SCI Steel Knowledge
Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 636525
Fax: (01344) 636570
CALCULATION SHEET

1 TWO STOREY OFFICE BUILDING

1.1 Arrangement and loading

The arrangement of the steel frame for this two storey office building is shown in Figures 1.1 to 1.5. Two layouts are shown in the Figures, one arrangement with the external columns inside the cladding (‘Solution A’, shown on half of each plan and sectional elevation) and one arrangement with the columns outside the cladding (‘Solution B’).

The steelwork is to be verified designed for the actions shown in Table 1.1, using the values of partial factors given by the UK National Annex to EN 1990, as summarised in Table 1.2.

The beam and column sizes shown in the Figures were determined by considering the structure as a braced frame, comprising non-composite steel beams and columns supporting a precast concrete floor slab. The initial design was carried out at ambient temperature.

Table 1.1 Actions on the first floor

Actions on the first floor	
Permanent actions	
Precast units (floor slab)	2.9 kN/m^2
Screed	1.2 kN/m^2
Self-weight of floor beam	0.3 kN/m^2
Ceiling and services	0.6 kN/m^2
Variable actions	
Occupancy load	2.5 kN/m^2
Partitions	0.8 kN/m^2

Table 1.2 Partial factors on actions

Factor	Value
γ_G	1.35
γ_Q	1.50

1.2 Verification at elevated temperature

This worked example demonstrates the verification at elevated temperature of a beam, a column and a beam to column connection.

Two verifications of the beam and column are demonstrated:

- Unprotected.
- Protected with board.

The verifications follow a simplified calculation model, as permitted by clause 4.1(2) of EN 1993-1-2.

The verifications use the standard temperature-time curve given in EN 1991-1-2 clause 3.2.1 (1).

A complementary worked example demonstrates the use of an advanced calculation model to verify an unprotected external column.

1.2.1 Verification process

The verification process for beams may be summarised as:

- Under fire conditions, a reduced design value of actions is calculated.
- The critical temperature for the steel member under the reduced design value of actions is determined.
- The time taken for the steel to reach this critical temperature is determined using an incremental process, as the steel temperature depends on the gas temperature (which itself depends on time) and the transfer of heat to the steel (the convective and radiative heat flux).

The verification process for columns is similar to that for beams:

- Under fire conditions, a reduced design value of actions is calculated.
- A reduced buckling length may be assumed, depending on the storey under consideration and the compartmentalisation in the building.
- The reduced design resistance is calculated at elevated temperature, allowing for changes in material strength and properties as necessary.
- The time taken for the design resistance to drop below the design load is calculated using an incremental process, as the steel temperature and properties depend on the gas temperature (which itself is depends on time) and the transfer of heat to the steel.

The verification process for a connection may be summarised as:

- Under fire conditions, a reduced design value of actions (in this case, the shear force on the connection) is calculated.
- The temperature of each connection component is calculated, based on the distance of the component from the bottom (hot) flange.
- Reduction factors are determined for the bolts, welds and other connection components, based on the calculated temperature.
- The design resistance of the connection is determined from the (reduced) design resistance of each of the connection components.

