is the steel members with or without passive fire protection fully engulfed in fire.

Nevertheless, the application of advanced numerical models in case of natural fire conditions will not be limited due to the fact that they can predict both the accurate thermal response of all structural members subjected to variable thermal actions and the mechanical response of structural members, parts of the structure or entire structure by taking into account the real material strength and stiffness reduction factors, thermal expansion effect, temperature gradient, etc.

❑Thermal action defined
with natural fires



Type of analysis	Tabulated data	Simple calculation models	Advanced calculation models
Member analysis	Not applicable	Yes (if available)	Yes
Analysis of a part of the structure	Not applicable	Not applicable	Yes
Global structural analysis	Not applicable	Not applicable	Yes

Figure 11 Application field of different design methods under natural fire situation

All above application procedures and strategy are also clearly defined in all Eurocodes (see figure 12 shown below)

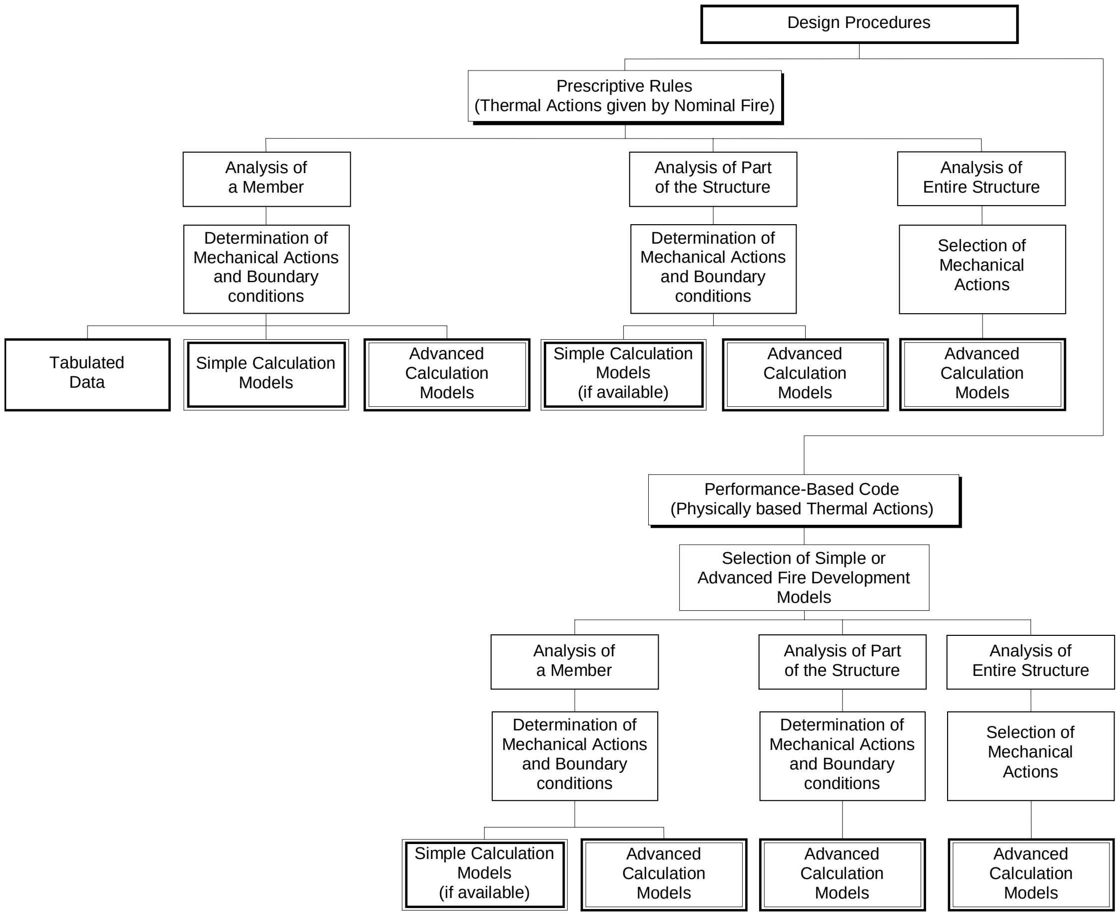


Figure 12 Alternative design procedures

3 PRINCIPLE DESCRIPTION OF CALCULATION MODELS FOR MEMBER ANALYSIS OF STEEL AND COMPOSITE STRUCTURES

3.1 *Tabulated data*

After having a good idea about the application domain of all design methods, it could be interesting to get a clear idea about the application principle of these design approaches. Let's start first of all from one of the mostly used simple calculation methods for steel and concrete composite member analysis, the tabulated data.

As it is shown in figure 13, this type of design model is applicable to following structural members:

- Steel and concrete composite beams with partially or fully concrete encasement of steel beams
- Steel and concrete composite columns with partially or fully concrete encased profiles
- Steel and concrete composite columns with concrete filled steel hollow sections (CHS or RHS)

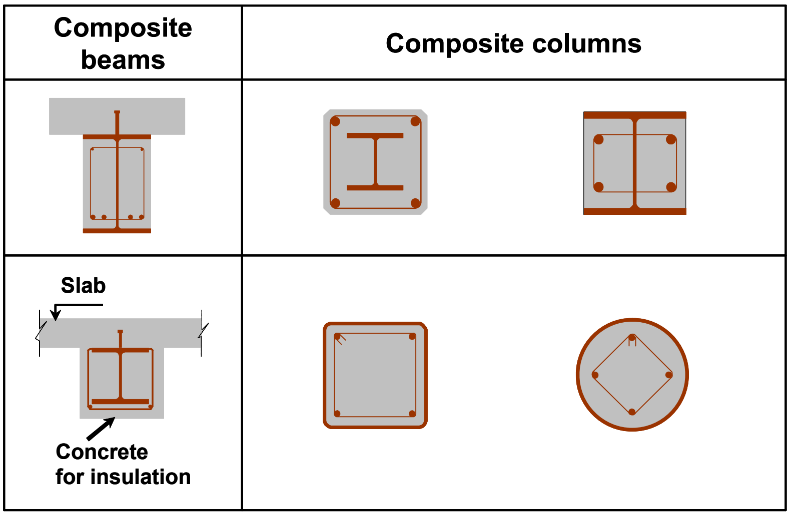
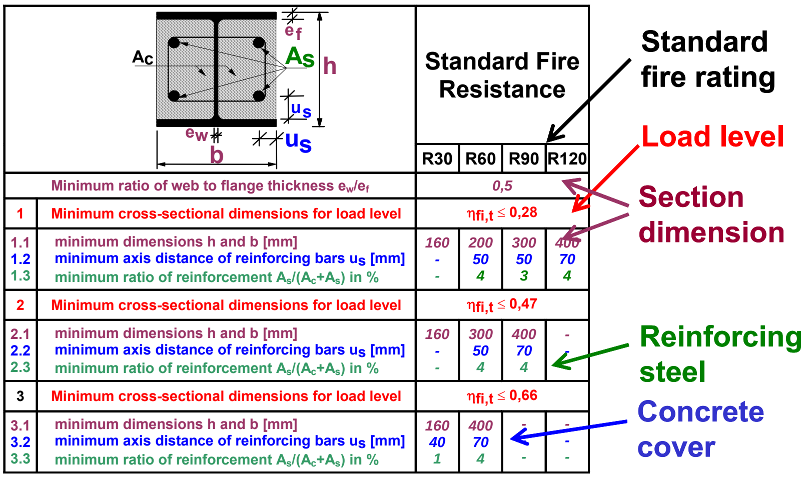


Figure 13 Application domain of design methods with tabulated data

What is the content of the design method with tabulated data for steel and concrete composite member analysis? It uses predefined values based mainly on standard fire test results and improved with analytical investigation as shown in figure 14. All the values relate together the specific standard fire ratings, the load level, the minimum dimensions of member section, the necessary reinforcing steel area and its minimum concrete cover in one or more tables in order to obtain quickly the member size to be used for a determined fire duration.

The most important advantage of this method is the easiness of its application and it shall give safer results compared to other simple calculation models or advanced calculation models. As a consequence, people as architects or engineers can apply it during the pre-design of a building to get the approximate minimum section size of structural members under fire situation.



