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FIRE RESISTANCE OF LONG SPAN CELLULAR BEAM MADE OF ROLLED PROFILES

Design Guide

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Contents

1. ANALYTICAL MODEL FOR THE CELLULAR BEAM MADE OF HOT ROLLED SECTIONS IN CASE OF FIRE

1.1. Objectives

The aim of this document is to describe the calculation methods developed to assess the resistance of simply supported cellular beams in fire conditions. This development has been made in the scope of the RFCS FiCEB$^+$ [23] and in the scope of the PHD of O.Vassart [24].

This calculation procedure has been introduced in the ACB+ software available on www.arcelormittal.com/sections

1.2. Principles

The assessment of the fire resistance of a beam consists in calculation for each of the strength criteria, the critical temperature (for which this strength criterion is equal to 1) and the corresponding heating up time. This calculation is made for each of the loads combinations in fire situation.

Among the strength criteria, two types are distinguished:
- the " plastic resistance " criteria, for which the resistance depends only on the steel strength limit f_y
- the " resistance to instability " criteria, for which the resistance depends on the steel strength limit f_y and on the Young modulus E.

Table 1-1 : Criteria taken into account for the fire resistance calculation

Criteria in Plastic Resistance
Resistance of the gross sections (at the level of the web post and filled openings):
Γ_M (*): Bending resistance
Γ_V: Shear resistance
Γ_{MV}: Interaction MV
Resistance of the web posts :
Γ_{Vh}: Resistance to horizontal shear of a web post
Resistance of the net section (at the level of an opening) :
Γ_M (*): Bending resistance
Γ_N (*): Axial resistance
Γ_V: Shear resistance
Γ_{MN} (*): Interaction MN
Γ_{MV} (*): Interaction MV
Γ_{MNV} (*): Interaction MNV
(*): criteria for which a section classification is necessary.
Criteria resistance to instability
Resistance of the gross sections:
Γ_{Vbw}: Resistance to shear buckling
Resistance of the web posts :
Γ_b: Web post buckling resistance
Resistance of the beam :
Γ_{LT}: Resistance to lateral torsional buckling (Only for pure steel beam).

1.2.1. Fire resistance for the plastic criteria

1.2.1.1. Principles

The principles for the calculation of the fire resistance for the plastic criteria are the following:

1. The value of the strength criterion Γ for the time 0 of the fire is calculated taking into account the load combination chosen for the fire calculation. The calculation of the Γ is made in a similar way than in cold conditions by replacing the partial coefficient γ_{M0} with $\gamma_{M,fi}$ see [26].

   For the strength criteria dependent on the section classification, the classification differs from the one in cold conditions (cf. 3.1.2).

2. The critical temperature associated with the value Γ obtained in 1 is calculated from the steel strength reduction factor $k_{y,\theta}$ given in Table 1-3.

   If the value of Γ was obtained for a section of class 4, the critical temperature is calculated with the reduction factor $k_{p,0,2,\theta}$, given in the following table (Table E1 of the EN 1993-1-2).

Table 1-2 : Steel strength reduction factor for a class 4 section

Steel Temperature θ (°C)	Reduction Factor $k_{p,0,2,\theta}$
20	1,000
100	1,000
200	0,890
300	0,780
400	0,650
500	0,530
600	0,300
700	0,130
800	0,070
900	0,050
1000	0,030
1100	0,020
1200	0,000

3. From the massivity factor associated with the considered section and from the critical temperature calculated in 2, the heating up time of the section is calculated in a incremental way.
4.

The following parameters are considered:

θ_{Ref}: "ambient" temperature of the beam; by default θ_{Ref} : = 20°C
Δt: increment of time ; by default Δt = 1 sec
k_{sh}: correction factor for the shadow effect (value of the factor for the rebuilt section) by default $k_{sh} = 0.7$
ρ_a: density of the steel; ρ_a = 7850 kg / m3

Assuming that the temperature of the section in time t_i is equal to θ_i, the temperature θ_{i+1} in time $t_{i+1} = t_i + \Delta t$ is calculated in the following way (formula (4.25) of EN 1993-1.2):

$$\theta_{i+1} = \theta_i + \Delta\theta$$
$$\Delta\theta = k_{sh} \frac{Am/V}{c_a\ \rho_a} h_{net}\ \Delta t$$

Hence c_a is the specific heat of the steel, calculated according to the temperature θ_i with the following formulae (according to 3.4.1.2 of EN 1993-1-2 - all the relations are expressed in J / kgK):

for 20°C ≤ θ_i < 600°C :
$$c_a = 425 + 0.773\ \theta_i - 1.69\ 10^{-3}\ \theta_i^2 + 2.22\ 10^{-6}\ \theta_i^3$$
for 600°C ≤ θ_i < 735°C :
$$c_a = 666 + \frac{13002}{738 - \theta_i}$$
for 735°C ≤ θ_i < 900°C :
$$c_a = 545 + \frac{17820}{\theta_i - 731}$$
for 900°C ≤ θ_i ≤ 1200°C :

$c_a = 650$

h_{net} is the value of calculation of the heat flux, determined according to 3.1 of EN 1991-1-2 by the following relations:

$h_{net} = h_{net,c} + h_{net,r}$
$h_{net,c}$ is the convective part and $h_{net,r}$ is the radiative part.
$h_{net,c} = \alpha_c\,(\theta_{Gi} - \theta_i)$
$h_{net,c} = \Phi\,\varepsilon_m\,\varepsilon_f\,\sigma\,[(\theta_{Gi}+273)^4 - (\theta_i+273)^4]$

Where:

θ_{Gi} is the hot gas temperature for the time i, calculated from the normalised ISO Curve (Eq 3.4 of EN 1991-1-2), according to the following function:

$$\theta_{Gi} = 20 + 345 \log_{10}(8\,t_i + 1)\ [°C]$$

α_c is the thermal transfer coefficient for convection. It's equal by default to 25 W/m^2K (value recommended in 3.2.1 (2) of EN 1991-1-2).
Φ is the shape factor. By default equal to 1.0.
ε_m is the steel surface emissivity, by default equal to 0.7.
ε_f is the fire emissivity, by default equal to 1.0.
σ is the Boltzmann constant (= 5.67 10^{-8} W/m^2K^4)

The critical heating up time is reached when $\theta_i = \theta_{Critique}$.

1.2.1.2. Classification of the sections

For a criterion in fire resistance involving the classification of the studied section, the class of the section is determined with the parameter ε_θ:

$$\varepsilon_\theta = 0.85\,\varepsilon = 0.85\sqrt{\frac{235}{f_y}}$$

All the other parameters of the verification (in particular the reduced slenderness for the calculation of the participating widths) remain unchanged in respect to the cold calculation.

1.2.2. Fire resistance for the instability criteria

The principles of justification of the resistance in fire condition for the instability criteria are the following:

1. From the stresses formed by the fire load combination, the critical temperature is reached when the instability criterion is equal to 1. The calculation of the strength criterion according to the temperature is detailed in the following chapters. The partial safety factor $\gamma_{M,fi}$ is used.
2. From the massivity criterion described below and from the critical temperature calculated in 1, the heating up time is calculated in an incremental way according to the same method as in 2.1.

The considered massivity criterions are the following ones:

- Criterion of instability of the web post: massivity of a straight web post section can be estimated by the following value:
  $A_m / V = 2 / t_w$,
  where t_w, is the thickness of the considered web.
- Lateral torsional buckling criterion: massivity of a "T" section at the level of an opening for the compressed member giving the considered criterion Γ.

1.2.2.1. Instability of a web post

The criterion for resistance to buckling of an intermediate web post at elevated temperature is given by the following equation:

$$\Gamma(\theta_m) = \frac{\left|\sigma_{w,fi,Ed}\right|}{\kappa_\theta \ \sigma_{w,fi,Rd}(\theta_m)}$$

It is based on the calculation of the principal stress resistance in fire situation for the half post being studied $\sigma_{w,fi,Rd}$ and the principal compressive stress in fire situation in the half post being studied $\sigma_{w,fi,Ed}$ ($\sigma_{w,fi,Ed,up}$ for the upper half post and $\sigma_{w,fi,Ed,low}$ for the lower half post).

$\sigma_{w,fi,Ed}$ is calculated from the fire load combinations in the same way as in cold situation (see. 5.8 (5) [27]).

$\sigma_{w,fi,Rd}$, the principal stress resistance is calculated using the following formula based on EN1993-1-2 :

$$\sigma_{w,fi,Rd} = \frac{\chi_{fi} \cdot \xi \cdot k_{y,\theta} \cdot f_y}{\gamma_{M,fi}}$$

Where:

f_y is the steel strenght limit of the considered member

$\gamma_{M,fi}$ is the partial safety factor in fire condition

ξ is a shape factor for the critical section that has been calibrated using the Finite Element modelling (see 5.8 (9) [27])

χ_{fi} is a reduction factor for out-of-plane buckling of the web post adapted for fire situation following EN1993-1-2, and calculated using the following formulae :

$$\chi_{fi} = \frac{1}{\phi_\theta + \left(\phi_\theta^2 - \bar{\lambda}_\theta^2\right)^{0.5}} \quad \text{and} \quad \chi_{fi} \le 1.0$$

$$\phi_\theta = 0.5[1 + \alpha \ \bar{\lambda}_\theta + \bar{\lambda}_\theta^2]$$

$$\alpha = 0.65\sqrt{\frac{235}{f_y}}$$

The reduced non-dimensional slenderness $\bar{\lambda}_\theta$ of the web post being considered in case of fire is given by:

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}} = \sqrt{\frac{\xi f_{yw}}{\sigma_{w,fi,cr}}}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}}$$

Where $k_{y,\theta}$ and $k_{E,\theta}$ are the reduction factors for steel strength limit and Young modulus, respectively, at elevated temperature.

$\bar{\lambda}$ is the non-dimensional slenderness in « cold » conditions (See 5.8 (10) [27])

The values of $k_{y,\theta}$ and $k_{E,\theta}$ are given in Table 1-3 (from table 3.1 of EN 1933-1-2) :

Table 1-3 : Reduction factor for the steel strenght limit and the Young Modulus

Steel temperature θ (°C)	Reduction factor $k_{y,\theta}$	Reduction factor $k_{E,\theta}$
20	1,000	1,000
100	1,000	1,000
200	1,000	0,900
300	1,000	0,800
400	1,000	0,700
500	0,780	0,600
600	0,470	0,310
700	0,230	0,130
800	0,110	0,090
900	0,060	0,0675
1000	0,040	0,0450
1100	0,020	0,0225
1200	0,000	0,000

For the intermediate values of temperature, a linear interpolation is used.

In the calculation of the critical stress, the reference Euler buckling load depends on the Young Modulus E but remains independent from the temperature.

The post-critical reserve of strength κ_θ is calculated from the following relation:

$\kappa_\theta = 1 + 0.625\ (\psi_\theta - 0.3)$ and $1 \leq \kappa_\theta \leq 1.25$

$\psi_\theta = k_{y,\theta}\ \psi$

Where ψ is the non-dimensional factor calculated in the same way as in cold situation (see 5.8 (13) [27]).

1.2.2.2. Resistance to shear buckling

It is suggested not to calculate the shear buckling in fire situation.

