4.3.4 Composite beams

4.3.4.1 Structural Behaviour

4.3.4.1.1 General

(1)P Composite beams shall be checked for:

- resistance of critical cross-sections in accordance with 6.1.1(P) of EN 1994-1-1 to bending (4.3.4.1.2);
- vertical shear (4.3.4.1.3);
- resistance to longitudinal shear (4.3.4.1.5).

NOTE: Guidance on critical cross-sections is given in 6.1.1(4)P of EN1994-1-1.

(2) Where in the fire situation, test evidence (see EN 1365 Part 3) of composite action between the floor slab and the steel beam is available, beams which for normal conditions are assumed to be non-composite may be assumed to be composite in fire conditions.

(3) The temperature distribution over the cross-section may be determined from test, advanced calculation models (4.4.2) or for composite beams comprising steel beams with no concrete encasement, from the simple calculation model of 4.3.4.2.2.

4.3.4.1.2 Bending resistance of cross-sections of beams

(1) The design bending resistance may be determined by plastic theory for any class of cross sections except for class 4.

(2) For simply supported beams, the steel flange in compression may be treated, independent of its class, as class 1, provided it is connected to the concrete slab by shear connectors placed in accordance to 6.6.5.5 of EN1994-1-1.

(3) For class 4 steel cross-sections, refer to 4.2.3.6 of EN 1993-1-2.

4.3.4.1.3 Vertical shear resistance of cross-sections of beams

(1)P The resistance to vertical shear shall be taken as the resistance of the structural steel section (see 4.2.3.3(6) and 4.2.3.4(4) of EN 1993-1-2), unless the value of a contribution from the concrete part of the beam has been established by tests.

NOTE: For the calculation of the vertical shear resistance of the structural steel section, a method is given in E.4 of Annex E.

(2) For simply supported beams with webs encased in concrete no check is required provided for normal design the web was assumed to resist all vertical shear.

4.3.4.1.4 Combined bending and vertical shear

(1) For partially encased beams under hogging bending, the web may resist the vertical shear even if this web does not contribute to the moment resistance.

NOTE 1: For partially encased beams under hogging bending, a method is given in F.2(7) of Annex F.

NOTE 2: For composite beams comprising steel beams with no concrete encasement, a method is given in E.2 and E.4 of Annex E.

4.3.4.1.5 Longitudinal Shear Resistance

(1)P The total design longitudinal shear shall be determined in a manner consistent with the design bending resistance, taking account of the difference in the normal force in concrete and in structural steel over a critical length.

(2) In case of design by partial shear connection in the fire situation, the variation of longitudinal shear forces in function of the heating should be considered.

(3) The total design longitudinal shear over the critical length in the area of sagging bending is calculated from the compression force in the slab given by:

$$F_c = \alpha_{slab} \sum_{j=1}^{m} A_j k_{c,\theta,j} \left(\frac{f_{c,j}}{\gamma_{M,fi,c}} \right) \tag{4.4}$$

or by the tension force in the steel profile given by:

$$F_a = \sum_{i=1}^{n} A_i k_{y,\theta,i} \left(\frac{f_{y,i}}{\gamma_{M,fi,a}} \right) \quad \text{whichever is smaller.} \tag{4.5}$$

NOTE: For the calculation of the longitudinal shear in the area of hogging bending, a method is given in E.2 of Annex E.

(4)P Adequate transverse reinforcement shall be provided to distribute the longitudinal shear according to 6.6.6.2 of EN 1994-1-1.

4.3.4.2 Composite beams comprising steel beams with no concrete encasement

4.3.4.2.1 General

(1) The following assessment of the fire resistance of a composite beam comprising a steel beam with no concrete encasement is applicable to simply supported elements and continuous beams (see Figure 1.2).

4.3.4.2.2 Heating of the cross-section

Steel beam

(1) When calculating the temperature distribution of the steel section, the cross section may be divided into various parts according to Figure 4.3.