		Standard Fire Resistance			
		R30	R60	R90	R120
	Minimum ratio of web to flange thickness e_w/e_f	0,5			
1	Minimum cross-sectional dimensions for load level	$\eta_{fi,t} \leq 0,28$			
1.1	minimum dimensions h and b [mm]	160	200	300	400
1.2	minimum axis distance of reinforcing bars u_s [mm]	-	50	50	70
1.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	-	4	3	4
2	Minimum cross-sectional dimensions for load level	$\eta_{fi,t} \leq 0,47$			
2.1	minimum dimensions h and b [mm]	160	300	400	-
2.2	minimum axis distance of reinforcing bars u_s [mm]	-	50	70	-
2.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	-	4	4	
3	Minimum cross-sectional dimensions for load level	$\eta_{fi,t} \leq 0,66$			
3.1	minimum dimensions h and b [mm]	160	400	-	-
3.2	minimum axis distance of reinforcing bars u_s [mm]	40	70		-
3.3	minimum ratio of reinforcement $A_s/(A_c+A_s)$ in %	1	4	-	-

Figure 14 What is the design method with tabulated data (example of partially encased concrete composite columns)

The application of simple calculation method with tabulated data can be made under two different situations (see figure 15), one for verification case where the dimension of structural members is already known and another one for pre-design case where only the design action is defined.

In verification cases, the cross section dimensions of structural member as well as the loadbearing capacity of the member R_d are already know, one can calculate the mechanical action in fire situation $E_{fi,d}$ in order to derive the load level $\eta_{fi,t} = E_{fi,d}/R_d$. From the value of load level and the dimension and construction requirements of cross section of the member, the tabulated data gives the fire rating to be provided by the member.

In pre-design cases, the cross section dimensions of structural member are not defined. On the contrary, one knows the effects of actions E_d and $E_{fi,d}$ from the appropriate load combination for room temperature and fire designs. In this case, one can adopt safely that the load level $\eta_{fi,t} = E_{fi,d}/E_d$. Based on this value and the standard fire rating, the minimum cross section dimensions as well as relevant construction condition of the member can be defined. Then such defined cross section will be checked for room temperature design, that is $R_d \geq E_d$.

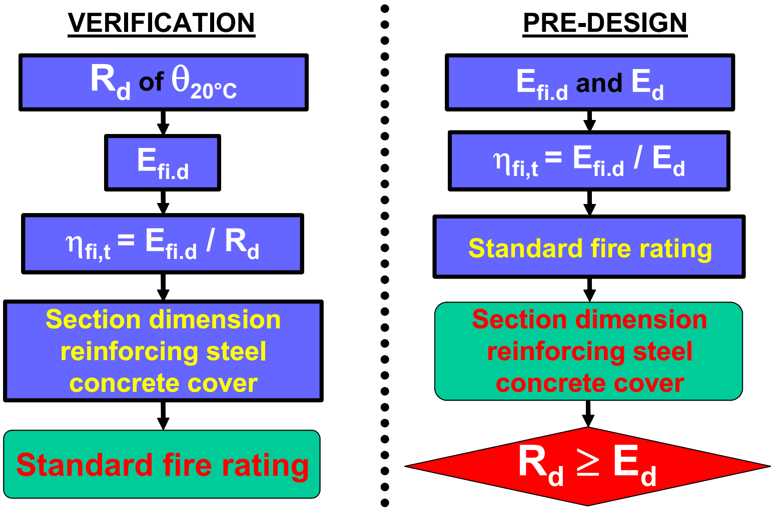


Figure 15 Application of tabulated data in fire design under two different situations

3.2 *Simple calculation models*

Compared to design method with tabulated data, the simple calculation models may be applied to both steel and steel and concrete composite members, so it covers an application domain much larger than tabulated data.

As it is shown in figure 16, this type of design model is applicable to following structural members:

- Almost all types of steel members, such as the tensile elements, beams, columns etc with or without passive fire protection
- Steel and concrete composite beams with or without concrete encasement of steel profiles

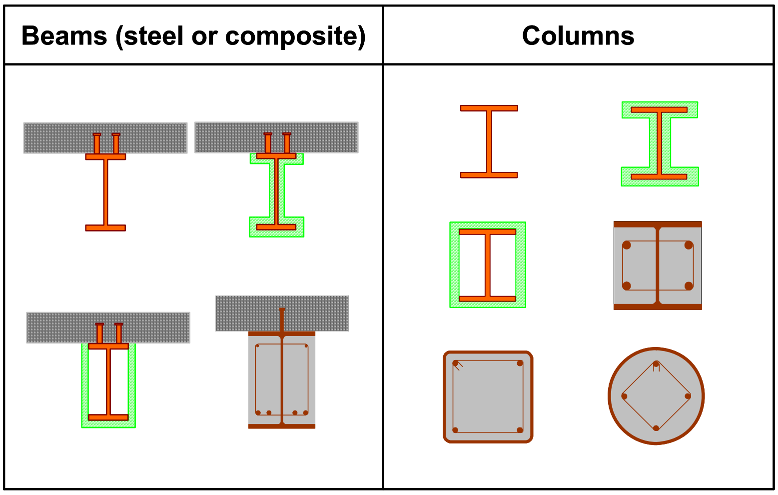


Figure 16 Application domain of design methods with simple calculation models

The design method with simple calculation models can be divided into three following families:

- members subjected to either axial force or bending moment without any instability problem, in this case, the simple calculation model is based on plastic diagram of cross section at elevated temperatures
- members under simple axial compression force but implying instability phenomenon, such as axially loaded slender columns, in this case, the simple calculation method is generally based on buckling curve approach adapted for fire situation
- members subjected to combined bending and axial compression, such as slender columns under eccentric load, long beams with lateral buckling, etc, for this type of members, the simple calculation model takes into account the combination effect of bending and compression by combining above two models for simple loading condition

3.2.1 *Example of simple calculation models – steel and concrete composite beams exposed to fire*

One typical example of first family members is the simply supported steel and concrete composite beam as shown in figure 17. In the simple calculation model, the temperature of steel section may have three different values corresponding to lower flange, web and upper flange of steel section and for the temperature of concrete slab one dimensional temperature gradient through its thickness is considered. In this case, it is quite easy to establish the plastic stress equilibrium diagram and calculate bending moment resistance of its cross section from which the loadbearing capacity of the beam may be derived.

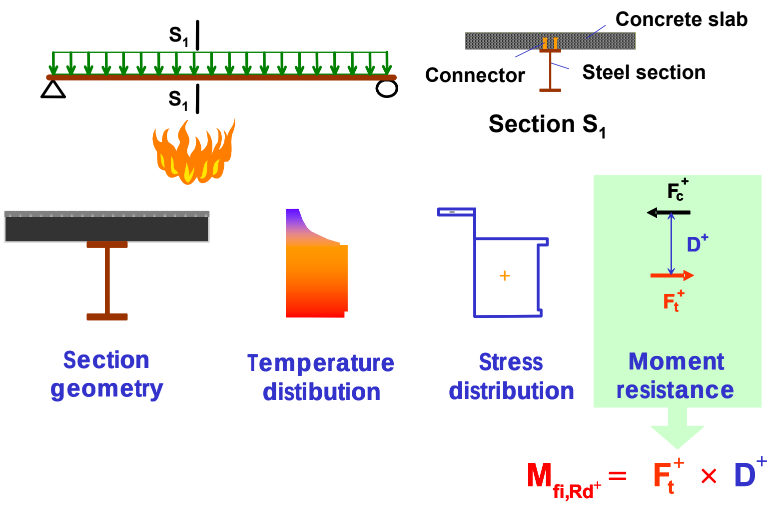


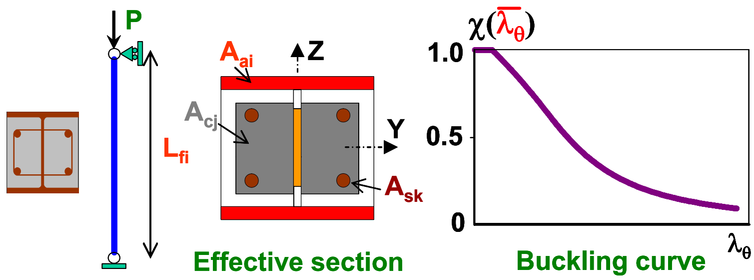
Figure 17 Example of steel and concrete composite beams exposed to fire

3.2.2 *Example of simple calculation models – partially concrete encased composite columns exposed to fire*

Another typical example of simple calculation models is the steel and concrete composite columns with partially concrete encased profiles (see figure 18).