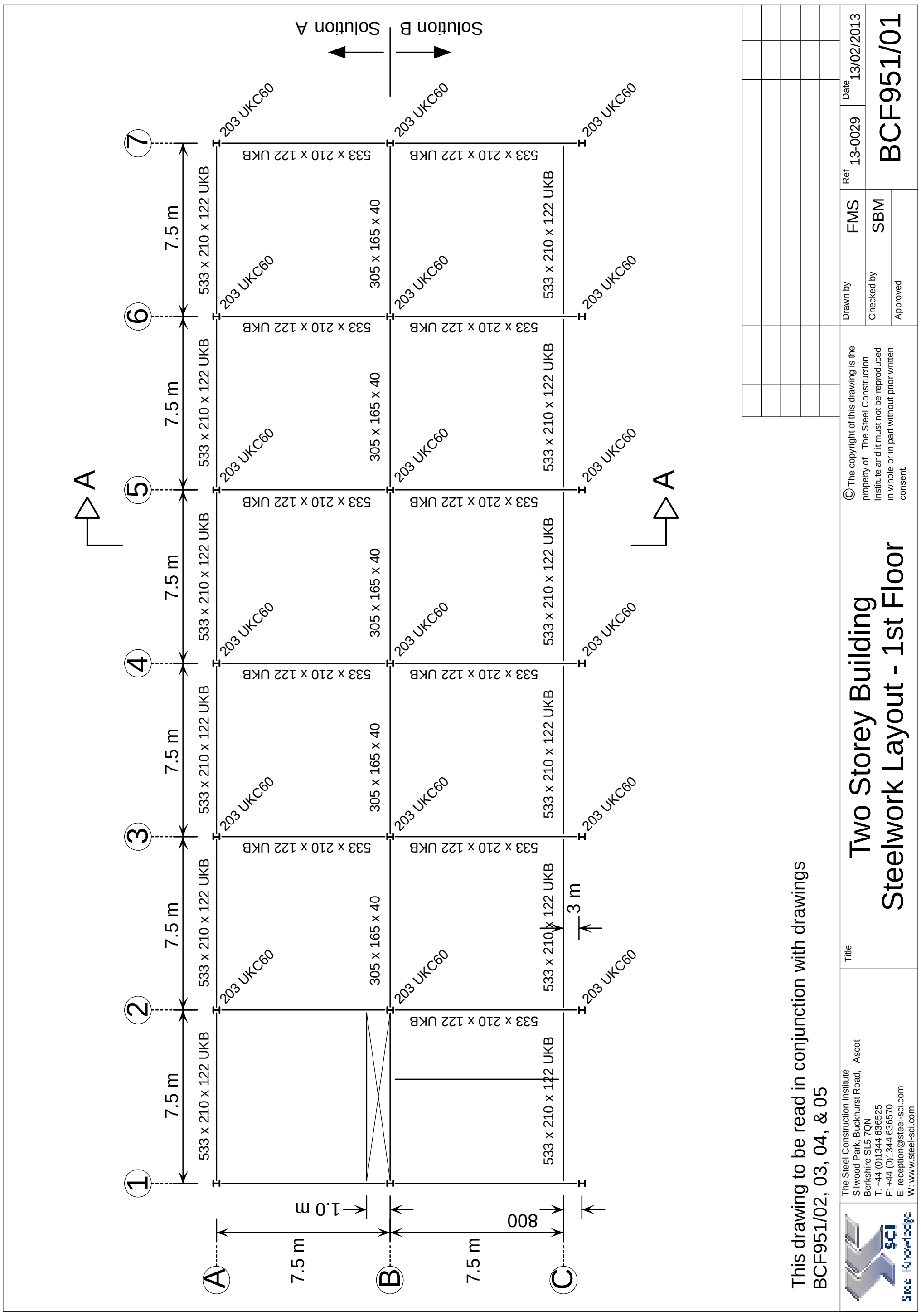

Figure 1.1 Steelwork Layout of the First Floor