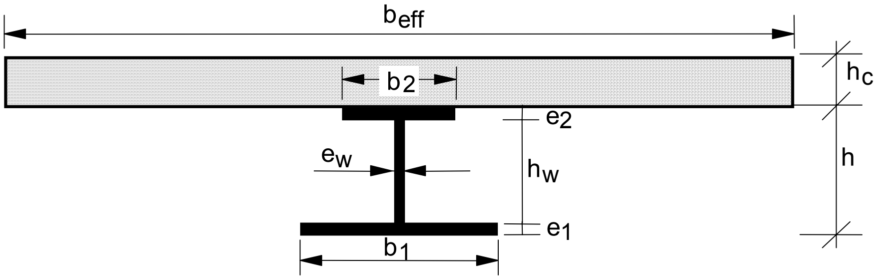


Figure 4.3: Elements of a cross-section

(2) It is assumed that no heat transfer takes place between these different parts nor between the upper flange and the concrete slab.

(3) The increase of temperature $\Delta\theta_{a,t}$ of the various parts of an **unprotected steel beam** during the time interval Δt may be determined from:

$$\Delta\theta_{a.t} = k_{shadow}\left(\frac{1}{c_a \rho_a}\right)\left(\frac{A_i}{V_i}\right)\dot{h}_{net}\,\Delta t \qquad [°C] \qquad (4.6)$$

where

k_{shadow} is a correction factor for the shadow effect (see(4))

c_a is the specific heat of steel in accordance with (4) of 3.3.1 [J/kgK]

ρ_a is the density of steel in accordance with (1)P of 3.4 [kg/m^3]

A_i is the exposed surface area of the part i of the steel cross-section per unit length [m²/m]

A_i/V_i is the section factor [m^{-1}] of the part i of the steel cross-section

V_i is the volume of the part i of the steel cross section per unit length [m^3/m]

$\dot{h}_{net}$ is the design value of the net heat flux per unit area in accordance with 3.1 of EN 1991-1-2

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} \qquad [W/m^2]$$

$$\dot{h}_{net,c} = \alpha_c\left(\theta_t - \theta_{a,t}\right) \qquad [W/m^2]$$

$$\dot{h}_{net,r} = \varepsilon_m\,\varepsilon_f\left(5{,}67\,.\,10^{-8}\right)\left[\left(\theta_t + 273\right)^4 - \left(\theta_{a,t} + 273\right)^4\right] \qquad [W/m^2]$$

ε_m as defined in 2.2 (2)

ε_f is the emissivity of the fire according to 3.1 (6) of EN 1991-1-2

θ_t is the ambient gas temperature at time t [°C]

$\theta_{a,t}$ is the steel temperature at time t [°C] supposed to be uniform in each part of the steel cross-section

Δt is the time interval [sec]

(4) The shadow effect may be determined from:

$$k_{shadow} = 0{,}9\left(\frac{e_1 + e_2 + 1/2 \cdot b_1 + \sqrt{h_w^{\;2} + 1/4 \cdot (b_1 - b_2)^2}}{h_w + b_1 + 1/2 \cdot b_2 + e_1 + e_2 - e_w}\right) \qquad (4.7)$$

with $e_1, b_1, e_w, h_w, e_2, b_2$ and cross sectional dimensions according to Figure 4.3.

NOTE: The above equation giving the shadow effect (k_{shadow}), and its use in (3), is an approximation, based on the results of a large amount of systematic calculations; for more refined calculation models, the configuration factor concept as presented in 3.1 and Annex G of EN1991-1-2 should be applied.

(5) The value of Δt should not be taken as more than 5 seconds for (3).