In general, following points are expected:

- Loadbearing capacity may be simply defined in relating axial plastic section resistance at elevated temperatures $N_{fi,pl,Rd}$ with reduction coefficient of relevant buckling curve $\chi(\bar{\lambda}_\theta)$
- The reduction coefficient of relevant buckling curve $\chi(\bar{\lambda}_\theta)$ depends on the relative slenderness in fire situation $\bar{\lambda}_\theta$ which in turn is related to axial plastic section resistance $N_{fi,pl,Rd}$, effective rigidity of cross section $(EI)_{eff,fi}$ and the buckling length L_{fi} at elevated temperatures



Load capacity: $N_{fi.Rd} = \chi(\bar{\lambda}_\theta)\, N_{fi.pl.Rd}$

with:

Relative slenderness: $\bar{\lambda}_\theta = (N_{fi.pl.Rd} / N_{fi.cr})^{0.5}$

Plastic load: $N_{fi.pl.Rd} = \sum A_{ai} f_{ay,\theta i} / \gamma_{M,fi,a} + \sum A_{cj} f_{c,\theta j} / \gamma_{M,fi,c} + \sum A_{sk} f_{s,\theta k} / \gamma_{M,fi,s}$

Euler buckling load: $N_{fi.cr} = \pi^2 (E\,I)_{eff,fi} / L_{fi}^2$

Effective rigidity: $(EI)_{eff,fi} = \sum \varphi_{a,\theta} E_{a,\theta i} I_{ai} + \sum \varphi_{c,\theta} E_{c,\theta j} I_{cj} + \sum \varphi_{s,\theta} E_{s,\theta k} I_{sk}$

Figure 18 Example of the design method with simple calculation models for partially concrete encased composite columns

It can be found that in case of members having instability problem, their fire resistance should be evaluated not only on the basis of strength at elevated temperatures but also with stiffness included.

3.3 *Critical temperature method*

Among simple calculation models given in Eurocodes 3 and 4, one can find a specific method called as "critical temperature method". In principle, this method is applicable only to structural members comprising steel section heated uniformly or with slight temperature gradient. As a consequence, this method may be applied to following structural members (see figure 19):

- Protected or non protected steel or composite beams with steel section exposed to four or three sides
- Steel columns with or without passive fire protection engulfed entirely in fire
- Tensile members exposed to fire

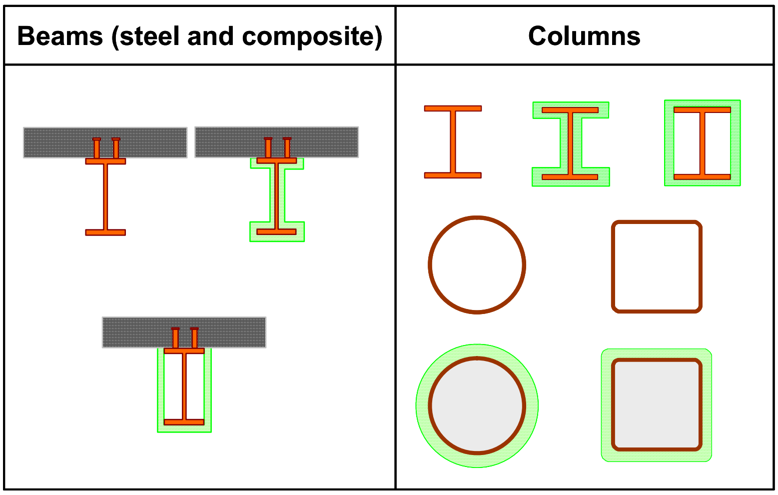


Figure 19 Application domain of the design method with critical temperature

The critical temperature method is in fact based on simple calculation models for steel members heated uniformly. In this case, it is not difficult to understand that the strength of the member at elevated temperatures $R_{fi,d,t}$ can be obtained by multiplying the member resistance at 20°C $R_{fi,d,o}$ with the strength reduction factor $k_{y,\theta}$, that is $R_{fi,d,t} \geq k_{y,\theta} R_{fi,d,0}$.

On the other hand, the fire resistance of the member is satisfied if $R_{fi,d,t} \geq E_{fi,d}$. From this relation, one can easily have $R_{fi,d,t} \geq \mu_0 R_{fi,d,0}$ (see figure 20) with $\mu_0 = E_{fi,d} / R_{fi,d,0}$ defined as utilisation level. Therefore, to have an enough fire resistance of the member, there must be $k_{y,\theta} \geq \mu_0$. In case where $k_{y,\theta} = \mu_0$ (also the optimum case to satisfy required fire resistance), the corresponding temperature θ_{cr} is defined as critical temperature.

This critical temperature may be obtained on the basis of the values of $k_{y,\theta}$ given in table 3.1 of EN1993-1-2. However, in most cases, interpolation is necessary to get the exact critical temperature value. In order to overcome this inconvenience, a simple formula based directly on utilisation level μ_0 is proposed to calculate very quickly the critical temperature, namely:

$$\theta_{cr} = 39.19 \ell n \left[\frac{1}{0,9674 \, \mu_0^{3,833}} - 1 \right] + 482$$

If two curves respectively with $k_{y,\theta}$ and μ_0 versus temperature are established in the same figure (see figure 20), one can find that they are almost overlapped showing the validity of using this formula to determine the critical temperature of any appropriate structural member exposed to fire.

- **According to simple calculation models, for uniformly heated steel members: $R_{fi,d,t} = k_{y,\theta}\, R_{fi,d,0}$**
- **On the other hand, fire resistance should satisfy:**

$$R_{fi,d,t} \geq E_{fi,d} = \frac{E_{fi,d}}{R_{fi,d,0}} R_{fi,d,0} = \mu_0 R_{fi,d,0} \Rightarrow k_{y,\theta} \geq \mu_0$$

- **In particular, when $k_{y,\theta} = \mu_0$ the corresponding temperature is defined as critical temperature θ_{cr}**
- **In EN1993-1-2, a simple formula is given to determine critical temperature θ_{cr}**

$$\theta_{cr} = 39.19 \ln\left[\frac{1}{0.9674\mu_0^{3.833}} - 1\right] + 482$$

Figure 20: Principle of the design method with critical temperature

In practical fire design, the critical temperature method may be applied according to following steps (see figure 21):

- First of all, it is necessary to determine the effect of action under fire situation $E_{fi,d}$
- Secondly, the design resistance R_d or the design action E_d should be calculated
- Thirdly, the corresponding load level $\eta_{fi,t}$ may be obtained using $\eta_{fi,t} = E_{fi,d}/R_d$
- Then, the utilisation level μ_0 can be easily determined with $\mu_0 = \eta_{fi,t}\, \gamma_{M,fi}/\gamma_M$
- Finally the critical temperature θ_{cr} may be calculated directly with

$\theta_{cr} = 39.19\ell n\left[\frac{1}{0,9674\,\mu_0^{3,833}} - 1\right] + 482$ or with a small iterative application procedure (limited to two iterations) of this relation

Particular attention should be paid here to the calculation of utilisation level μ_0 from load level $\eta_{fi,t}$. The difference between them is that the utilisation level μ_0 is determined with respect to the fire resistance at time 0 $R_{fi,d,o}$, so at room temperature but with safety factor $\gamma_{M,fi}$ in fire situation; on the contrary, the load level $\eta_{fi,t}$ is determined using R_d, the ultimate resistance in room temperature design so with corresponding safety factor γ_M which is different from $\gamma_{M,fi}$ (see figure 21). Consequently, there is $R_{fi,d,0} = R_d\, \gamma_M/\gamma_{M,fi}$ from which:

$$\mu_0 = \frac{E_{d,fi}}{R_{d,fi,0}} = \frac{E_{d,fi}}{R_d\, \gamma_M/\gamma_{M,fi}} = \frac{E_{d,fi}}{R_d}\frac{\gamma_{M,fi}}{\gamma_M} = \eta_{fi,t}\frac{\gamma_{M,fi}}{\gamma_M}$$

One important remark is that the utilisation level μ_0 is generally lower than load level $\eta_{fi,t}$ since γ_M is usually higher than $\gamma_{M,fi}$.