1.2.2.3. Lateral torsional buckling

In fire situation, the composite beams are not concerned by this criterion.
As for the cold calculation, the resistance criterion for the lateral torsional buckling of the beam in fire situation is calculated like the buckling of the compressed member. It can be written for a member at the temperature θ:

$$\Gamma_{LT}(\theta) = \frac{N_{m,fi,Ed}}{N_{b,fi,Rd}(\theta)}$$

Where:

$N_{m,fi,Ed}$ is the normal force in the member taking into account the fire load combination. This value is independent of the temperature θ.

$N_{b,fi,Ed}$ is the resisting force to buckling of the member.

This member is the "T" shape between two lateral supports. This value depend on the temperature θ :

$$N_{b,fi,Rd} = \chi_{fi}\, A_0\, k_{y,\theta}\, f_y / \gamma_{M,fi}$$

Where,

$k_{y,\theta}$ is the reduction factor for the steel strength given in the Table 1-3.

A_0 is the surface of the considered section at the level of the opening ("T" section) see relation given in 5.10.1 [27].

$\gamma_{M,fi}$ is the partial safety factor in fire situation

χ_{fi} is the reduction factor for lateral torsional buckling given by the following relationships:

$$\chi_{fi} = \frac{1}{\phi_\theta + (\phi_\theta{}^2 - \bar{\lambda}_\theta{}^2)^{0.5}}$$

$$\phi_\theta = 0.5\left[1 + \alpha\, \bar{\lambda}_\theta + \bar{\lambda}_\theta{}^2\right]$$

$$\alpha = 0.65\sqrt{\frac{235}{f_y}}$$

The reduced non-dimensional slenderness $\bar{\lambda}_\theta$ considered in case of fire is given by:

$$\bar{\lambda}_\theta = \bar{\lambda}\sqrt{\frac{k_{y,\theta}}{k_{E,\theta}}}$$

Where $k_{y,\theta}$ and $k_{E,\theta}$ are the reduction factors for steel strength limit and Young modulus, respectively, at elevated temperature given in Table 1-3.

$\bar{\lambda}$ is the non-dimensional slenderness in « cold » conditions calculated from 5.10.1 (4) [27]

Nota: the critical load N_{cr} used in the calculation of $\bar{\lambda}$ is independent of the temperature and is obtained from the relationship given in 5.10.1 (5) [27].

1.3. Example of application

1.3.1. Characteristics of the beam

Beam:	IPE400 non composite
Steel grade:	S355
Span:	20m
a_0:	500mm
w:	125mm
H_t:	633.8mm
Distance between beams:	3m
Permanet Load:	$1kN/m^2$
Snow Load:	$0.5kN/m^2$
Fire load Combinations:	1*G + 0*Q

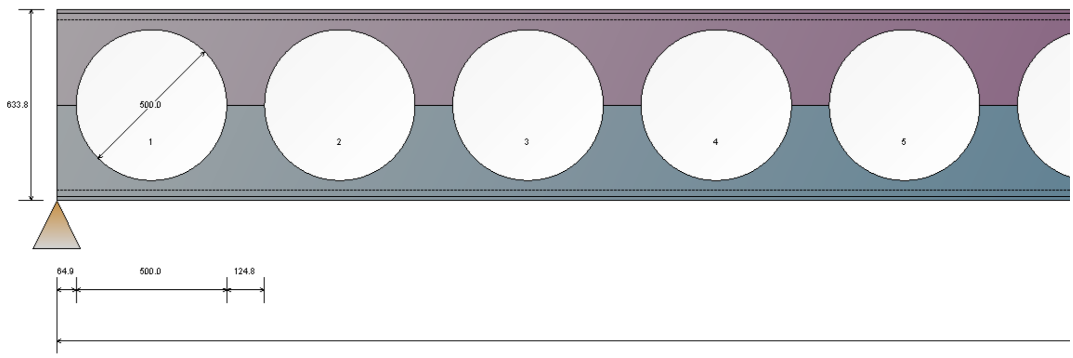


Figure 1–1 : Geometry of the beam

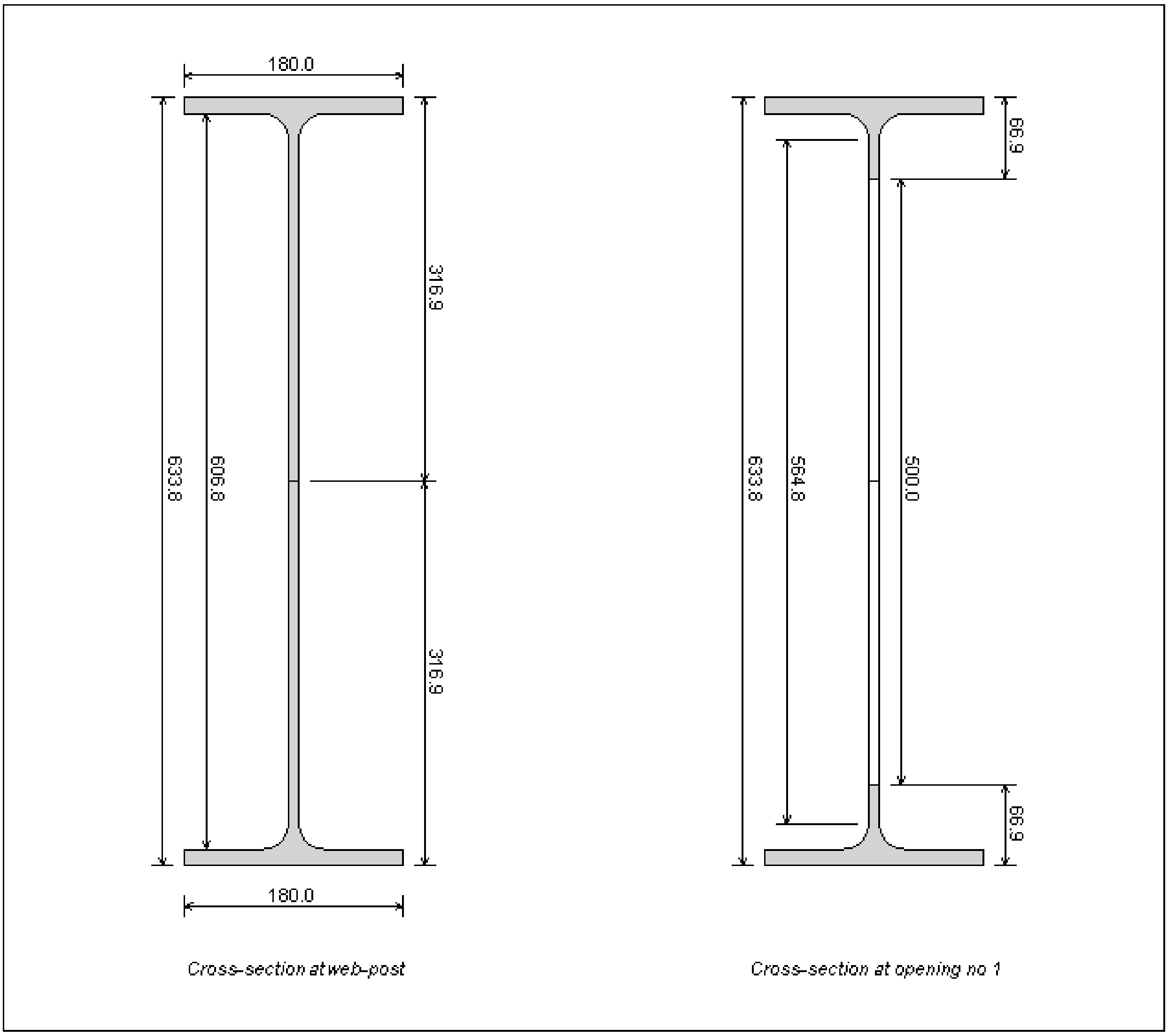


Figure 1–2 : Cross section of the beam

1.3.2. Resistance check

1.3.2.1. Net section at opening no 1 : Resistance to bending moment

Bending moment	M_{Ed} = 11.07 kNm	
Shear forces	$V_{Ed,l}$ = -34.85 kN	$V_{Ed,r}$ = -34.85 kN
Axial forces	$N_{Ed,l}$ = 0.0 kN	$N_{Ed,r}$ = 0.0 kN
Axial forces in chord	$N_{m,sup,l}$ = 18.17 kN	$N_{m,sup,r}$ = 18.17 kN
	$N_{m,inf,l}$ = -18.17 kN	$N_{m,inf,r}$ = -18.17 kN
Shear forces in chord	$V_{m,sup,l}$ = -17.42 kN	$V_{m,sup,r}$ = -17.42 kN
	$V_{m,inf,l}$ = -17.42 kN	$V_{m,inf,r}$ = -17.42 kN
Angle	ϕ = 24.0	
Partial factor	$\gamma_{M,fi}$ = 1.00	
Yield strengths	$f_{y,top}$ = 355 Mpa	$f_{y,bot}$ = 355 MPa

Top chord

Inclined Tee section	h_ϕ = 96.9 mm	
	A_ϕ = 3555 mm^2	$A_{v\phi}$ = 1269 mm^2
Projected forces	N_ϕ = 23.69 kN	V_ϕ = -13.28 kN
	M_ϕ = -2.266 kNm	
Class of the chord	Class 2	
Bending resistant moment at 20°C	$M_{c,Rd\phi}$ = 16.05 kNm	
Criterion	$\Gamma_{M,fi}$ = 0.141	

Bottom chord

Inclined Tee section	h_ϕ = 96.9 mm	
	A_ϕ = 3555 mm^2	A_v = 1269 mm^2
Projected forces	N_ϕ = -9.515 kN	V_ϕ = -18.56 kN
	M_ϕ = -2.400 kNm	
Class of the chord	Class 1	
Bending resistant moment	$M_{c,Rd\phi}$ = 16.05 kNm	
Criterion	$\Gamma_{M,fi}$ = 0.150	

Critical temperature Γ_M :	767°C
Am/V Γ_M:	150.7 m^{-1}

1.3.2.2. Net section at opening no 16 - Resistance to normal force

Bending moment	M_{Ed} = 179.7 kNm	
Shear forces	$V_{Ed,l}$ = -1.124 kN	$V_{Ed,r}$ = -1.124 kN
Axial forces	$N_{Ed,l}$ = 0.0 kN	$N_{Ed,r}$ = 0.0 kN
Axial forces in chord	$N_{m,sup,l}$ = 295.1 kN	$N_{m,sup,r}$ = 295.1 kN
	$N_{m,inf,l}$ = -295.1 kN	$N_{m,inf,r}$ = -295.1 kN
Shear forces in chord	$V_{m,sup,l}$ = -0.562 kN	$V_{m,sup,r}$ = -0.562 kN
	$V_{m,inf,l}$ = -0.562 kN	$V_{m,inf,r}$ = -0.562 kN
Angle	ϕ = 0.0	
Partial factor	$\gamma_{M,fi}$ = 1.00	
Yield strengths	$f_{y,top}$ = 355 Mpa	$f_{y,bot}$ = 355 MPa

Top chord

Inclined Tee section	h_ϕ = 66.9 mm	
	A_ϕ = 3078 mm^2	$Av\phi$ = 990.0 mm^2
Projected forces	N_ϕ = 295.1 kN	$V\phi$ = -0.562 kN
	M_ϕ = 0.0 kNm	
Class of the chord	Class 2	
Normal resistant force at 20°C	$N_{c,Rd\phi}$ = 1093 kN	
Criterion	$\Gamma_{N,fi}$ = 0.270	