(6) The increase of temperature $\Delta\theta_{a,t}$ of various parts of an **insulated steel beam** during the time interval Δt may be obtained from:

$$\Delta\theta_{a,t} = \left[\left(\frac{\lambda_p / d_p}{c_a \rho_a} \right) \left(\frac{A_{p,i}}{V_i} \right) \left(\frac{1}{1 + w/3} \right) \left(\theta_t - \theta_{a,t} \right) \Delta t \right] - \left[\left(e^{w/10} - 1 \right) \Delta\theta_t \right] \tag{4.8}$$

with $w = \left(\frac{c_p \rho_p}{c_a \rho_a} \right) d_p \left(\frac{A_{p,i}}{V_i} \right)$ and

where:

λ_p	is the thermal conductivity of the fire protection material as specified in (1)P of 3.3.4	[W/mK]
d_p	is the thickness of the fire protection material	[m]
$A_{p,i}$	is the area of the inner surface of the fire protection material per unit length of the part i of the steel member	[m²/m]
c_p	is the specific heat of the fire protection material as specified in (1)P of 3.3.4	[J/kgK]
ρ_p	is the density of the fire protection material	[kg/m^3]
θ_t	is the ambient gas temperature at time t	[°C]
$\Delta\theta_t$	is the increase of the ambient gas temperature [°C] during the time interval Δt	

(7) Any negative temperature increase $\Delta\theta_{a,t}$ obtained by (6) should be replaced by zero.

(8) The value of Δt should not be taken as more than 30 seconds for (6).

(9) For non protected members and members with contour protection, the section factor A_i / V_i or $A_{p,i} / V_i$ should be calculated as follows:

for the lower flange:

$$A_i / V_i \text{ or } A_{p,i} / V_i = 2\,(b_1 + e_1)/ b_1\, e_1 \tag{4.9a}$$

for the upper flange, when at least 85% of the upper flange of the steel profile is in contact with the concrete slab or, when any void formed between the upper flange and a profiled steel deck is filled with non-combustible material:

$$A_i / V_i \text{ or } A_{p,i} / V_i = (b_2 + 2\, e_2)/ b_2\, e_2 \tag{4.9b}$$

for the upper flange when used with a composite floor when less than 85% of the upper flange of the steel profile is in contact with the profiled steel deck:

$$A_i / V_i \text{ or } A_{p,i} / V_i = 2\,(b_2 + e_2)/ b_2\, e_2 \tag{4.9c}$$

(10) If the beam depth h does not exceed 500 mm, the temperature of the web may be taken as equal to that of the lower flange.

(11) For members with box-protection, a uniform temperature may be assumed over the height of the profile when using (6) together with A_p / V .

where:

A_p is the area of the inner surface of the box protection per unit length of the steel beam [m²/m]

V is the volume of the complete cross-section of the steel beam per unit length [m³/m]

(12) As an alternative to (6), temperatures in a steel section after a given time of fire duration may be obtained from design flow charts determined in conformity with EN 13381 Part 4 and Part 5.

(13) Protection of a steel beam bordered by a concrete floor on top, may be achieved by a horizontal screen below, and its temperature development may be calculated according to 4.2.5.3 of EN 1993-1-2.

Flat concrete or steel deck-concrete slab system

(14) The following rules (15) to (16) may be used for flat concrete slabs or for steel deck-concrete slab systems with re-entrant or trapezoidal steel sheets.

(15) A uniform temperature distribution may be assumed over the effective width b_{eff} of the concrete slab.

NOTE: In order to determine temperatures over the thickness of the concrete slab a method is given in the Table D.5 of Annex D.

(16) For the mechanical analysis it may be assumed, that for concrete temperatures below 250°C, no strength reduction of concrete is considered.

4.3.4.2.3 Structural behaviour - critical temperature model

(1) In using the following critical temperature model, the temperature of the steel section is assumed to be uniform.

(2)P The method is applicable to symmetric sections of a maximum depth h of 500 mm and to a slab depth h_c not less than 120 mm, used in connection with simply supported beams exclusively subject to sagging bending moments.