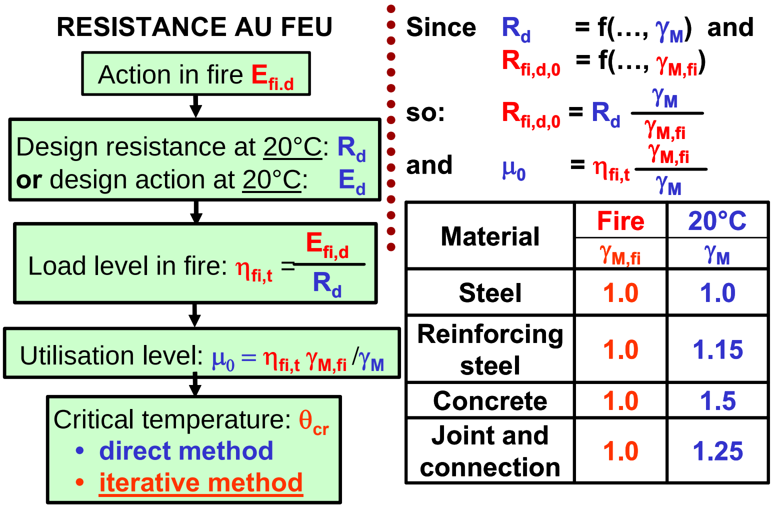


Figure 21 How to apply critical temperature method in fire design

One has certainly noticed in previous figure (see figure 21) that the critical temperature θ_{cr} should be obtained by an iterative rather than a direct calculation. How can this situation occur? Let's have a look of a steel column exposed to fire (see figure 22).

- If the column is short enough so that its buckling will not be present, its strength at elevated temperatures can be simply calculated by $N_{b,fi,t,Rd} = Ak_{y,\theta max} f_y / \gamma_{M,fi}$. In this case, the strength of the column versus temperature will depend only on the strength reduction factor $k_{y,\theta}$ since all other values are fixed parameters.
- Otherwise, if the column is slender so that it will be subjected to buckling failure at elevated temperatures, its strength at elevated temperatures should be calculated by $N_{b,fi,t,Rd} = \chi(\lambda_\theta) Ak_{y,\theta max} f_y / \gamma_{M,fi}$. Under this circumstance, the strength of the column versus temperature will depend on both the strength reduction factor $k_{y,\theta}$ and the relative slenderness under fire situation $\bar{\lambda}_\theta$ which varies as a function of not only strength, that is $k_{y,\theta}$ but also stiffness, namely $k_{E,\theta}$ because there is $\bar{\lambda}_\theta = \bar{\lambda}\,[k_{y,\theta} / k_{E,\theta}]^{0,5}$. As a consequence, it is no longer possible to obtain in a direct calculation the critical temperature θ_{cr} which depends only on $k_{y,\theta}$ and a simple iterative procedure (maximum two iterations) is needed to find the accurate θ_{cr} in case of instability problem.

The iterative procedure explained above seems to be troublesome to apply the critical temperature method. In order to avoid it, it is possible to take a fixed and safe value for $[k_{y,\theta} / k_{E,\theta}]^{0,5}$ so that $\bar{\lambda}_\theta = \bar{\lambda}\,[k_{y,\theta} / k_{E,\theta}]^{0,5}$ will no longer vary with temperature and the direct calculation of critical temperature becomes applicable even in case of instability phenomenon.

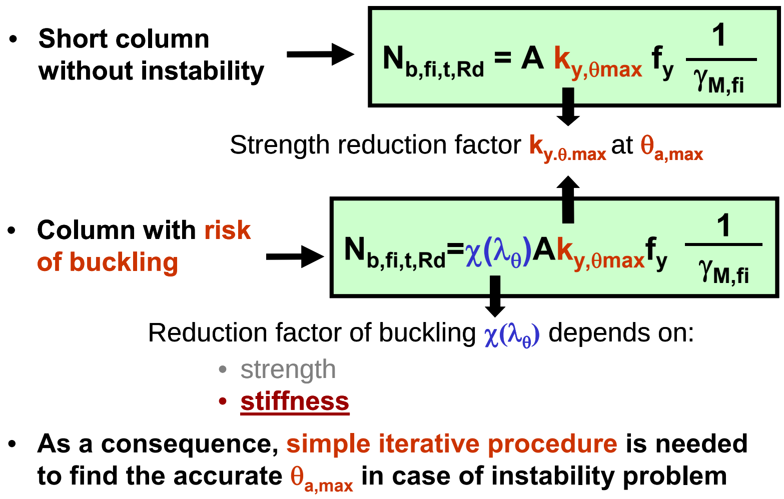


Figure 22 What is the reason to use both direct and iterative calculation to obtain the critical temperature

3.4 *Advanced calculation models*

As far as advanced calculation models are concerned, in principle, they can be applied for any type of structural member analysis in fire design. However, following features have to be considered:

- Advanced calculation methods for mechanical response should be based on the acknowledged principles and assumptions of the theory of structural mechanics, taking into account the changes of mechanical properties with temperature
- Any potential failure modes not covered by the advanced calculation method (including local buckling and failure in shear) should be eliminated by appropriate means. For example, in case of numerical analysis using beam elements
- Advanced calculation methods may be used in association with any heating curve, provided that the material properties are known for the relevant temperature range
- The effects of thermally induced strains and stresses both due to temperature rise and due to temperature differentials, should be considered
- The model for mechanical response should also take account of:
  - the combined effects of mechanical actions, geometrical imperfections and thermal actions
  - the temperature dependent mechanical properties of the material, see section 3
  - geometrical non-linear effects
  - the effects of non-linear material properties, including the unfavourable effects of loading and unloading on the structural stiffness

One typical application example of advanced calculation model is given in figure 23 which concerns a cellular steel beam exposed to standard fire. The necessity to use the advanced calculation model is due to the fact that none of all existing simple rules of Eurocodes covers this kind of member. The only way for the time being to solve the problem is to resort to advanced calculation models. However, before applying the advanced calculation models, it is necessary to validate them against relevant fire tests not only on global behaviour (deflection, failure time, etc) but also the detailed failure mode of the member during fire exposure. One can find from the example given in figure 23 that all these conditions may be fulfilled quite easily if appropriate advanced numerical models are adopted.

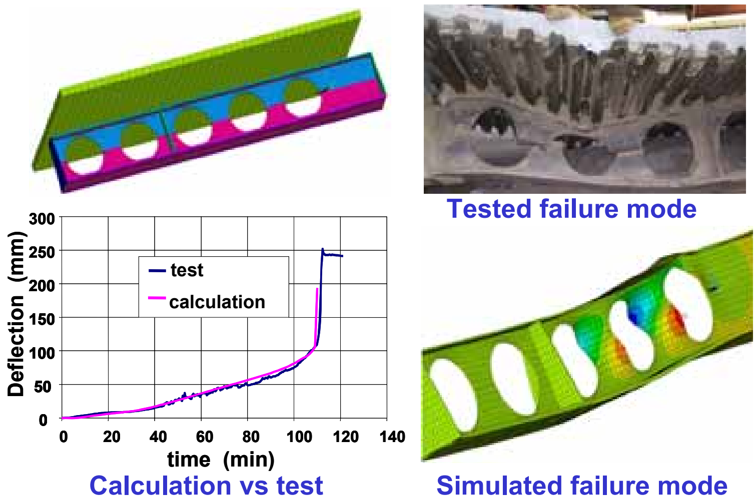


Figure 23 Application example of advanced calculation models for fire design (cellular beam)

4 PRINCIPLE DESCRIPTION OF GLOBAL STRUCTURAL ANALYSIS

4.1 *General application rules of fire design by global structural analysis*

The global structural analysis is more and more employed in model fire safety engineering. As a consequence, Eurocodes have provided precise rules of how to realise this type of analysis. Regarding the analysis of mechanical response using this approach, following features should be taken into account:

- First of all, global structural analysis needs in most cases to use advanced calculation models
- It is important to choose an appropriate structural modelling strategy (size, type, etc)
- The existing boundary conditions should be rightly represented
- The loading condition of modelled structure must correspond to that for fire situation
- Material models used in numerical modelling should be representative of real material behaviour at elevated temperatures
- In case of modelling part of a structure, the restrained condition provided by no-modelled parts of the structure should be taken into consideration in appropriate way
- It is necessary to provide a deep analysis of numerical results and from which a detailed check of failure criteria must be realised
- A review of features which are not dealt with in direct analysis shall be made in order to have a consistency between numerical model and constructional details

All above features will be explained in detail in following figures for a real application example of global structural analysis in a fire safety engineering project.