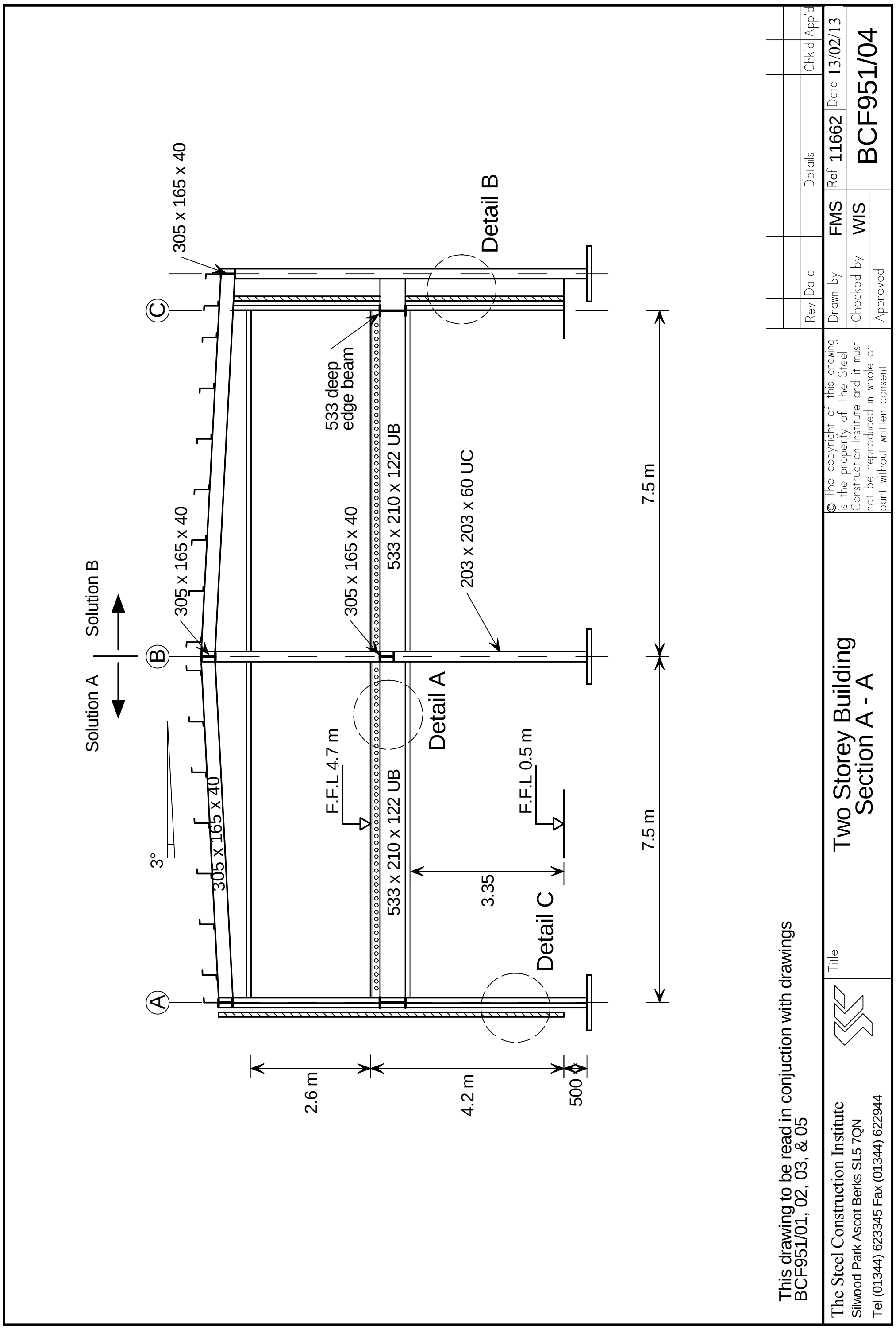

Figure 1.2 Section A-A

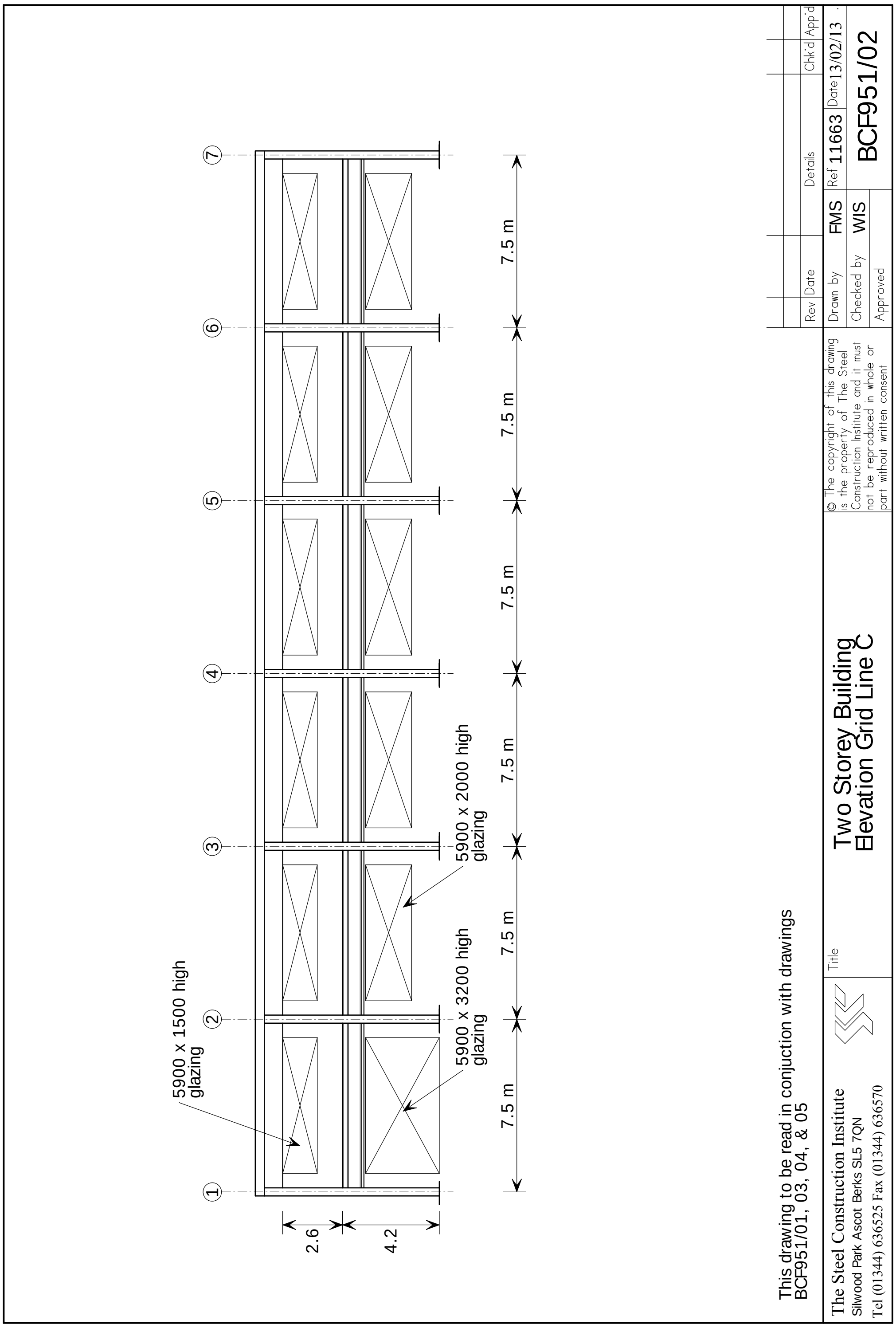