(3) The critical temperature θ_{cr} may be determined from the load level $\eta_{fi,t}$ applied to the composite section and from the strength of steel at elevated temperatures $f_{ay,\theta cr}$ according to the relationship:

for R30 $$0{,}9\, \eta_{fi,t} = f_{ay,\theta cr} / f_{ay} \tag{4.10a}$$

in any other case $$1{,}0\, \eta_{fi,t} = f_{ay,\theta cr} / f_{ay} \tag{4.10b}$$

where $\eta_{fi,t} = E_{fi,d,t} / R_d$ and $E_{fi,d,t} = \eta_{fi}\, E_d$ according to (7)P of 4.1 and (3) of 2.4.2.

(4) The temperature rise in the steel section may be determined from (3) or (6) of 4.3.4.2.2 using the section factor A_i / V_i or $A_{p\,i} / V_i$ of the lower flange of the steel section.

4.3.4.2.4 Structural behaviour - bending moment resistance model

(1) As an alternative to 4.3.4.2.3 the bending moment resistance may be calculated by the plastic theory, taking into account the variation of material properties with temperature (see 4.3.4.1.2).

(2) The sagging and hogging moment resistances may be calculated taking into account the degree of shear connection.

NOTE: For the calculation of sagging and hogging moment resistances, a method is given in Annex E.

4.3.4.2.5 Verification of shear resistance of stud connectors

(1) The design shear resistance in the fire situation of a welded headed stud should be determined both for solid and steel deck-concrete slab systems in accordance with EN 1994-1-1, except that the partial factor γ_v should be replaced by $\gamma_{M,fi,v}$ and the smaller of the following reduced values is to be used:

$P_{fi,Rd} = 0{,}8 \, . \, k_{u,\theta} \, . \, P_{Rd}$, with P_{Rd} as obtained from equation 6.18 of EN 1994-1-1 or (4.11a)

$P_{fi,Rd} = k_{c,\theta} \, . \, P_{Rd}$, with P_{Rd} as obtained from equation 6.19 of EN 1994-1-1 and (4.11b)

where values of $k_{u,\theta}$ and $k_{c,\theta}$ are taken from Tables 3.2 and 3.3 respectively.

(2) The temperature θ_v [°C] of the stud connectors and θ_c [°C] of the concrete may be taken as 80 % and 40 % respectively of the temperature of the upper flange of the beam.

4.3.4.3 Composite beams comprising steel beams with partial concrete encasement

4.3.4.3.1 General

(1) The bending moment resistance of a partially encased steel beam connected to a concrete slab may be calculated using 4.3.4.1.2 or alternatively using the method given hereafter.

(2) The following assessment of the fire resistance of a composite beam, comprising a steel beam with partial concrete encasement according to Figure 1.5, is applicable to simply supported or continuous beams including cantilever parts.

(3) The following rules apply to composite beams heated from below by the standard temperature-time curve.

(4)P The effect of temperatures on material characteristics is taken into account either by reducing the dimensions of the parts composing the cross section or by multiplying the characteristic mechanical properties of materials by a reduction factor.

NOTE: For the calculation of this reduction factor, a method is given in Annex F

(5)P It is assumed that there is no reduction of the shear resistance of the connectors welded to the upper flange, as long as these connectors are fixed directly to the effective width of that flange.

NOTE: For the evaluation of this effective width, a method is given in F.1 of Annex F

(6) This method may be used to classify composite beams in the standard fire classes R30, R60, R90, R120 or R180.

(7) This method may be used in connection with a slab with profiled steel sheets, if for trapezoidal profiles void fillers are used on top of the beams, if re-entrant profiles are chosen or if (16) of 4.1 is fulfilled.

(8) The slab thickness h_c (see Figure 4.4) should be greater than the minimum slab thickness given in Table 4.8. This table may be used for solid and steel deck-concrete slab systems.