4.2 *Application requirement of advanced calculation model in global structural analysis of steel and composite structures*

For steel and composite structures, the application of the global structural analysis needs to pay attention to following points:

- Regarding material models, one must think of:
  - strain composition with several strain components at elevated temperatures
  - kinematical material model for temperature evolution
  - strength of certain material such as concrete during cooling phase

- The transient heating regime of structures during fire requires to use step by step iterative solution procedure rather than a steady state analysis for a given time instant
- The existing boundary conditions should be rightly represented
- The loading condition of modelled structure must correspond to that for fire situation
- Material models used in numerical modelling should be representative of real material behaviour at elevated temperatures
- When doing advanced calculation for fire design of steel and composite structures, it must be always careful about certain specific features which in general are not taken into account in direct modelling, such as the reinforcing steel rupture due to excessive elongation, cracking and crushing of concrete, joint resistance, connection between steel and concrete, etc.

4.2.1 *Strain composition of material model in advanced numerical modelling*

In advanced numerical modelling for global structural analysis of steel and composite structures, it has to be kept in mind that the strain of any element exposed to fire is composed of several components and they may be expressed explicitly using following relation: (see figure 24)

$$\varepsilon_t = \varepsilon_{th} + (\varepsilon_\sigma + \varepsilon_c + \varepsilon_{tr}) + \varepsilon_r$$

where:

ε_t is the total strain

ε_{th} is the strain due to thermal elongation

ε_σ is the strain due to stress

ε_c is the strain due to creep effect at elevated temperatures

ε_{tr} is the strain due to transient and non uniform heating regime for concrete

ε_r is the strain due to residual stress often present in steel

In Eurocodes, the creep strain as well as the transient strain is all considered to be included implicitly in stress-strain relationships of corresponding material at elevated temperatures. In addition, the residual stress is in general also neglected except for some special structural analysis.

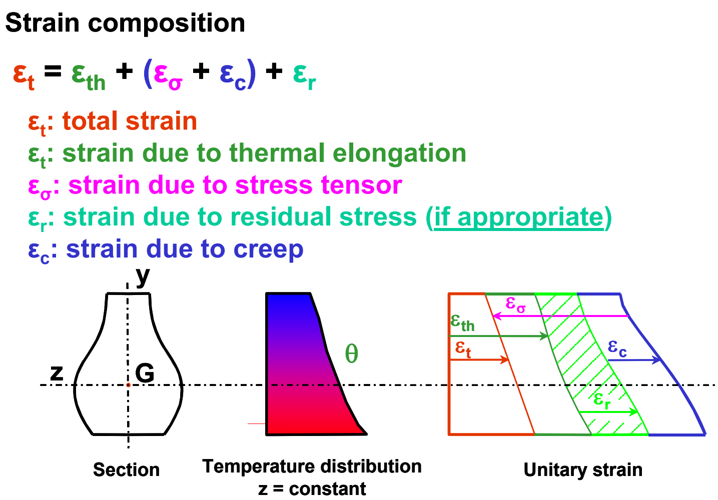


Figure 24 Strain composition of material model in advanced numerical modelling

4.2.2 *Kinematical material model for account of temperature evolution*

Under fire situation, the temperature field of structural members varies versus time. On the other hand, all material mechanical properties are more or less temperature dependant. As a consequence, during a fire, materials of a structure will behave in such a way that their properties change constantly. This type of material behaviour has to be taken into account appropriately in advanced calculation models by so called kinematical material model. As far as the two main materials of steel and composite structures, that is steel and concrete, are concerned, they are two very different materials for which different kinematical rules should be applied (see figure 25).

For steel, the shift from one stress-strain curve to another one due to the change of temperature shall be made by staying at a constant plastic strain value between two temperature levels. This shift rule remains available under any stress state of steel (tension or compression).

For concrete, it is much more complicated since it is a material having different behaviour in tension and in compression. Therefore, one can use different shift rules based on if the material is in tension or in compression (see figure 25).

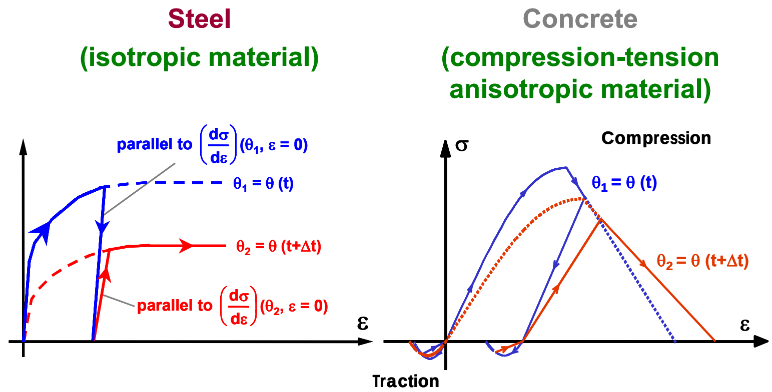


Figure 25 Kinematical material model for account of temperature evolution

Normally, these material models are already implementaed all relevant advanced calculation models for fire safety engineering application. However, it is important for applicators to know how to use these material models in their practical application.

4.2.3 *Principle of step by step iterative solution procedure in advanced numerical calculation*

In general, the structural analysis under fire situation is based on ultimate limit state analysis which means to establish the equilibrium of the structure between its resistance and applied loading for various heating states. However, important displacement of the structure will occur inevitably due to both material softening and thermal expansion leading to large material plastification. Therefore, advanced fire analysis is no longer linear elastic but non-linear elasto-plastic calculation in which both strength and stiffness behave non-linearly. From mathematical point of view, the solution of such analysis can not be obtained directly and has to use following specific procedure (see figure 26):

- Step by step analysis in order to get the equilibrium state of the structure under various instants, so different temperature fields
- Within each time step, an iterative solution procedure is necessary to find out the equilibrium state of the structure behaving in elasto-plastic way.

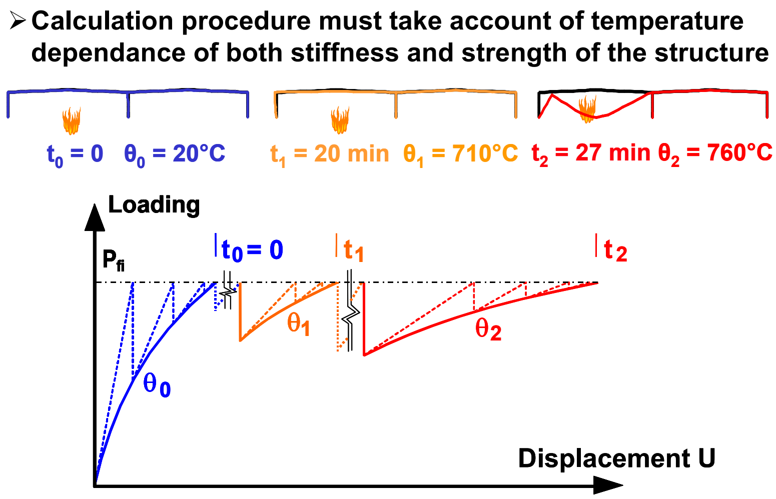


Figure 26 Principle of step by step iterative solution procedure in advanced numerical calculation

4.2.4 *Concrete mechanical behaviour during cooling phase*

Another specific point to be noted in the application of advanced calculation models for steel and composite structures under natural fire conditions is the material behaviour during cooling phase. It is well known that for commonly used steel grades, they are considered as a reversible material regarding temperature effect concerning the mechanical properties which mean that once heated up and cooling down, they will recover their initial mechanical properties. However, this positive phenomenon is no longer true with concrete whose composition will be totally modified if heated up to certain temperature level. After cooling down, it can not recover its initial strength at all. Moreover, its strength could be even worse than that at maximum heating state. As a consequence, EN1994-1-2 has defined special rule to represent this phenomenon (see figure 27). According to it, if concrete is heated to over 300 °C, once cooling down to 20°C, its residual strength will be decreased by another 10% in addition to strength reduction at its maximum heating state.