Bottom chord

Inclined Tee section	h_ϕ = 66.9 mm	
	A_ϕ = 3078 mm^2	$A_{v\phi}$ = 990.0 mm^2
Projected forces	N_ϕ = -295.1 kN	V_ϕ = -0.562 kN
	M_ϕ = 0.0 kNm	
Class of the chord	Class 1	
Normal resistant force at 20°C	$N_{c,Rd\phi}$ = 1093 kN	
Criterion	$\Gamma_{N,fi}$ = 0.270	

Critical temperature Γ_N :	683°C
Am/V Γ_N:	154.5 m^{-1}

1.3.2.3. Net section at opening no 15 - Resistance to shear force

Bending moment	M_{Ed} = 178.3 kNm	
Shear forces	$V_{Ed,l}$ = -3.372 kN	$V_{Ed,r}$ = -3.372 kN
Axial forces	$N_{Ed,l}$ = 0.0 kN	$N_{Ed,r}$ = 0.0 kN
Axial forces in chord	$N_{m,sup,l}$ = 292.8 kN	$N_{m,sup,r}$ = 292.8 kN
	$N_{m,inf,l}$ = -292.8 kN	$N_{m,inf,r}$ = -292.8 kN
Shear forces in chord	$V_{m,sup,l}$ = -1.686 kN	$V_{m,sup,r}$ = -1.686 kN
	$V_{m,inf,l}$ = -1.686 kN	$V_{m,inf,r}$ = -1.686 kN
Angle	ϕ= -39.0	
Partial factor	$\gamma_{M,fi}$ = 1.00	
Yield strengths	$f_{y,top}$ = 355 MPa	$f_{y,bot}$ = 355 MPa

Top chord

Inclined Tee section	h_ϕ = 158 mm	
	A_ϕ = 4523 mm^2	$A_{v\phi}$ = 1836 mm^2
Projected forces	N_ϕ = 226.5 kN	V_ϕ = -76.10 kN
	M_ϕ = 3.652 kNm	
Shear resistant force at 20°C	$V_{c,Rd\phi}$ = 376.3 kN	
Criterion	$\Gamma_{V,fi}$ = 0.202	

Bottom chord

Inclined Tee section	h_ϕ = 158 mm	
	A_ϕ = 4523 mm^2	$A_{v\phi}$ = 1836 mm^2
Projected forces	N_ϕ = -228.6 kN	V_ϕ = 73.48 kN
	M_ϕ = -2.851 kNm	
Shear resistant force at 20°C	$V_{c,Rd\phi}$ = 376.3 kN	
Criterion	$\Gamma_{V,fi}$ = 0.195	

Critical temperature Γ_V :	723°C
Am/V Γ_V:	145.4 m^{-1}

1.3.2.4. Net section at opening no 12 - Interaction M-N-V

Bending moment	M_{Ed} = 165.6 kNm	
Shear forces	$V_{Ed,l}$ = -10.12 kN	$V_{Ed,r}$ = -10.12 kN
Axial forces	$N_{Ed,l}$ = 0.0 kN	$N_{Ed,r}$ = 0.0 kN
Axial forces in chord	$N_{m,sup,l}$ = 272.0 kN	$N_{m,sup,r}$ = 272.0 kN
	$N_{m,inf,l}$ = -272.0 kN	$N_{m,inf,r}$ = -272.0 kN
Shear forces in chord	$V_{m,sup,l}$ = -5.059 kN	$V_{m,sup,r}$ = -5.059 kN
	$V_{m,inf,l}$ = -5.059 kN	$V_{m,inf,r}$ = -5.059 kN
Angle	ϕ = -21.0	
Partial factor	$\gamma_{M,fi}$ = 1.00	
Yield strengths	$f_{y,top}$ = 355 MPa	$f_{y,bot}$ = 355 MPa

Top chord

Inclined Tee section	h_ϕ = 89.4 mm	
	A_ϕ = 3437 mm^2	$A_{v\phi}$ = 1200.0 mm^2
Projected forces	N_ϕ = 255.8 kN	V_ϕ = -29.31 kN
	M_ϕ 0.163 kNm	
Shear resistant force at 20°C	$V_{c,Rd\phi}$ = 246.0 kN	$\Gamma_{V,fi}$ = 0.119
Reduction	ρ = 0.000	(No reduction)
Normal resistant force at 20°C	$N_{V,Rd}$ = 1220 kN	$\Gamma_{NV,fi}$ = 0.210
Bending resistant moment at 20°C	$M_{V,Rd}$ = 14.04 kNm	$\Gamma_{MV,fi}$ = 0.012
Interaction MNV	$\Gamma_{MNV,fi}$ = 0.221	

Bottom chord

Inclined Tee section	h_ϕ = 89.4 mm	
	A_ϕ = 3437 mm^2	$A_{v\phi}$ = 1200.0 mm^2
Projected forces	N_ϕ = -252.1 kN	V_ϕ = -29.31 kN
	M_ϕ = -1.335 kNm	
Shear resistant force at 20°C	$V_{c,Rd\phi}$ = 246 kN	$\Gamma_{V,fi}$ = 0.158
Reduction	ρ = 0.000	(No reduction)
Normal resistant force at 20°C	$N_{V,Rd}$ = 1220 kN	Γ_{NV} = 0.249
Bending resistant moment at 20°C	$M_{V,Rd}$ = 14.04 kNm	Γ_{MV} = 0.095
Interaction MNV	Γ_{MNV} = 0.302	

Critical temperature Γ_{MNV} :	670°C
Am/V Γ_{MNV}:	151.5 m^{-1}

1.3.2.5. Shear resistance of Web post no 31

Tee geometrical centres	d_G = 608.9 mm	
Bending moments	$M_{Ed,l}$ = 32.14 kNm	$M_{Ed,r}$ = 11.07 kNm

Axial forces in tees	$N_{m,Sup,l}$ = 52.79 kN	$N_{m,Inf,l}$ = -52.79 kN
	$N_{m,Sup,r}$ = 18.17 kN	$N_{m,Inf,r}$ = -18.17 kN
Horizontal shear force in post	V_{hm} = -34.62 kN	
Post width	w = 125.0 mm	
Resistant shear forces at 20°C	$V_{hRd,top}$ = 220.33 kN	$V_{hRd,bot}$ = 220.33 kN
Checkings	$\Gamma_{Vh,top}$ = 0.157	$\Gamma_{Vh,bot}$ = 0.157

Critical temperature Γ_{Vh} :	761°C
Am/V Γ_{Vh}:	232.6 m^{-1}

1.3.2.6. Stability of Web post no 31

Diameter	a_0 = 500.0 mm	
Cells spacing	e = 625.0 mm	$\alpha = e / a_0 = 1.25$
Height of cross section	H_t = 633.8 mm	
Heights of chords	$h_{m,top}$ = 316.9 mm	$h_{m,bot}$ = 316.9 mm
Heights of tees	$h_{Te,top}$ = 66.9 mm	$h_{Te,bot}$ = 66.9 mm
Tees geometrical centres	$d_{G,top}$ = 304.4 mm	$d_{G,bot}$ = 304.4 mm
dG = dG,top + dG,bot	d_G = 608.9 mm	
Area of tees	$A_{0,top}$ = 3078.4 mm^2	$A_{0,bot}$ = 3078.4 mm^2
Shear area of tees	$A_{v0,top}$ = 990.0 mm^2	$A_{v0,bot}$ = 990.0 mm^2
Yield strengths	$f_{y,top}$ = 355 MPa	$f_{y,bot}$ = 355 MPa
Shear forces	$V_{Ed,l}$ = -32.60 kN	$V_{Ed,r}$ = -34.85 kN
Moments	$M_{Ed,l}$ = 32.14 kNm	$M_{Ed,r}$ = 11.07 kNm
Shear parameters	η = 0.292	k_{Av} = 0.500
Normal forces in chords	$N_{m,ltop}$ = 52.79 kN	$N_{m,lbot}$ = -52.79 kN
	$N_{m,rtop}$ = 18.17 kN	$N_{m,rbot}$ = -18.17 kN
Shear forces in chords	$V_{m,ltop}$ = -16.30 kN	$V_{m,lbot}$ = -16.30 kN
	$V_{m,rtop}$ = -17.42 kN	$V_{m,rbot}$ = -17.42 kN
Forces in the post	V_{hm} = -34.62 kN	M_{hm} = 0.00 kNm
Critical section	d_W = 97.3 mm	L_W = 164.4 mm
Moments in the critical section	$M_{cEd,top}$ = -3.37 kNm	$M_{cEd,bot}$ = -3.37 kNm
Principal stresses	$\sigma_{W,fi,top}$ = 102 MPa	$\sigma_{W,fi,bot}$ = 102 MPa
Critical forces	$V_{hCr,top}$ = 341.48 kN	$V_{hCr,bot}$ = 341.48 kN
	$N_{mCr,top}$ = 1533.08 kN	$N_{mCr,bot}$ = 1533.08 kN
Critical coefficients	$\beta_{Cr,top}$ = 9.628	$\beta_{Cr,bot}$ = 9.988
	$\alpha_{Cr,top}$ = 9.805	$\alpha_{Cr,bot}$ = 9.988
Critical stresses	$\sigma_{Cr,top}$ = 1004 MPa	$\sigma_{Cr,bot}$ = 1023 MPa

Reduced slendernesses at 20°C	λ_{top} = 0.729	λ_{bot} = 0.723
With	ξ = 1.505	
Reduction factors at 20°C	χ_{top} = 0.834	χ_{bot} = 0.837
Resistant stresses at 20°C	$\sigma_{WRd,top}$ = 445 MPa	$\sigma_{MPa\ WRd,bot}$ = 447MPa
Plastic moments of tees at 20°C	$M_{plRd,Te,top}$ = 8.92 kNm	$M_{plRd,Te,bot}$ = 8.92 kNm

Psi factor at 20°C	$\Psi_{top} = 0.820$	$\Psi_{bot} = 0.820$
Post-buckling factor	$\kappa_{top} = 1.250$	$\kappa_{bot} = 1.250$
Critical temperature	$\theta_{crit,top} = 646°C$	$\theta_{crit,bot} = 647°C$
$k_{y,\theta}$ at critical temperature	$k_{y,\theta,top} = 0.3596$	$k_{y,\theta,bot} = 0.3572$
$k_{E,\theta}$ at critical temperature	$k_{E,\theta,top} = 0.229$	$k_{E,\theta,bot} = 0.2272$
Reduced slendernesses at θ_{crit}	$\lambda\theta_{,top} = 0.92$	$\lambda\theta_{,bot} = 0.92$
Reduction factors at θ_{crit}	$\chi_{\theta,top} = 0.53$	$\chi_{\theta,bot} = 0.53$
Psi factor at θ_{crit}	$\Psi_{\theta,top} = 0.24$	$\Psi_{\theta,bot} = 0.24$

1.3.2.7. Bending resistance of gross sections

Section at web post no 16 (Section no 33)