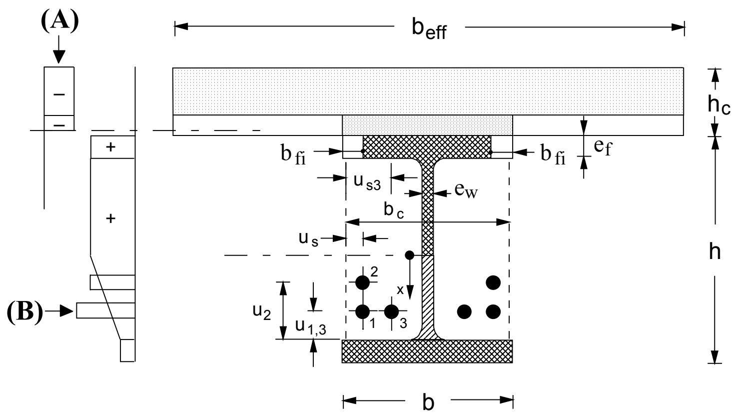
Table 4.8: Minimum slab thickness

Standard Fire Resistance	Minimum Slab Thickness h_c [mm]
R30	60
R60	80
R90	100
R120	120
R180	150

4.3.4.3.2 Structural behaviour

(1) For a simply supported beam, the maximum sagging bending moment produced by loads should be compared to the sagging moment resistance which is calculated according to 4.3.4.3.3.

(2) For the calculation of the sagging moment resistance M_{fi,Rd^+} Figure 4.4 may be considered.



NOTE to Figure 4.4: **(A)** Example of stress distribution in concrete; **(B)** Example of stress distribution in steel

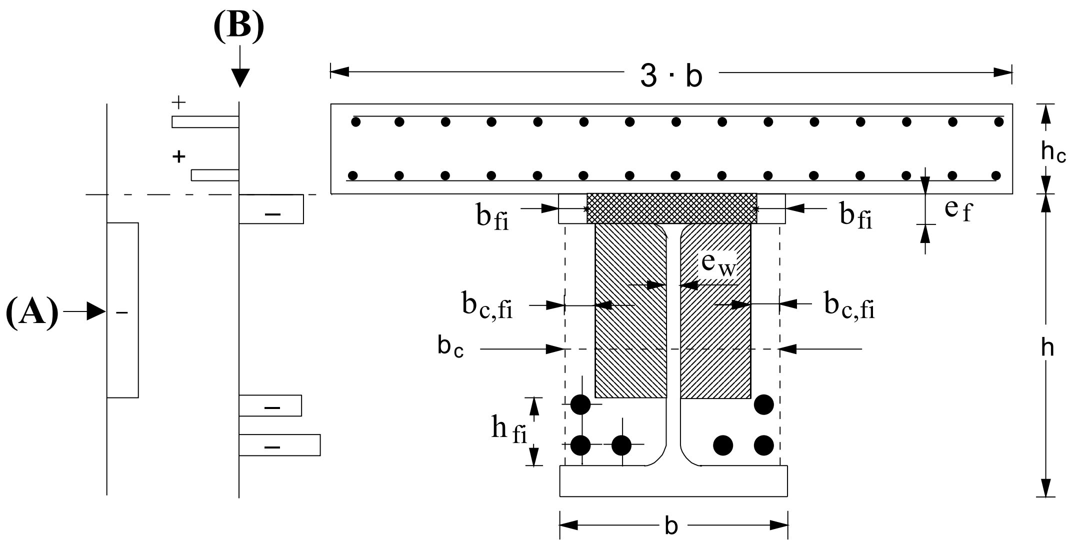
Figure 4.4: Elements of a cross-section for the calculation of the sagging moment resistance

(3)P For a span of a continuous beam, the sagging moment resistance in any critical cross- section and the hogging moment resistance on each support shall be calculated according to 4.3.4.3.3 and 4.3.4.3.4.