This behaviour is quite important since it means that a structure with concrete could go to collapse during the cooling phase of a fire.

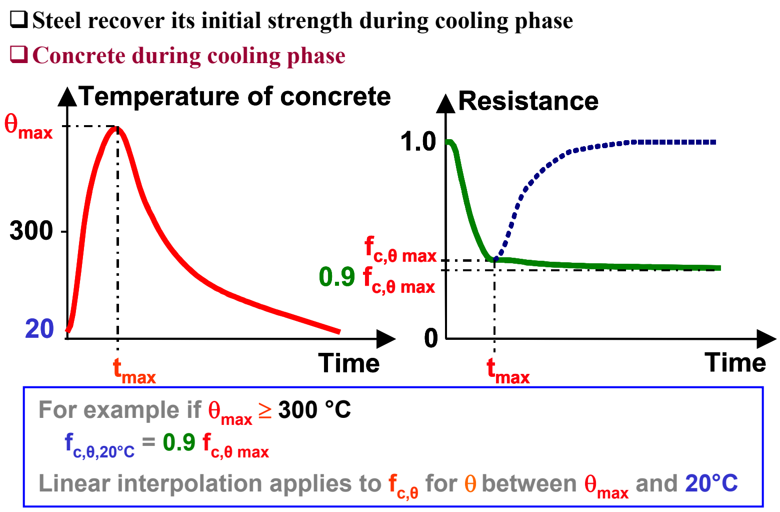


Figure 27 Concrete mechanical behaviour during cooling phase

4.3 *Application example of global structural analysis of steel and composite structures*

4.3.1 *Description of the structure to be investigated*

After the explanation about the application requirement of advanced calculation model in global structural analysis of steel and composite structures, it could be more interesting give an example of such application in order to get a better understanding of it. The chosen example (see figure 28) corresponds to a two-level steel and concrete composite structure composed of composite floor system (steel beams connected with composite slab) and steel columns. The main dimensions of the structure are the following:

- Span of secondary beam: 15 m
- Span of primary beam: 10 m
- Span of composite slab: 3.33 m
- Height of first level: 4.2 m
- Height of second level: 3.2 m

Under fire situation, one of two-level floor will be heated locally by a natural fire source occupying an area of 5 m x 12 m, that is 60 m².

The fire design will be carried out by using a natural fire rather than a standard fire heating regime.

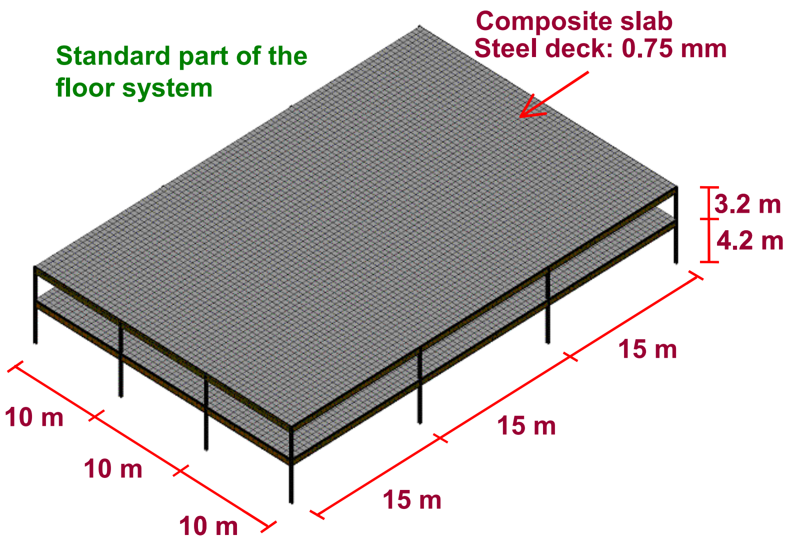


Figure 28 Example of a steel and composite floor structure

As far as this example is concerned, here are explained only the features related to mechanical response analysis of the structure. Other features, such as fire scenario, fire development and thermal response of the structure are deliberately neglected.

4.3.2 *Choice of modelled structure and modelling details*

Coming back to the mechanical analysis of this structure exposed to localised natural fire, two ways of advanced calculation modelling are possible to deal with this structure, the first one corresponding to 2D composite frame model and the second one using more complicated 3D composite floor model. There is a necessity to make a choice between above two advanced models. Before making a proposal, let us first of all have a review of the advantages and disadvantages of the two approaches:

- 2D composite frame model with only beam elements:
  - load redistribution along composite beam is possible
  - membrane effect of composite slab between parallel beams is not taken into account

- several numerical simulations are necessary to deal with one fire scenario
- computation cost for each numerical simulation is low so high efficiency

- 3D composite floor model with shell, beam and link elements:
  - membrane effect over whole composite floor is taken into account
  - load redistribution becomes possible with help of shell elements
  - one numerical simulation is enough for one fire scenario
  - computation cost is high because of important number of elements used in the modelling

Comparing above two modelling strategy, it can be found out that 2D modelling is more efficient but certain important mechanical advantages of the composite floor exposed to localised heating will not be taken into account which will penalise the fire performance of the structure and lead to either a heavier steel structure or a fire protection of it. As a consequence, this consideration has given a definitive preferance to 3D modelling.

However, the application of such complicated calculation model needs to be careful regarding their validity against reality. For example, the 3D composite floor model mentionned above has been fully validated through different natural fire tests performed in recent ECSC research projects with full scale natural fire tests.

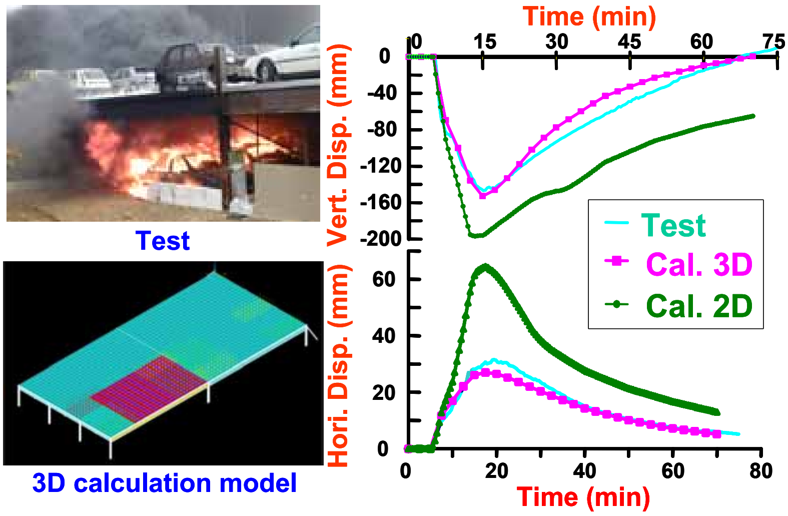


Figure 29 Validity of 3D composite floor model

A typical validation case is the natural fire tests on open car parks (see figure 29), one can find easily that the 3D model gives better and more realistic results than 2D model.Once decided to use 3D modelling strategy, it is then necessary to think to what extent the 3D modelling should be because it is impossible to model the whole composite structure in 3D modelling due to too high computation cost (up to weeks or months using ordinary computers).

However, the fact to have a locally heated floor (see figure 30) gives the possibility of using a reduced area in the 3D model which corresponds exactly to one of three structural analysis procedures proposed by Eurocodes for fire design, namely analysis of part of the structure. In this case, let's take the floor area as small as possible in the numerical modelling which leads to a largely reduced modelled part of the structure (one level occupying an area of 15x20=300 m²
from a two-level structure occupying a ground area of 45x30=1350 m²).