Internal moment and force	$M_{Ed} = 179.86$ kNm	$N_{Ed} = 0.00$ kN
Upper flange under compression:	Class 1	
Class of the web		
Steel	$f_{y,w} = 355$ MPa	$\varepsilon_w = 0.814$
Slenderness:	$c / t = 65.67$	
Plastic distribution factor	$\alpha = 0.50$	
Class of the web	2	
Check of the resistance (Class2)		
Steel	$f_{y,top} = 355$ MPa	$f_{y,bot} = 355$ MPa
Partial factor	$\gamma_{M,fi} = 1.00$	
Plastic resistant moment at 20°C		$M_{pl,Rd} = 856.24$ kNm
Criterion	$\Gamma_{Mg,fi} = 0.210$	
Critical temperature Γ_{Mg} :	717°C	
Am/V Γ_{Mg}:	185 m^{-1}	

1.3.2.8. Shear resistance of gross sections

Section at left end (Section no 1)

Height of the cross-section	$h = 633.8$ mm	
Shear area	$A_{v,top} = 3140.0\ mm^2$	$A_{v,bot} = 3140.0\ mm^2$
Yield strengths	$f_{y,top} = 355$ MPa	$f_{y,bot} = 355$ MPa
Shear design force	$V_{Ed} = 35.97$ kN	
Shear resistance force at 20°C	$V_{plRd} = 1287.14$ kN	$\gamma_{M,fi} = 1.00$
Criterion	$\Gamma_{Vg} = 0.028$	
Critical temperature Γ_{Vg} :	1060°C	
Am/V Γ_{Vg}:	232.6 m^{-1}	

1.3.3. Summary of the results

1.3.3.1. Checking of net sections at openings

Parameter	Γ	Angle (°)	Am/V (m^{-1})	θ_{crit} (°C)
Γ_M	0.150	24.0	150.7	767
Γ_N	0.270	0.0	154.5	683
Γ_V	0.202	-39.0	145.4	723
Γ_{MN}	0.302	21.0	151.5	670
Γ_{MV}	0.150	24.0	150.7	767
Γ_{NV}	0.270	0.0	154.5	683
Γ_{MNV}	0.302	21.0	151.5	670

1.3.3.2. Post checking

Parameter	Γ	Am/V (m^{-1})	θ_{crit} (°C)
Γ_{Vh}	0.157	232.6	761
Γ_b	-	232.6	646

1.3.3.3. Gross section checking

Parameter	Γ	Am/V (m^{-1})	θ_{crit} (°C)
Γ_{Mg}	0.210	185	717
Γ_{Vg}	0.028	232.6	1060

2. BAILEY'S METHODS EXTENDED TO LONG SPAN CELLULAR BEAMS

Executive summary

Large-scale fire tests conducted in a number of countries and observations of actual building fires have shown that the fire performance of composite steel framed buildings is much better than is indicated by fire resistance tests on isolated elements. It is clear that there are large reserves of fire resistance in modern steel-framed buildings and that standard fire resistance tests on single unrestrained members do not provide a satisfactory indicator of the performance of such structures.

This publication presents guidance on the application of a simple design method, as implemented in FiCEB design spreadsheet, which has been developed as a result of observation and analysis of the BRE Cardington large-scale building fire test program carried out during 1995 and 1996 and more recent testing on floor slabs containing cellular beams. The recommendations are conservative and are limited to structures similar to those tested, i.e. non-sway steel-framed buildings with composite floors. The guidance gives designers access to whole building behaviour and allows them to determine which members can remain unprotected while maintaining levels of safety equivalent to traditional methods.

In recognition that many fire safety engineers are now considering natural fires, a natural fire model may be inputted or calculated using the parametric fire method from EN1991-1-2. These options are included alongside the use of the standard fire model; all three are expressed as temperature-time curves.

2.1. Introduction

The design recommendations in this publication are based on the performance of composite floor plates, observed during actual building fires and full-scale fire tests[1,2,3]. These conservative recommendations for fire design may be considered as equivalent to advanced methods in the Eurocodes.

Large-scale natural fire tests carried out in a number of countries have shown consistently that the inherent fire performance of composite floor plates with unprotected steel elements is much better than the results of standard tests with isolated elements would suggest. Evidence from real fires indicates that the amount of protection being applied to steel elements may be excessive in some cases. In particular, the Cardington fire tests presented an opportunity to examine the behaviour of a real structure in fire and to assess the fire resistance of unprotected composite structures under realistic conditions. Most test evidence is available for composite beams with plain webs but this project has included a test on a 15m by 9m floor plate with cellular composite beams and similar good behaviour was observed.

Where national building regulations permit performance-based design of buildings in fire, the design method provided by this guide may be applied to demonstrate the fire resistance of the structure without applied fire protection. In some countries acceptance of such demonstration may require special permission from the national building control authority.

The recommendations presented in this publication can be seen as extending the fire engineering approach in the area of structural performance and developing the concept of fire safe design. It is intended that designs carried out in accordance with these recommendations will achieve at least the level of safety required by national regulations while allowing some economies in construction costs.

In addition to fire resistance for the standard temperature-time curve, recommendations are presented for buildings designed to withstand a natural fire. Natural fires can be defined using the parametric temperature-time curve given in EN1991-1-2 or be user define time temperature curves from other fire analysis software.

The recommendations apply to composite frames broadly similar to the eight-storey building tested at Cardington, as illustrated in Figure 2–1. This project has shown that the scope may also be extended to cellular beams fabricated from rolled sections.

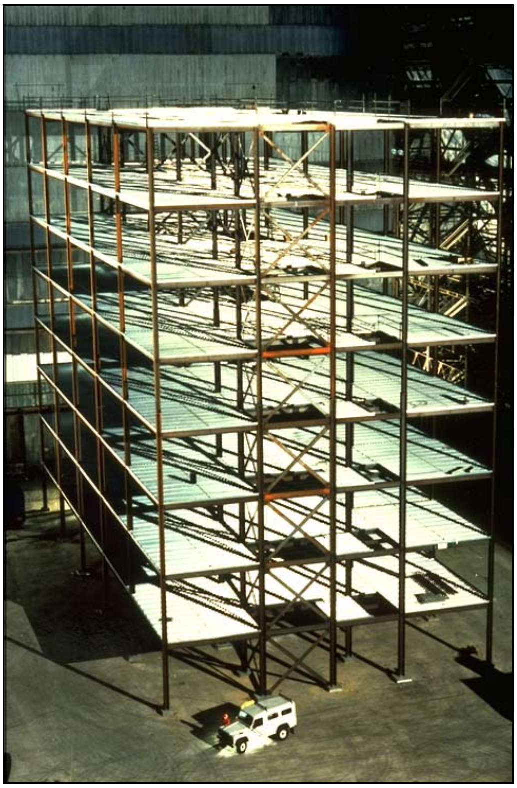


Figure 2–1 : Cardington test building prior to the concreting of the floors

2.2. Basis of design

This Section gives an overview of the design principles and assumptions underlying the development of the simple design method.

The design guidance has been developed from research based on the results from fire tests, ambient temperature tests and finite element analyses.

2.2.1. Fire safety

The design recommendations given in the simple design method have been prepared such that the following fundamental fire safety requirements are fulfilled:

- There should be no increased risk to life safety of occupants, fire fighters and others in the vicinity of the building, relative to current practice.
- On the floor exposed to fire, excessive deformation should not cause failure of compartmentation, in other words, the fire will be contained within its compartment of origin and should not spread horizontally or vertically.

2.2.2. Type of structure

The design guidance given in the simple design method applies only to steel-framed buildings with composite floor beams and slabs of the following general form:

- braced frames not sensitive to buckling in a sway mode,
- frames with connections designed using simple joint models,
- composite floor slabs comprising steel decking, a single layer of reinforcing mesh and normal or lightweight concrete, designed in accordance with EN1994-1-1 [7],
- floor beams designed to act compositely with the floor slab and designed to EN 1994-1-1.
- cellular beams fabricated from hot rolled steel sections

The guidance does **not** apply to:

- floors constructed using precast concrete slabs,
- internal floor beams that have been designed to act non-compositely (beams at the edge of the floor slab may be non-composite),
- beams with service openings (except cellular beams as defined above).

2.2.3. Simple joint models

The joint models adopted during the development of the guidance given in this publication assume that bending moments are not transferred through the joint. The joints are known as ‘simple’.

Beam-to-column joints that may be considered as ‘simple’ include joints with the following components:

- Flexible end plates (Figure 2–2)
- Fin plates (Figure 2–3)
- Web cleats (Figure 2–4)

Further information on the design of the components of ‘simple’ joints is given in Section 2.3.9.

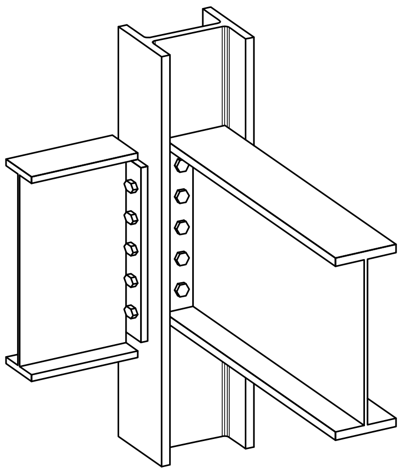


Figure 2–2 : Example of a joint with flexible end plate connections

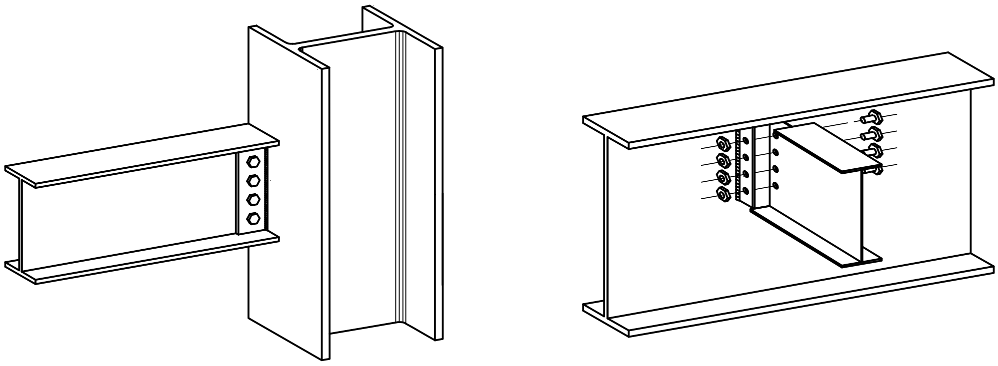


Figure 2–3 : Examples of joints with fin plate connections

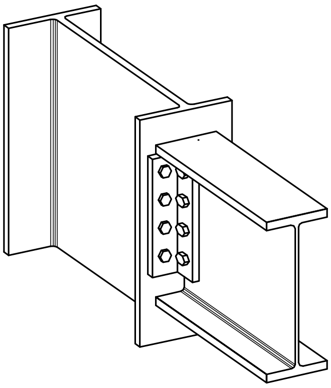


Figure 2–4 : Example of a joint with a web cleat connection

2.2.4. Floor slabs and beams

The design recommendations given in this guide are applicable to profiled steel decking up to 80 mm deep with depths of concrete above the steel decking from 60 to 90 mm. The resistance of the steel decking is ignored in the fire design method but the presence of the steel decking prevents spalling of the concrete on the underside of the floor slab. This type of floor construction is illustrated in Figure 2–5.