(4) For the calculation of the hogging moment resistance M_{fi,Rd^-} Figure 4.5 may be considered.

(5) For the calculation of the moment resistance corresponding to the different fire classes, the following mechanical characteristics may be adopted:

- for the profile, the yield point f_{ay} possibly reduced;
- for the reinforcing bars, the reduced yield point $k_r\ f_{ry}$ or $k_s\ f_{sy}$;
- for the concrete, the compressive cylinder strength f_c .



NOTE to Figure 4.5: (A) Example of stress distribution in concrete;
(B) Example of stress distribution in steel

Figure 4.5: Elements of a cross-section for the calculation of the hogging moment resistance

(6)P The design values of the mechanical characteristics given in (5) are obtained by applying the partial factors given in (1)P of 2.3.

(7) Beams, which are considered as simply supported for normal temperature design, may be considered as continuous in the fire situation if (5) of 5.4.1 is fulfilled.

4.3.4.3.3 Sagging moment resistance $M_{fi,Rd}{}^{+}$

(1) The width b_{eff} of the concrete slab should be equal to the effective width chosen according to 5.4.1.2 of EN 1994-1-1.

(2) In order to calculate the sagging moment resistance, the concrete of the slab in compression, the upper flange of the profile, the web of the profile, the lower flange of the profile and the reinforcing bars should be considered. For each of these parts of the cross section, a corresponding rule may define the effect of the temperature. The concrete in tension of the slab and the concrete between the flanges of the profile should be ignored (see Figure 4.4).

(3) On the basis of the essential equilibrium conditions and on the basis of the plastic theory, the neutral bending axis may be defined and the sagging moment resistance may be calculated.

4.3.4.3.4 Hogging moment resistance $M_{fi,Rd}$ -

(1) The effective width of the concrete slab is reduced to three times the width of the steel profile (see Figure 4.5). This effective width determines the reinforcing bars to be taken into account.

(2) In order to calculate the hogging moment resistance, the reinforcing bars in the concrete slab, the upper flange of the profile except when (4) is applicable, and the concrete in compression between the flanges of the profile should be considered. For each of these parts of the cross-section a corresponding rule may define the effect of the temperature. The concrete in tension of the slab, the web and the lower flange of the profile should be ignored.

NOTE: For the design of the web, regarding vertical shear, a method is given in F.2 of Annex F.

(3) The reinforcing bars situated between the flanges may participate in compression and be considered in the calculation of the hogging moment resistance, provided the corresponding stirrups fulfil the relevant requirements given in EN 1992-1-1, in order to restrain the reinforcing bars against local buckling, and provided either both the steel profile and the reinforcing bars are continuous at the support or (5) of 5.4.1 is applicable.

(4) In the case of a simply supported beam according to (5) of 5.4.1, the upper flange should not be taken into account if it is in tension.

(5) On the basis of the essential equilibrium conditions and on the basis of the plastic theory, the neutral bending axis may be defined and the hogging moment resistance may be calculated.

(6)P The principles of plastic global analysis apply for the combination of sagging and hogging moments if plastic hinges develop at supports.

(7) Composite beams comprising steel beams with partial concrete encasement may be assumed not to fail through lateral torsional buckling in the fire situation.

4.3.4.4 Steel beams with partial concrete encasement

(1) If the partially encased beam supports a concrete slab, without shear connection according to Figure 1.3, the rules given in 4.3.4.3 may be applied by assuming no mechanical resistance of the reinforced concrete slab.

4.3.5 Composite columns

4.3.5.1 Structural behaviour

(1)P The simple calculation models described hereafter shall only be used for columns in braced frames.

NOTE: EN1994-1-1, 6.7.3.1(1), in all cases limits the relative slenderness $\bar{\lambda}$ for normal design, to a maximum of 2.