In addition to above choice of part of the structure, it is also worthwhile to give a word about the modelling details adopted. In fact, the composite floor is represented by following finite elements:

- shell element for solid part of composite slab as well as reinforcing steel grid
- beam-column element for steel members, steel sheet and ribs of composite slab
- rigid link element for full connection between steel beams and composite slab

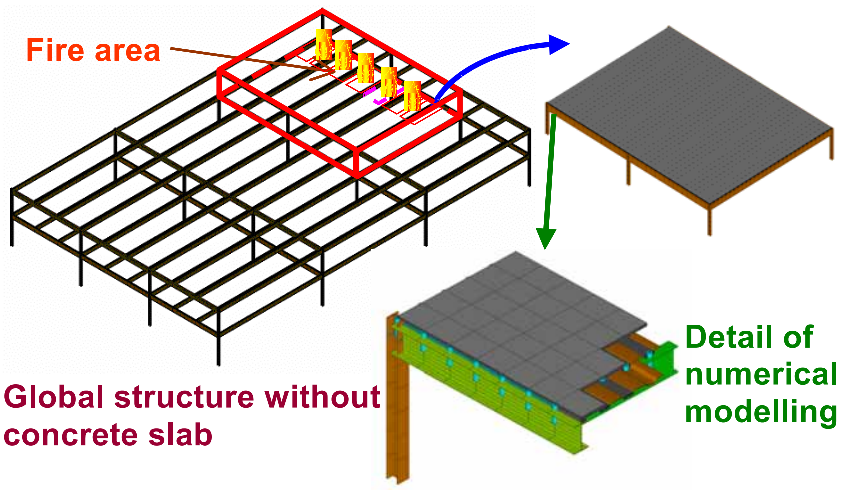


Figure 30 Choice of part of the structure in advanced calculation model

4.3.3 *Loading and boundary conditions*

After the choice of part of the structure used for fire design, it is necessary to consider two following aspects:

- loading condition of the structure
- boundary condition of modelled part of the structure

In room temperature design, the floor is supposed to be submitted to four types of loads:

- dead load (self weight of the structure, lighting system, etc): **G**
- live load: **Q**
- wind load: **W**
- snow load: **S**

Under fire situation, one should consider different load combination to find out the most unfavorable one. In case of this structure, since the lateral stability is ensured by separate bracing system so the wind effect to the floor may be neglected. Then, there are following possibilities to combine dead load **G**, live load **Q** and snow load **S**:

- $G + \Psi_{1,1}Q + \Psi_{2,1}S = G + 0.7Q + 0.0S = G + 0.7Q$
- $G + \Psi_{1,1}S + \Psi_{2,1}Q = G + 0.6Q + 0.2S$

Among above load combinations, it has been found that the most unfavorable one is the first combination because it leads to more important total load value.

Regarding the boundary conditions, the modelled structure is not subjected to any initial boundary conditions. Nevertheless, because it is only a part of the structure, there are some restrained conditions from unmodelled part of the structure to be taken into account. These restrained conditions may be represented by equivalent boundary conditions, such as (see figure 31):

- fully fixed column bases due to continuous column condition and lower floor staying cold
- rotation and lateral displacement restrained slab because of the continuity condition of the slab

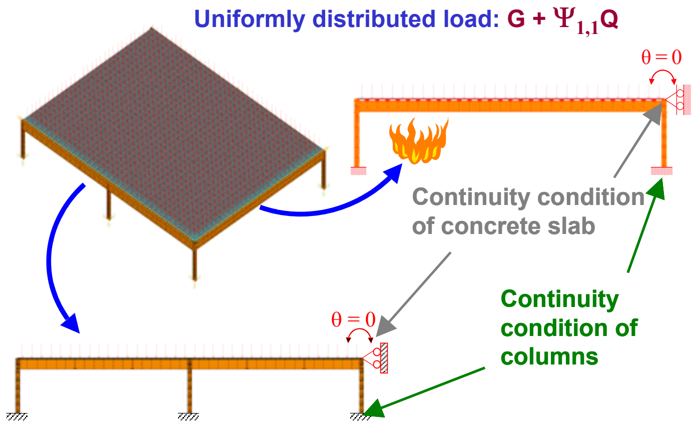


Figure 31 Application of mechanical loading as well as boundary condition to modelled part of the structure

4.3.4 *Investigation of numerical results*

From the modelled part of the structure, the numerical simulation may be performed to investigate its fire performance. As far as the example is concerned, the obtained numerical results are shown in figure 32 in which the deformation states of the floor at two different fire instants are illustrated. Because the floor is subjected to a localised natural fire heating, one can observe easily the consequence of the fire development on the displacement behaviour of the floor whose maximum vertical deflection increases from 140 mm at 20 minutes of fire to 310 minutes at 40 minutes of fire.

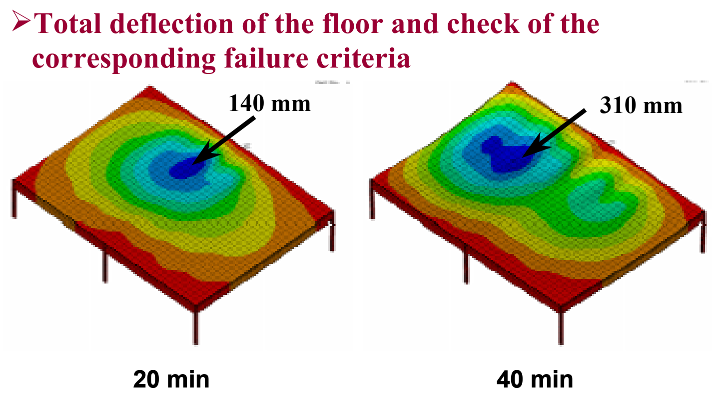


Figure 32 Analysis of numerical results of global mechanical behaviour of modelled part of the structure

Then at 60 minutes of fire, the maximum deflection of the floor decreases to 230 mm but the deformed area increases because of the fire development. The decrease of deflection is caused by the fact that the fire has passed its maximum heating phase and entered the cooling phase (see figure 33).

Concerning the maximum deflection of steel beams, it can be found that it is only 280 mm for secondary beams and 110 mm for primary beams which is far away from the defined failure criteria limiting the maximum deflection to 20^{th} of the span. From this point of view, the performance of the floor can be considered as fully satisfying under corresponding fire scenario.

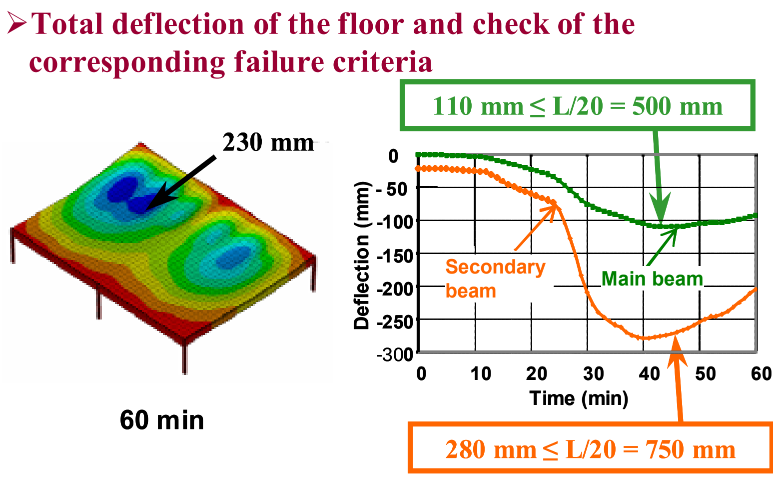


Figure 33 Check of failure criteria related to deflection

Another failure criterion to be investigated for above modelled structure is the elongation of reinforcing steel grid in the composite slab (see figure 34). It has been considered that the maximum elongation of reinforcing steel shall not exceed 5%, which, in fact, corresponds to the minimum value of elongation capacity of all types of reinforcing steel specified in EN1992-1-2 (fire part of concrete structure). Moreover, these failure criteria have been validated in two of ECSC projects through the numerical modelling of fire tests in real buildings (see references).

For actual example, the maximum elongation of reinforcing steel grid obtained in numerical simulation is 1.4% so much less than 5%. Therefore, this failure criterion is also fully satisfied with adopted composite floor.

Above global structural approaches with advanced numerical models have been largely used in several ECSC projects to analyse the fire tests carried out on real scale steel and concrete composite buildings. It has been shown that the agreement between this type of advanced numerical models and experimental results is fully satisfactory (see references).