The design method can be used with either isotropic or orthotropic reinforcing mesh, that is, meshes with either the same or different areas in orthogonal directions. The steel grade for the mesh reinforcement should be specified in accordance with EN10080. As the design method requires ductile mesh reinforcement in order to accommodate large slab deflections Class B or Class C should be specified. The FiCEB design spreadsheet can only be used for welded mesh reinforcement and can not consider more than one layer of reinforcement. Reinforcement bars in the ribs of the composite slab are **not** required**.**

The software allows user defined sizes of welded mesh the user must input the area of the mesh in each direction. Common French and UK mesh sizes are given in the table below.

Table 2-1 : Fabric mesh as defined by BS 4483 [9]

Mesh Reference	Size of mesh (mm)	Weight (kg/m^2)	Longitudinal wires		Transverse wires	
			Size (mm)	Area (mm^2/m)	Size (mm)	Area (mm^2/m)
A142	200x200	2.22	6	142	6	142
A193	200x200	3.02	7	193	7	193
A252	200x200	3.95	8	252	8	252
A393	200x200	6.16	10	393	10	393
B196	100x200	3.05	5	196	7	193
B283	100x200	3.73	6	283	7	193
B385	100x200	4.53	7	385	7	193
B503	100x200	5.93	8	503	8	252

Table 2-2 : Fabric mesh commonly used in French market

Mesh Reference	Size of mesh (mm)	Weight (kg/m^2)	Longitudinal wires		Transverse wires	
			Size (mm)	Area (mm^2/m)	Size (mm)	Area (mm^2/m)
ST 20	150x300	2.487	6	189	7	128
ST 25	150x300	3.020	7	257	7	128
ST 30	100x300	3.226	6	283	7	128
ST 35	100x300	6.16	7	385	7	128
ST 50	100x300	3.05	8	503	8	168
ST 60	100x300	3.73	9	636	9	254
ST 15 C	200x200	2.22	6	142	6	142
ST 25 C	150x150	4.03	7	257	7	257
ST 40 C	100x100	6.04	7	385	7	385
ST 50 C	100x100	7.90	8	503	8	503
ST 60 C	100x100	9.98	9	636	9	636

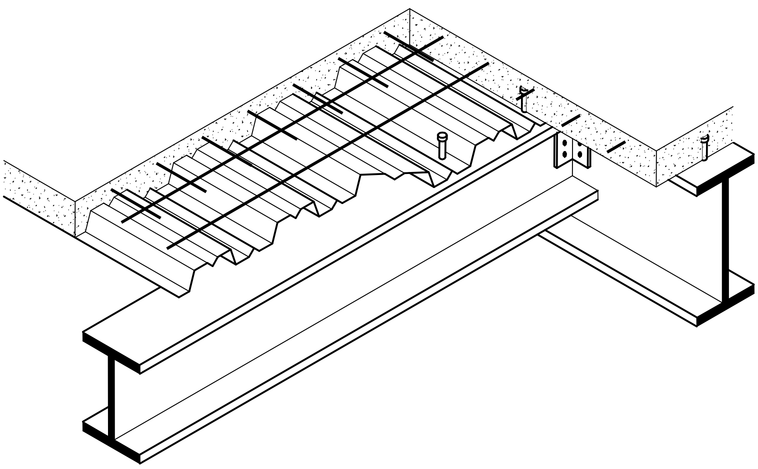


Figure 2–5 : Cut away view of a typical composite floor construction

It is important to define the beam sizes used in the construction of the cellular beams within the floor plate as this will influence the fire performance of the floor plate. The designer will need to have details of the serial size, steel grade and degree of shear connection available for the top and bottom tee of the internal cellular beams. The FiCEB spreadsheet allows the user to choose from a predefined list of serial sizes covering common British and European I and H sections.

2.2.5. Floor design zones

The design method requires the designer to split the floor plate into a number of floor design zones as shown in Figure 2–6. The beams on the perimeter of these floor design zones must be designed to achieve the fire resistance required for the floor plate and will therefore normally be fire protected.

A floor design zone should meet the following criteria:

- Each zone should be rectangular.
- Each zone should be bounded on all sides by beams.
- The beams within a zone should only span in one direction.
- Columns should not be located within a floor design zone; they may be located on the perimeter of the floor design zone.
- For fire resistance periods in excess of 60 minutes, or when using the parametric temperature-time curve, all columns should be restrained by at least one fire protected beam in each orthogonal direction.

All internal beams within the zone may be left unprotected, provided that the fire resistance of the floor design zone is shown to be adequate using the FiCEB spreadsheet. The size and spacing of these unprotected beams are not critical to the structural performance in fire conditions.

An example of a single floor design zone is given in Figure 2–6.

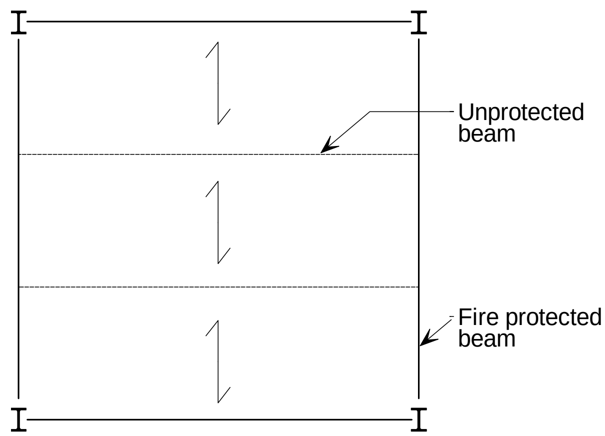


Figure 2–6 : Example of a floor design zone

2.2.6. Combination of actions

The combination of actions for accidental design situations given in 6.4.3.3 and Table A1.3 of EN 1990 [11] should be used for fire limit state verifications. With only unfavourable permanent actions and no prestressing actions present, the combination of actions to consider is:

$$\sum G_{k,j,\sup} + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) Q_{k,1} + \sum \psi_{2,i} Q_{k,i}$$

Where:

$G_{k,j,sup}$ Unfavourable permanent action

A_d Accidental action

$Q_{k,1}$ and $Q_{k,i}$ Accompanying variable actions, main and other respectively

$\psi_{1,1}$ Factor for the frequent value of the leading variable action

$\psi_{2,i}$ Factor for the quasi-permanent value of the ith variable action

The use of either $\psi_{1,1}$ or $\psi_{2,1}$ with $Q_{k,1}$ should be specified in the relevant National Annex. The National Annex for the country where the building is to be constructed should be consulted to determine which factor to use.

The values used for the ψ factors relate to the category of the variable action they are applied to. The Eurocode recommended values for the ψ factors for buildings are given in Table A1.1 of EN 1990; those values are confirmed or modified by the relevant National Annex. The ψ factor values for buildings in the UK and France are summarised in Table 2-3. For floors that allow loads to be laterally distributed, the following uniformly distributed loads are given for moveable partitions in 6.3.1.2(8) of EN 1991-1-1 [12]:

- Movable partitions with a self-weight ≤ 1,0 kN/m wall length: q_k = 0,5 kN/m^2
- Movable partitions with a self-weight ≤ 2,0 kN/m wall length: q_k = 0,8 kN/m^2
- Movable partitions with a self-weight ≤ 3,0 kN/m wall length: q_k = 1,2 kN/m^2.
- Moveable partitions with self-weights greater than 3.0 kN/m length should be allowed for by considering their location.

The Eurocode recommended values for variable imposed loads on floors are given in Table 6.2 of EN 1991-1-1; those values may also be modified by the relevant National Annex. Table 2-4 presents the Eurocode recommended values and the values given in the UK and French National Annexes for the imposed load on an office floor.

Table 2-3 : Values of ψ factors

Actions	Eurocode recommended values		UK National Annex values		French National Annex values	
	ψ_1	ψ_2	ψ_1	ψ_2	ψ_1	ψ_2
Domestic, office and traffic areas where: 30 kN < vehicle weight ≤ 160 kN	0.5	0.3	0.5	0.3	0.5	0.3
Storage areas	0.9	0.8	0.9	0.8	0.9	0.8
Other*	0.7	0.6	0.7	0.6	0.7	0.6

* Climatic actions are not included

Table 2-4 : Imposed load on an office floor

Category of loaded area	Eurocode recommended values		UK National Annex values		French National Annex values	
	q_k (kN/m^2)	Q_k (kN)	q_k (kN/m^2)	Q_k (kN)	q_k (kN/m^2)	Q_k (kN)
B – Office areas	3.0	4.5	2.5* or 3.0**	2.7	3.5 – 5.0	15.0

* Above ground floor level
**At or below ground floor level

2.2.7. Fire exposure

The recommendations given in the simple design method may be applied to buildings in which the structural elements are considered to be exposed to a standard temperature-time curve or parametric temperature-time curve, both as defined in EN 1991-1-2. Advanced model may also be used to define a temperature –time curve for a natural fire scenario. The resulting temperature-time time curve may be input to the ‘User defined’ worksheet on the FiCEB spreadsheet.

In all cases, the normal provisions of national regulations regarding means of escape should be followed.

2.2.8. Fire resistance

The Cardington fire tests were conducted using both real (‘natural’) fires and non standard gas fires. The tests did not follow the standard temperature-time curve that is used to define the fire resistance periods given in national regulations. Design temperatures in terms of the standard fire resistance temperature-time curve must therefore be calculated using thermal analysis.

The recommended periods of fire resistance for elements of construction in various types of building may be found in national regulations. The structural elements of most two-storey buildings require 30 minutes fire resistance and those in most buildings between three and five storeys require 60 minutes fire resistance.

The following recommendations may be applied to buildings in which the elements of structure are required to have up to 120 minutes fire resistance. Provided that they are followed, composite steel framed buildings will maintain their stability for this period of fire resistance, when any compartment is subject to the standard temperature-time curve [1].

All composite steel framed buildings with composite floors may be considered to achieve 15 minutes fire resistance without fire protection, and so no specific recommendations are given in this case.

2.2.8.1. Natural fire (parametric temperature-time curve)

The FiCEB software allows the effect of natural fire on the floor plate to be considered using the parametric temperature-time curve as defined in EN1991-1-2 Annex A [25]. It should be noted that this is an Informative Annex and its use may not be permitted in some European countries, such as France. Before final design is undertaken the designer should consult the relevant National Annex.

Using this parametric fire curve, the software defines the compartment temperature taking account of:
- The compartment size:
- Compartment length
- Compartment width
- Compartment height

The height and area of windows:
- Window height
- Window length
- Percentage open window

The amount of combustibles and their distribution in the compartment
- Fire Load
- Combustion factor
- The rate of burning
- The thermal properties of the compartment linings

The temperature of a parametric fire will often rise more quickly than the standard fire in the early stages but, as the combustibles are consumed, the temperature will decrease rapidly. The standard fire steadily increases in temperature indefinitely.

The standard temperature-time curve and a typical parametric temperature-time curve are shown in Figure 2–7.