(2) In simple calculation models the design value in the fire situation, of the resistance of composite columns in axial compression (buckling load) should be obtained from:

$$N_{fi,Rd} = \chi \, N_{fi,pl,Rd} \tag{4.12}$$

where:

χ is the reduction coefficient for buckling curve c of 6.3.1 of EN 1993-1-1 and depending on the relative slenderness $\bar{\lambda}_\theta$,

$N_{fi,pl,Rd}$ is the design value of the plastic resistance to axial compression in the fire situation.

(3) The cross section of a composite column may be divided into various parts. These are denoted "a" for the steel profile, "s" for the reinforcing bars and "c" for the concrete.

(4) The design value of the plastic resistance to axial compression in the fire situation is given by:

$$N_{fi,pl,Rd} = \sum_j \left(A_{a,\theta}\, f_{ay,\theta}\right)/\gamma_{M,fi,a} + \sum_k \left(A_{s,\theta}\, f_{sy,\theta}\right)/\gamma_{M,fi,s} + \sum_m \left(A_{c,\theta}\, f_{c,\theta}\right)/\gamma_{M,fi,c} \tag{4.13}$$

where:

$A_{i,\theta}$ [AC⟩ is the area of each element of the cross-section ($i = a$ or c or s), which may be affected by the fire ⟨AC].

(5) The effective flexural stiffness is calculated as

$$(EI)_{fi,eff} = \sum_j \left(\varphi_{a,\theta} \, E_{a,\theta} \, I_{a,\theta} \right) + \sum_k \left(\varphi_{s,\theta} \, E_{s,\theta} \, I_{s,\theta} \right) + \sum_m \left(\varphi_{c,\theta} \, E_{c,sec,\theta} \, I_{c,\theta} \right) \tag{4.14}$$

where:

$I_{i,\theta}$ is the second moment of area, of the partially reduced part i of the cross-section for bending around the weak or strong axis,

$\varphi_{i,\theta}$ is the reduction coefficient depending on the effect of thermal stresses.

$E_{c,sec,\theta}$ is the characteristic value for the secant modulus of concrete in the fire situation, given by $f_{c,\theta}$ divided by $\varepsilon_{cu,\theta}$ (see Figure 3.2).

NOTE: A method is given in G.6 of Annex G, for the evaluation of the reduction coefficient of partially encased steel sections.

(6) The Euler buckling load or elastic critical load in the fire situation is as follows

$$N_{fi,cr} = \pi^2 \, (EI)_{fi,eff} \, / \, \ell_\theta^2 \tag{4.15}$$

where:

ℓ_θ is the buckling length of the column in the fire situation.

(7) The relative slenderness is given by:

$$\bar{\lambda}_\theta = \sqrt{N_{fi,pl,R} / N_{fi,cr}} \tag{4.16}$$

where

$N_{fi,pl,R}$ is the value of $N_{fi,pl,Rd}$ according to (4) when the factors $\gamma_{M,fi,a}$, $\gamma_{M,fi,s}$ and $\gamma_{M,fi,c}$ are taken as 1,0.

(8) For the determination of the buckling length ℓ_θ of columns, the rules of EN 1994-1-1 apply, with the exception given hereafter.

(9) A column at the level under consideration, fully connected to the column above and below, may be considered as effectively restrained at such connections, provided the resistance to fire of the building elements, which separate the levels under consideration, is at least equal to the fire resistance of the column.

(10) In the case of a composite frame, for which each of the storeys may be considered as a fire compartment with sufficient fire resistance, the buckling length ℓ_θ of a column on an intermediate storey subject to fire is given by L_{ei}. For a column on the top floor subject to fire the buckling length ℓ_θ in the fire situation is given by L_{et} (see Figure 4.6). For a column on the lowest floor subject to fire, the buckling length ℓ_θ may vary, depending on the rotation rigidity of the column base, from L_{ei} to L_{et}.

NOTE1: Values for L_{ei} and L_{et} may be defined in the National Annex. The recommended values are 0,5 and 0,7 times the system length L.