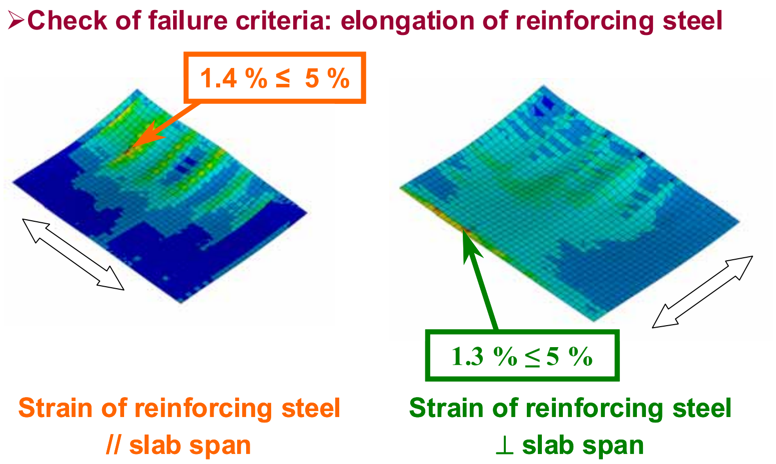


Figure 34 Check of failure criteria related to elongation of reinforcing steel

4.3.5 *Requirement of construction details to have consistent and available numerical analysis*

In parallel to numerical analysis, it is extremely important to impose specific construction details in order to be consistent with assumptions used in numerical models. For previous composite structure, following construction details have been required (see figure 35):

- mechanical link with help of additional reinforcing steel bars between edge as well as corner columns and composite slab to strengthen the fire performance of edge part of the floor
- small gap between the lower flange of beams and columns as well as between lower flanges of secondary and primary beams in order to benefit hogging moment resistance in case of fire
- simple beam to beam and beam to column joints may be used because of above two requirements
- full shear connection between steel beams and composite slab

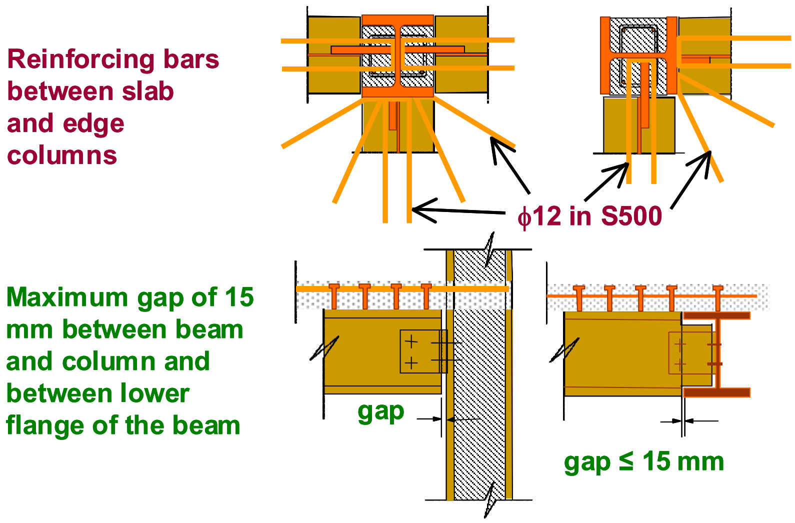


Figure 35 Consistency between numerical models and construction details

4.3.6 *Real building example designed with help of global structural analysis in fire assessment*

A typical French example is given in figure 36 showing a real construction built based on a fire design through global structural analysis with advanced calculation models under natural fire conditions.

For this building, several fire scenarios have been applied and for each scenario, a detailed advanced calculation model is established. For all fire scenarios, the failure criterion related to deflection of steel beams and elongation of reinforcing steel in composite slab are checked carefully.

This application of fire safety engineering has led to the construction of first so large building with fully bare steel structure in France.

Figure 36 Real example of fire design with global structural analysis under natural fire condition

5 SPECIFIC CONSIDERATION IN FIRE DESIGN OF STEEL AND COMPOSITE STRUCTURES

In the presentation of above application example, it has been mentioned that in fire design of steel and composite structures, specific consideration need to be taken regarding the construction details for:

- joints
- connections between concrete and steel

In fact, one can easily understand that the fire design based on global structural analysis assume that the integrity of the structure must be guaranteed. If it is not the case, the fundamental of this type of analysis will be no longer true. In addition, under any circumstance, it is not acceptable to have any kind of inadequate global collapse of the structure due to the failure of connection elements.

Another aspect related to joint which should be kept in mind is the possible failure of it during the cooling phase. This feature is very important not only for global structural analysis under natural fire which may have one part of the structure still heated up and another part of it already entering the cooling phase but also for standard fire design of steel and composite structures which must consider the real fire performance in any way.

In EN1993-1-2 (fire part of Eurocode 3) and EN1994-1-2 (fire part of Eurocode 4), either simple calculation model or construction details is recommended for fire design of joint, connections. One of typical examples concerns the joint detail of composite structure between beam and column (see figure 37). In fact, it is proposed to use a very small gap between the lower flange of steel beam and steel column. As a consequence, under room temperature condition, the joint is considered as a simple bolted joint because the deflection of the beam is very limited. However, under fire situation, due to both temperature bowing effect and loss of strength it will undergo large deflection leading to an important rotation at support. In case of a small gap between the lower flange and steel column, the lower flange of the beam will easily enter in contact with the column creating therefore together with the reinforcing steel bars in the concrete slab a hogging moment resistance of the beam. This additional hogging moment resistance will increase largely the fire resistance of the beam.

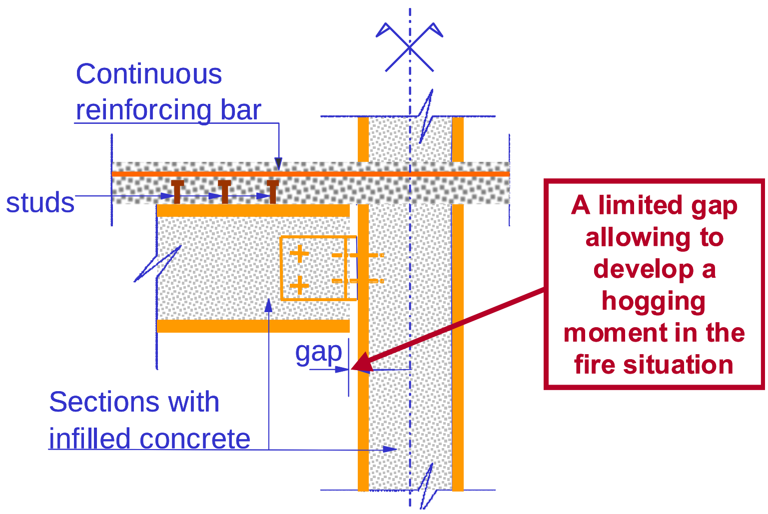


Figure 37 Example of constructional details to get hogging moment resistance in fire situation according to EN1994-1-2

Besides the connection between steel beams and concrete slab in case of composite structures, another typical example is the connection between steel and concrete in case of partially concrete encased composite beams. In order to have an enough connection resistance so that the additional steel bars are capable to work together with steel profile, EN1994-1-2 has recommended the construction details shown in figure 38. The main purpose of these construction details is not only to create a mechanical connection between different components of the beam but also to provide a protection system against concrete spalling, a very negative behaviour of concrete during fire exposure which in this case could lead to the direct exposure of reinforcing steel to fire.

A lot of other construction details exist. Under any circumstance, in the fire design, engineer should pay particular attention to them in order to get the best fire safety solution of steel and composite structures.

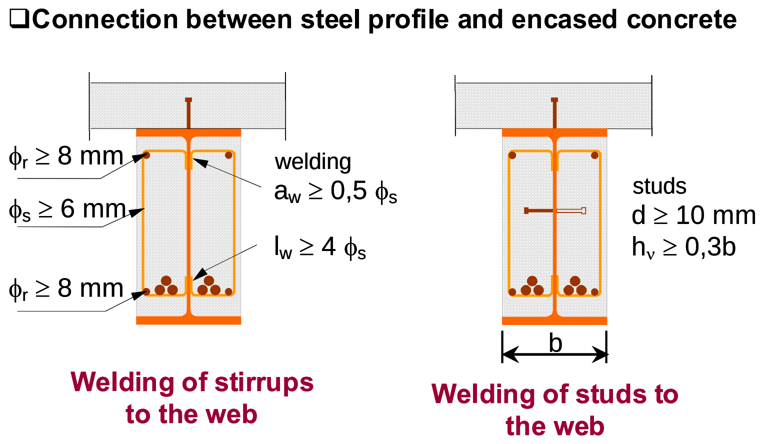


Figure 38 Example of constructional details to have enough connection resistance between steel and concrete in fire situation according to EN1994-1-2

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