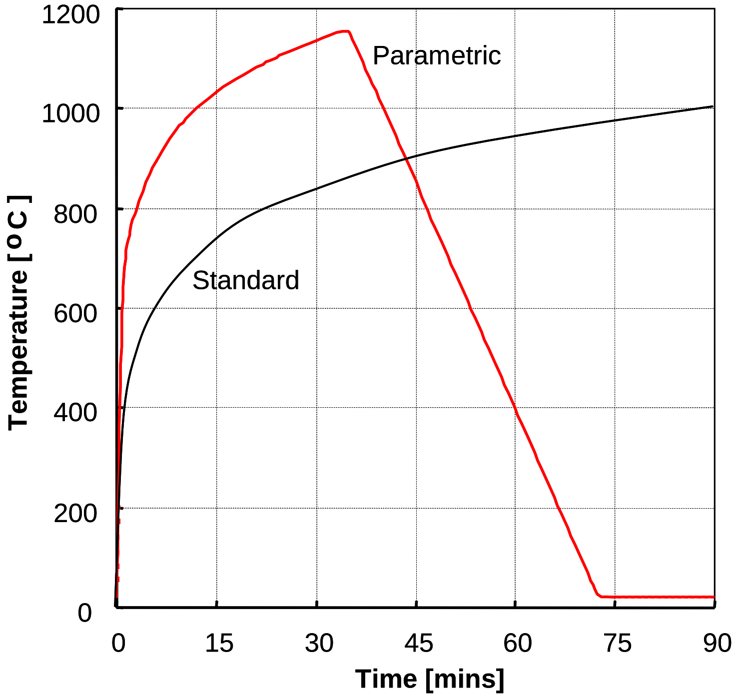


Figure 2–7 : Comparison of typical parametric and standard temperature-time curve

2.3. Recommendations for structural elements

2.3.1. Floor design zones

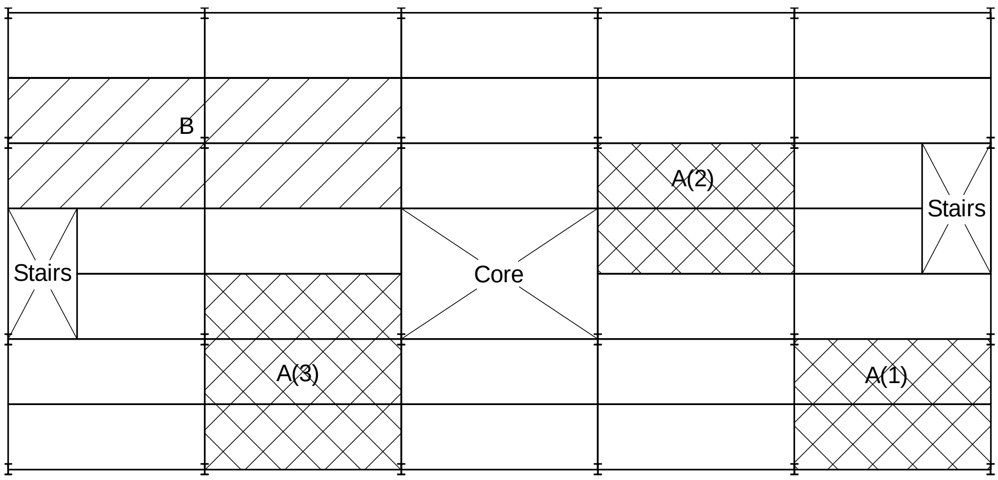
Each floor should be divided into design zones that meet the criteria given in Section 2.2.5.

The division of a floor into floor design zones is illustrated in Figure 2–8. Floor zones designated ‘A’ are within the scope of the design guide and their load bearing performance in fire conditions may be determined using the FiCEB spreadsheet. The zone designated ‘B’ is outside the scope of the software because it contains a column and the beams within the zone do not all span in the same direction.

A single floor zone is illustrated in Figure 2–9 showing the beam span designations used in the FiCEB software. Normal design assumes that floor loads are supported by secondary beams which are themselves supported on primary beams.

The fire design method assumes that at the fire limit state, the resistance of the unprotected internal beams reduces significantly, leaving the composite slab as a two way spanning element simply supported around its perimeter. In order to ensure that the slab can develop membrane action, the FiCEB spreadsheet computes the moment applied to each perimeter beam as a result of the actions on the floor design zone. To maintain the vertical support to the perimeter of the floor design zone in practice, the degree of utilisation and hence the critical temperature of these perimeter beams must be calculated using appropriate cellular beam design software. The fire protection for these beams should be designed on the basis of this critical temperature and the fire resistance period required for the floor plate in accordance with national regulations.

As noted in Section 2.2.4, a restriction on the use of the FiCEB spreadsheet is that for 60 minutes or more fire resistance, the zone boundaries should align with the column grid and the boundary beams should be fire protected. For 30 minutes fire resistance, this restriction does not apply and the zone boundaries do not have to align with the column grid. For example, in Figure 2–8, zones A2 and A3 have columns at only two of their corners and could only be considered as design zones for a floor that requires no more than 30 minutes fire resistance.



Key to figure

A: These zones may be designed using FiCEB
A(1) Any period of fire resistance
A(2) & A(3) only 30 minutes fire resistance

B: Outside the scope of FiCEB

Figure 2–8 : Possible floor design zones

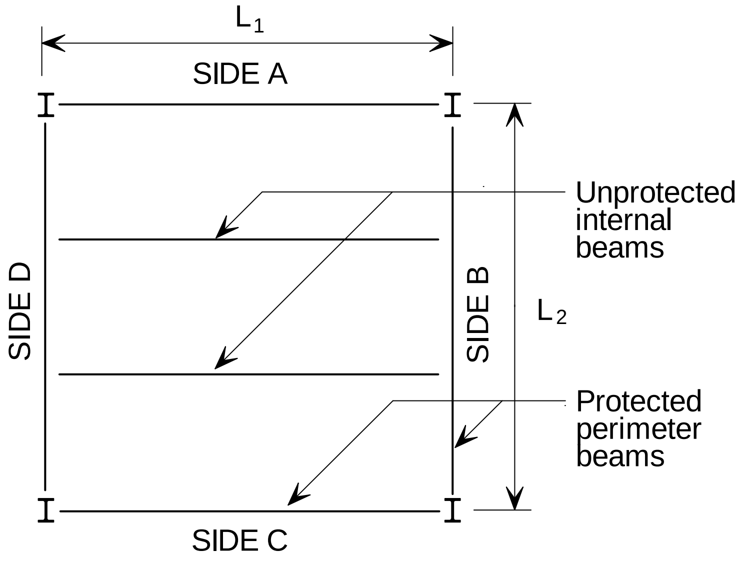


Figure 2–9 : Definition of span 1 (L1) and span 2 (L2) and the beam layout for a floor design zone in a building requiring fire resistance of 60 minutes or more.

2.3.2. Floor slab and beams

The FiCEB spreadsheet calculates the load bearing capacity of the floor slab and unprotected beams at the fire limit state. The simple design method, implemented in the software assumes that each floor design zone will have adequate support on its perimeter. This is achieved in practice by fire protecting the beams on the perimeter of each floor design zone. To ensure that adequate fire protection is provided, the software calculates the critical temperature for each perimeter beam based on the loading applied to the floor design zone.

2.3.3. Fire design of floor slab

Load bearing performance of the composite floor slab

When calculating the load bearing capacity of each floor design zone the resistance of the composite slab and the unprotected cellular beams are calculated separately. The slab is assumed to have no continuity along the perimeter of the floor design zone. The load that can be supported by the flexural behaviour of the composite slab within the floor design zone is calculated based on a lower bound mechanism assuming a yield line pattern as shown in Figure 2–10.

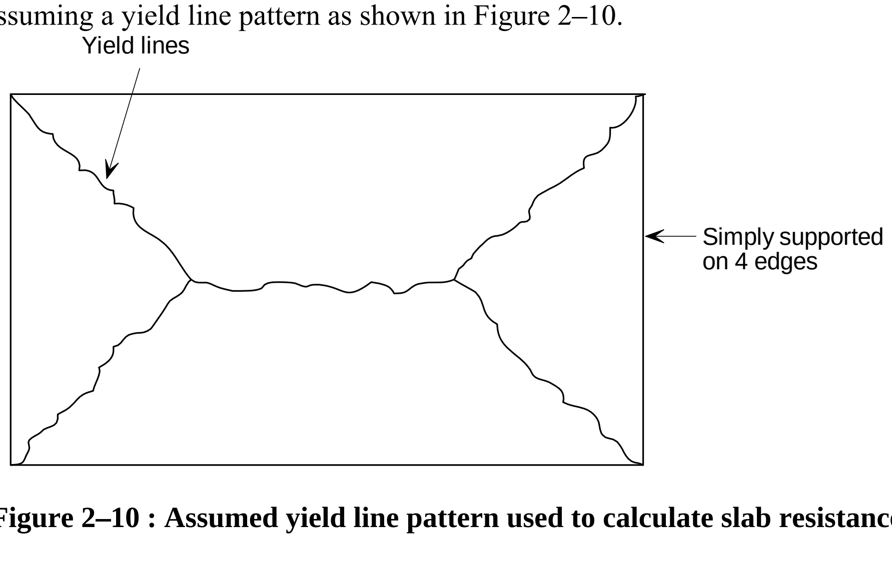


Figure 2–10 : Assumed yield line pattern used to calculate slab resistance

The value of the resistance calculated using the lower bound mechanism is enhanced by considering the beneficial effect of tensile membrane action at large displacements. This enhancement increases with increasing vertical deflection of the slab until failure occurs due to fracture of the reinforcement across the short slab span or compressive failure of the concrete in the corners of the slab, as shown by Figure 2–11. As the design method can not predict the point of failure, the value of deflection considered when calculating the enhancement is based on a conservative estimate of slab deflection that includes allowance for the thermal curvature of the slab and the strain in the reinforcement, as shown below.

$$w = \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \sqrt{\left(\frac{0.5 f_y}{E_a}\right)\frac{3L^2}{8}}$$

The deflection allowed due to elongation of the reinforcement is also limited by the following expression.

$$w \le \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \frac{l}{30}$$

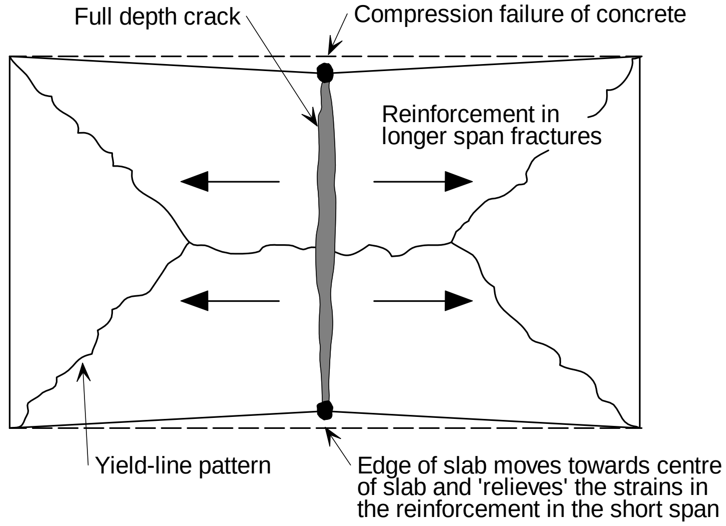
Where

(T2 – T1)	is the temperature difference between the top and bottom surface of the slab
L	is the longer dimension of the floor design zone
l	is the shorter dimension of the floor design zone
fy	is the yield strength of the mesh reinforcement
E	is the modulus of elasticity of the steel
h	is the overall depth of the composite slab
α	is the coefficient of thermal expansion of concrete

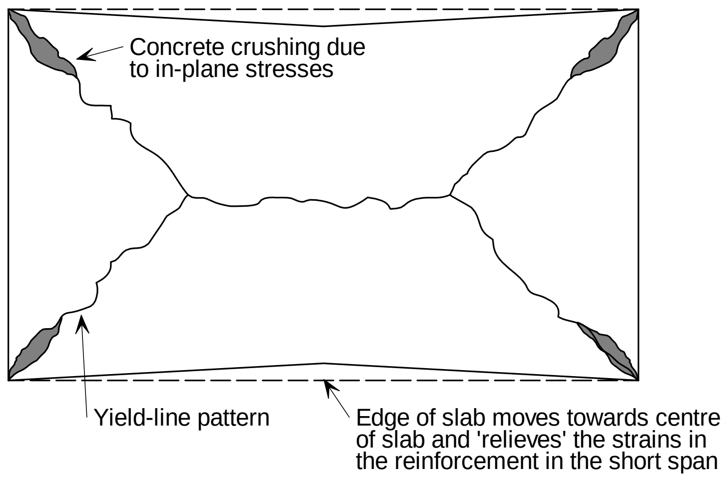
All of the available test evidence shows that this value of deflection will be exceeded before load bearing failure of the slab occurs. This implies that the resistance predicted using the design method will be conservative compared to its actual performance.

The overall deflection of the slab is also limited by the following expression.

$$w \le \frac{L + l}{30}$$



(a) Tensile failure of the reinforcement



(b) Compressive failure of the concrete

Figure 2–11 : Failure mode due to fracture of the reinforcement

The residual bending resistance of the unprotected composite beams is then added to the enhanced slab resistance to give the total resistance of the complete system.

Integrity and insulation performance of the composite slab

The FiCEB spreadsheet does not explicitly check the insulation or integrity performance of the floor slab. The designer must therefore ensure that the slab thickness chosen is sufficient to provide the necessary insulation performance in accordance with the recommendations given in EN 1994-1-2.

To ensure that the composite slab maintains its integrity during the fire and that membrane action can develop, care must be taken to ensure that the reinforcing mesh is properly lapped. This is especially important in the region of unprotected beams and around columns. Further information on required lap lengths and placement of the reinforcing mesh is given in Section 2.3.5.

2.3.4. Fire design of beams on the perimeter of the floor design zone.

The beams along the perimeter of the floor design zone, labelled A to D in Figure 2–9, should achieve the fire resistance required for the floor plate, in order to provide the required vertical support to the perimeter of the floor design zone. This usually results in these beams being fire protected.

The FiCEB spreadsheet calculates the design effect of actions on these perimeter beams and reports this in the output. In order to determine the required fire protection of these beams the room temperature moment of resistance of the beam must be calculated, in order to calculate the degree of utilisation for each perimeter beam, which is calculated using the guidance given in EN 1993-1-2 §4.2.4, as shown below.

$$\mu_0 = \frac{E_{\mathrm{fi,d}}}{R_{\mathrm{fi,d,0}}}$$

Where

$E_{\mathrm{fi,d}}$ is the design effect of actions on the beam in fire

$R_{\mathrm{fi,d,0}}$ is the design resistance of the beam at time t = 0

Having calculated the degree of utilisation, the critical temperature of the bottom flange of the perimeter beams may be calculated using cellular beam design software. This critical temperature should be use when specifying the fire protection required by each of the perimeter beams on the floor design zone.

When specifying fire protection for the perimeter beams, the fire protection supplier must be given the section factor for the member to be protected and the period of fire resistance required and the critical temperature of the member. Most reputable fire protection manufacturers will have a multi temperature assessment for their product which will have been assessed in accordance with EN 13381-4[13] for non-reactive materials or EN 13381-8[14] for reactive materials (intumescents). Design tables for fire protection which relate section factor to protection thickness are based on a single value of assessment temperature. This assessment temperature should be less than or equal to the critical temperature of the member.

2.3.5. Reinforcement details

The yield strength and ductility of the reinforcing steel material should be specified in accordance with the requirements of EN 10080. The characteristic yield strength of reinforcement to EN 10080 will be between 400 MPa and 600 MPa, depending on the national market. In order that the reinforcement has sufficient ductility to allow the development of tensile membrane action, Class B or Class C should be specified.

In most countries, national standards for the specification of reinforcement may still exist as non-contradictory complimentary information (NCCI), as a common range of steel grades have not been agreed for EN 10080.

In composite slabs, the primary function of the mesh reinforcement is to control the cracking of the concrete. Therefore the mesh reinforcement tends to be located as close as possible to the surface of the concrete while maintaining the minimum depth of concrete cover required to provide adequate durability, in accordance with EN 1992-1-1[0]. In fire conditions, the position of the mesh will affect the mesh temperature and the lever arm when calculating the bending resistance. Typically, adequate fire performance is achieved with the mesh placed between 15 mm and 45 mm below the top surface of the concrete.

Section 2.3.6 gives general information regarding reinforcement details. Further guidance and information can be obtained from, EN 1994-1-1 [7] and EN 1994-1-2[6] or any national specifications such as those given in reference [20].

2.3.6. Detailing mesh reinforcement

Typically, sheets of mesh reinforcement are 4.8 m by 2.4 m and therefore must be lapped to achieve continuity of the reinforcement. Sufficient lap lengths must therefore be specified and adequate site control must be put in place to ensure that such details are implemented on site. Recommended lap lengths are given in section 8.7.5 of EN1992-1-1[19] or can be in accordance with Figure 2–8. The minimum lap length for mesh reinforcement should be 250 mm. Ideally, mesh should be specified with 'flying ends', as shown in Figure 2–12, to eliminate build up of bars at laps. It will often be economic to order 'ready fit fabric', to reduce wastage.

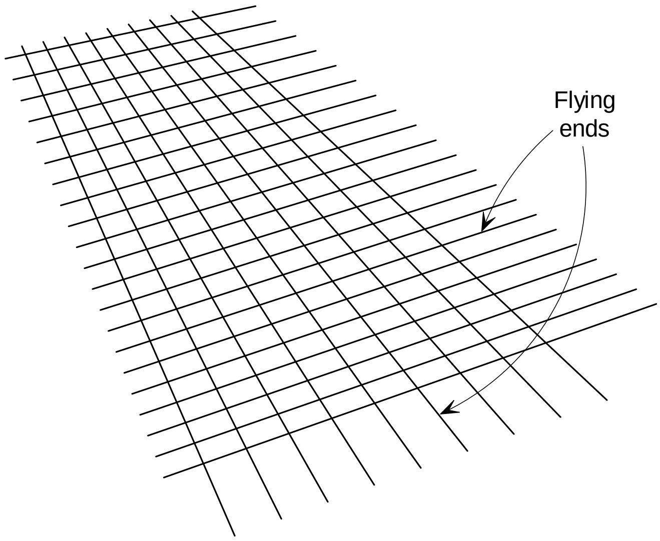


Figure 2–12 : Mesh with flying ends

Table 2-5 : Recommended tension laps and anchorage lengths for welded mesh

Reinforcement Type	Wire/Bar Type	Concrete Grade					
		LC 25/28	NC 25/30	LC 28/31	NC 28/35	LC 32/35	NC 32/40
Grade 500 Bar of diameter d	Ribbed	50d	40d	47d	38d	44d	35d
6 mm wires							
	Ribbed	300	250	300	250	275	250
7 mm wires							
	Ribbed	350	300	350	275	325	250
8 mm wires							
	Ribbed	400	325	400	325	350	300
10 mm wires							
	Ribbed	500	400	475	400	450	350

Notes:
These recommendations can be conservatively applied to design in accordance with EN 1992-1-1.
Where a lap occurs at the top of a section and the minimum cover is less than twice the size of the lapped reinforcement, the lap length should be increased by a factor of 1.4.
Ribbed Bars/Wires are defined in EN 10080
The minimum Lap/Anchorage length for bars and fabric should be 300 mm and 250 mm respectively.

2.3.6.1. Detailing requirements for the edge of a composite floor slab

The detailing of reinforcement at the edge of the composite floor slab will have a significant effect on the performance of the edge beams and the floor slab in fire conditions. The following guidance is based on the best practice recommendations for the design and construction of composite floor slabs to meet the requirements for room temperature design. The fire design method and guidance presented in this document assumes that the composite floor is constructed in accordance with these recommendations.

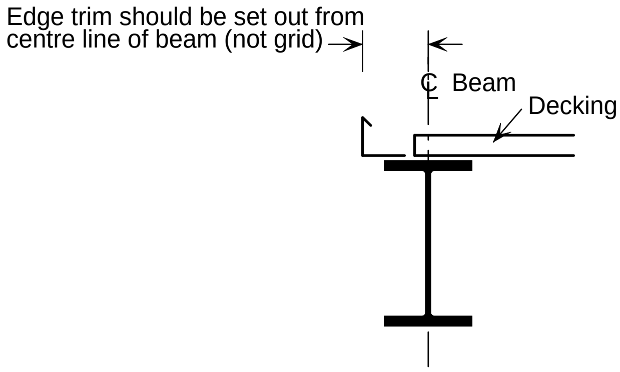
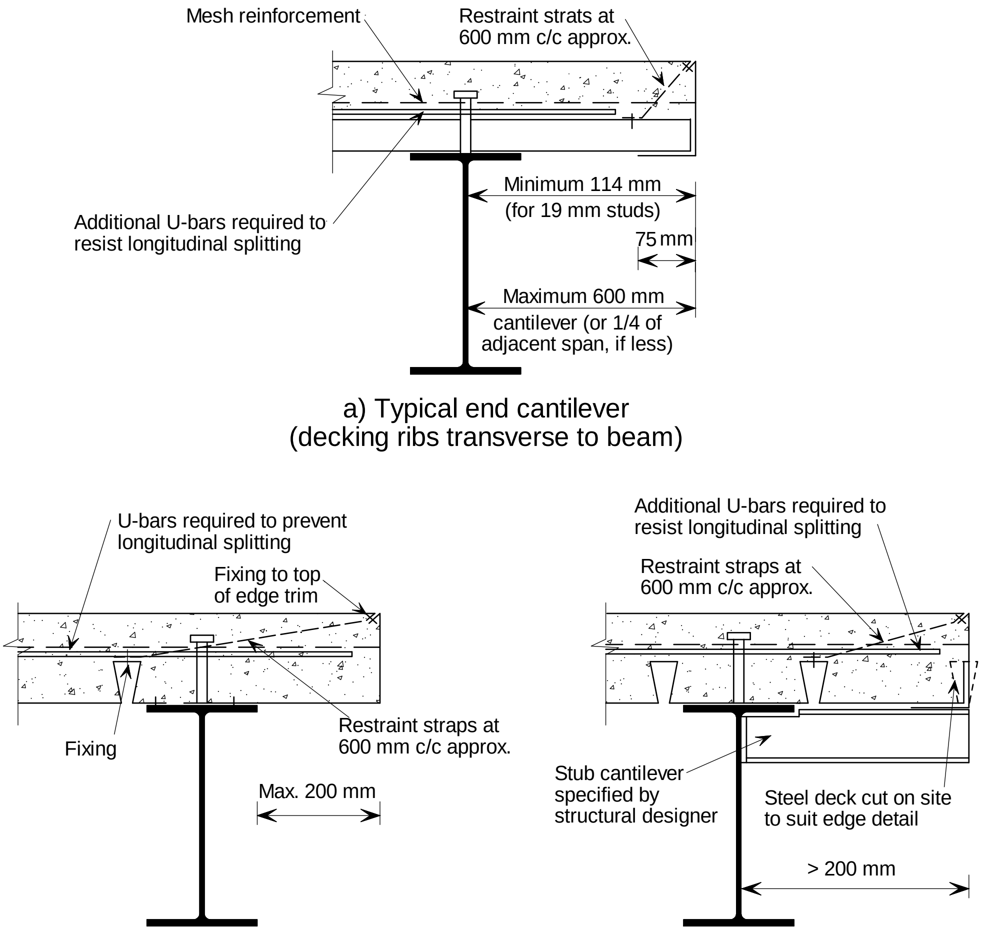


Figure 2–13 : Setting out of edge trim

The edge of the composite slab is usually formed using ‘edge trims’ made from strips of light gauge galvanized steel fixed to the beam in the same way as the decking, as shown in Figure 2–13. In cases where the edge beam is designed to act compositely with the concrete slab, U shaped reinforcing bars are required to prevent longitudinal splitting of the concrete slab. These reinforcement bars also ensure that the edge beam is adequately anchored to the slab when using this simple design method.

Some typical slab edge details covering the two deck orientations are given in Figure 2–14. Where the decking ribs run transversely over the edge beam and cantilevers out a short distance, the edge trim can be fastened in the manner suggested in Figure 2–14(a). The cantilever projection should be no more than 600 mm, depending on the depth of the slab and deck type used.

The more difficult case is where the decking ribs run parallel to the edge beam, and the finished slab is required to project a short distance, so making the longitudinal edge of the sheet unsupported Figure 2–14(b). When the slab projection is more than approximately 200 mm (depending on the specific details), the edge trim should span between stub beams attached to the edge beam, as shown in Figure 2–14(c). These stub beams are usually less than 3 m apart, and should be designed and specified by the structural designer as part of the steelwork package.’



a) Typical end cantilever
(decking ribs transverse to beam)

b) Typical edge detail
(decking ribs parallel to beam)

c) Side cantilever with stub bracket
(decking ribs parallel to beam)

Figure 2–14 : Typical edge details

2.3.7. Design of non composite edge beams

It is common practice for beams at the edge of floor slabs to be designed as non composite beams. This is because the costs of meeting the requirements for transverse shear reinforcement are more than the costs of installing a slightly heavier non composite beam. For fire design, it is important that the floor slab is adequately anchored to the edge beams, as these beams will be at the edge of floor design zones. Although not usually required for room temperature design of non composite edge beams, this guide recommends that shear connectors are provided at not more than 300 mm centres and U shaped reinforcing bars positioned around the shear connectors, as described in Section 2.3.6.1.

Edge beams often serve the dual function of supporting both the floors and the cladding. It is important that the deformation of edge beams should not affect the stability of cladding as it might increase the danger to fire fighters and others in the vicinity. (This does not refer to the hazard from falling glass that results from thermal shock, which can only be addressed by use of special materials or sprinklers.) Excessive deformation of the façade could increase the hazard, particularly when a building is tall and clad in masonry, by causing bricks to be dislodged.

2.3.8. Columns

The design guidance in this document is devised to confine structural damage and fire spread to the fire compartment itself. In order to achieve this, columns (other than those in the top storey) should be designed for the required period of fire resistance or designed to withstand the selected natural (parametric) fire.

Any applied fire protection should extend over the full height of the column, including the connection zone (see Figure 2–15). This will ensure that no local squashing of the column occurs and that structural damage is confined to one floor.

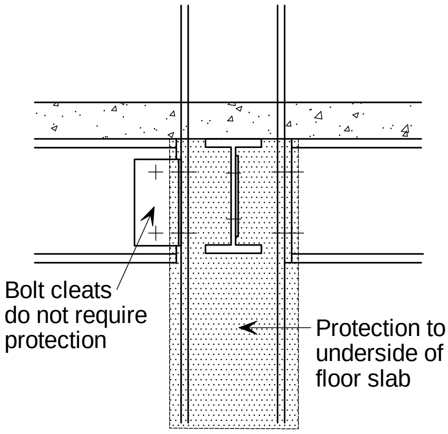


Figure 2–15 : Extent of fire protection to columns

In the Cardington fire tests, the protected columns performed well with no sign of collapse. However, subsequent finite element modelling has indicated the possibility that premature column failure could occur in some circumstances. A mode of behaviour has been identified (0) in which expansion of the floors induces moments in the columns. This can have the effect of reducing the temperature at which a column would fail.

It is recommended that, as a conservative measure, the protection to the columns at the edge of the floor plate in buildings of more than two storeys should be increased by basing its thickness on a critical temperature of 500°C, or 80ºC less than the critical temperature given in EN 1993-1-2, whichever is the lower.

For most board fire protection materials, this reduction in critical temperature will have no effect, as the minimum available thickness of board will suffice.

2.3.9. Joints

As stated in Section 2.2.3 the values given by the design method relate to ‘simple’ joints such as those with flexible end plates, fin plates and web cleats.

The steel frame building tested at Cardington contained flexible end plate and fin plate connections. Partial and full failures of some of the joints were observed during the cooling phase of the Cardington fire tests; however, no failure of the structure occurred as a result.

In the case where the plate was torn off the end of the beam, no collapse occurred because the floor slab transferred the shear to other load paths. This highlights the important role of the composite floor slab, which can be achieved with proper lapping of the reinforcement.

The resistances of the simple joints should be verified using the rules given in EN 1993-1-8[19].

2.3.9.1. Joint classification

Joint details should be such that they fulfil the assumptions made in the design model. Three joint classifications are given in EN 1993-1-8:

- Nominally pinned
  - Joints that transfer internal shear forces without transferring significant moments.
- Semi-rigid
  - Joints that do not satisfy the nominally pinned nor the rigid joint criteria.
- Rigid
  - Joints that provide full continuity.

EN 1993-1-8 §5.2 gives principles for the classification of joints based on their stiffness and strength; the rotation capacity (ductility) of the joint should also be considered.

As stated in Section 2.2.3 the values given by the simple design method have been prepared assuming the use of nominally pinned (simple) joints. To ensure that a joint does not transfer significant bending moments and so that it is a 'simple' joint it must have sufficient ductility to allow a degree of rotation. This can be achieved by detailing the joint such that it meets geometrical limits.

2.3.9.2. Fire protection

In cases where both structural elements to be connected are fire protected, the protection appropriate to each element should be applied to the parts of the plates or angles in contact with that element. If only one element requires fire protection, the plates or angles in contact with the unprotected elements may be left unprotected.

2.3.10. Overall building stability

In order to avoid sway collapse, the building should be braced by shear walls or other bracing systems. Masonry or reinforced concrete shear walls should be constructed with the appropriate fire resistance.

If bracing plays a major part in maintaining the overall stability of the building it should be protected to the appropriate standard.

In two-storey buildings, it may be possible to ensure overall stability without requiring fire resistance for all parts of the bracing system. In taller buildings, all parts of the bracing system should be appropriately fire protected.

One way in which fire resistance can be achieved without applied protection is to locate the bracing system in a protected shaft such as a stairwell, lift shaft or service core. It is important that the walls enclosing such shafts have adequate fire resistance to prevent the spread of any fire. Steel beams, columns and bracing totally contained within the shaft may be unprotected. Other steelwork supporting the walls of such shafts should have the appropriate fire resistance.

2.4. Compartmentation

National regulations require that compartment walls separating one fire compartment from another shall have stability, integrity and insulation for the required fire resistance period.

Stability is the ability of a wall not to collapse. For loadbearing walls, the loadbearing capacity must be maintained.

Integrity is the ability to resist the penetration of flames and hot gases.

Insulation is the ability to resist excessive transfer of heat from the side exposed to fire to the unexposed side.

2.4.1. Beams above fire resistant walls

When a beam is part of a fire resisting wall, the combined wall/beam separating element must have adequate insulation and integrity as well as stability. For optimum fire performance, compartment walls should, whenever possible, be located beneath and in line with beams.

Beams in the wall plane

The Cardington tests demonstrated that unprotected beams above and in the same plane as separating walls (see Figure 2–16), which are heated from one side only, do not deflect to a degree that would compromise compartment integrity, and normal movement allowances are sufficient. Insulation requirements must be fulfilled and protection for 30 or 60 minutes will be necessary; all voids and service penetrations must be fire stopped. Beams protected with intumescent coatings require additional insulation because the temperature on the non fire side is likely to exceed the limits required in the fire resistance testing standards[21,22].

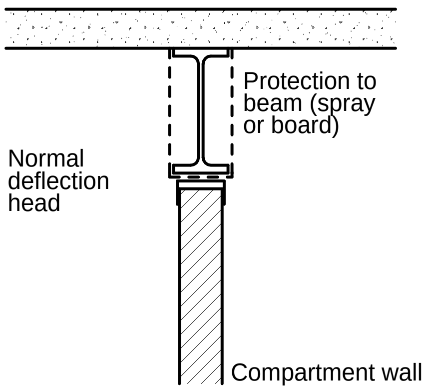


Figure 2–16 : Beams above and in line with walls

Beams through walls

The Cardington tests showed that floor stability can be maintained even when unprotected beams suffer large deflections. However, when walls are located off the column grid, large deflections of unprotected beams can compromise integrity by displacing or cracking the walls through which they pass. In such cases, the beams should either be protected or sufficient movement allowance provided. It is recommended that a deflection allowance of span/30 should be provided in walls crossing the middle half of an unprotected beam. For walls crossing the end quarters of the beam, this allowance may be reduced linearly to zero at end supports (see Figure 2–17). The compartment wall should extend to the underside of the floor.

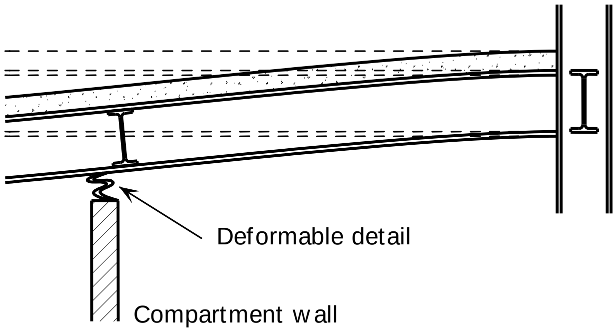


Figure 2–17 : Deformation of beams crossing walls

2.4.2. Stability

Walls that divide a storey into more than one fire compartment must be designed to accommodate expected structural movements without collapse (stability). Where beams span above and in the plane of the wall, movements, even of unprotected beams, may be small and the normal allowance for deflection should be adequate. If a wall is not located at a beam position, the floor deflection that the wall will be required to accommodate may be large. It is therefore recommended that fire compartment walls should be located at a beam positions whenever possible.

In some cases, the deflection allowance may be in the form of a sliding joint. In other cases, the potential deflection may be too large and some form of deformable blanket or curtain may be required, as illustrated in Figure 2–17.

National recommendations should be consulted for the structural deformations which should be considered when ensuring that compartmentation is maintained.

2.4.3. Integrity and insulation

Steel beams above fire compartment walls are part of the wall and are required to have the same separating characteristics as the wall. A steel beam without penetrations will have integrity. However, any service penetrations must be properly fire stopped and all voids above composite beams should also be fire stopped.

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