Figure 1.3 Elevation Gridline C

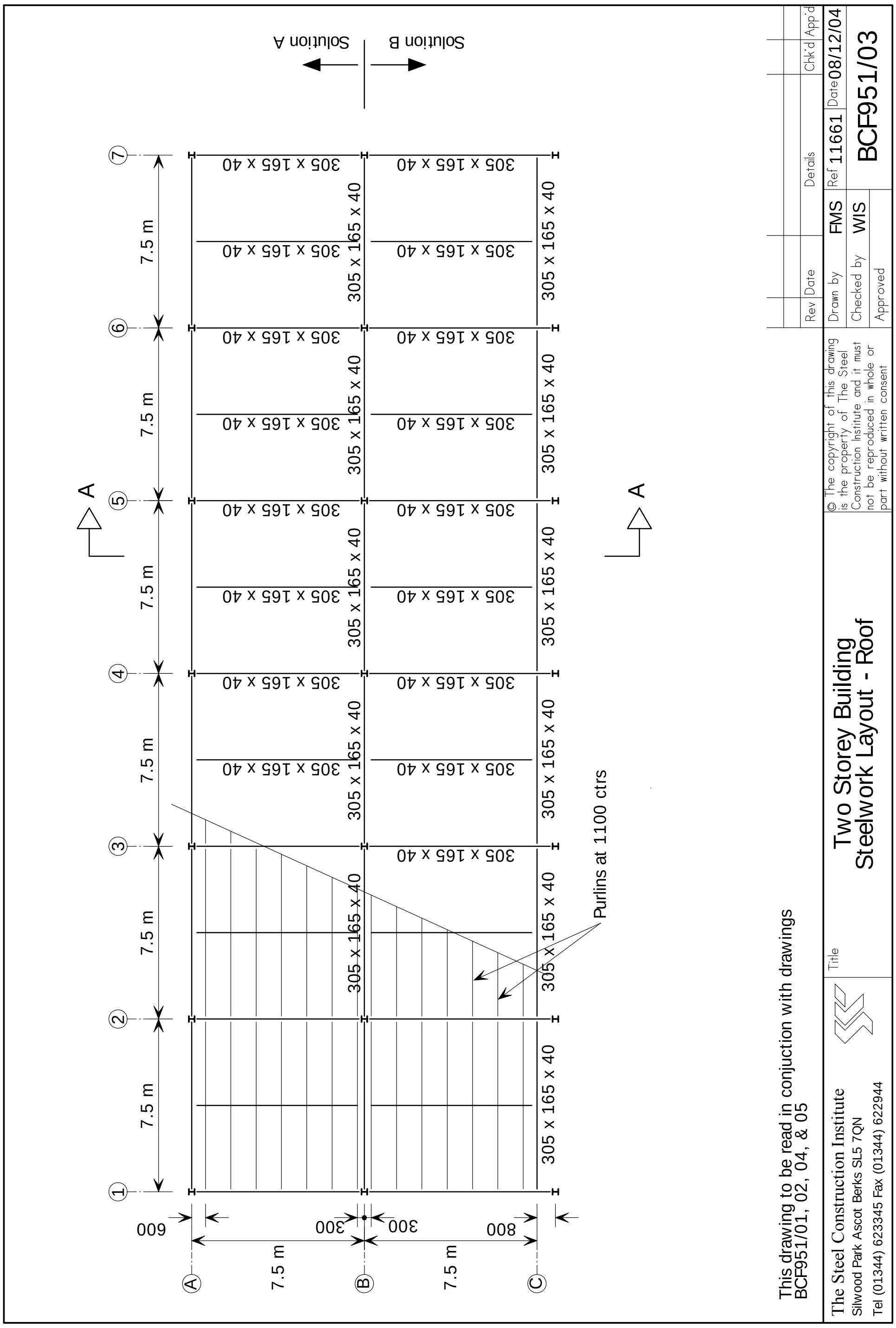

Figure 1.4 Layout of Steelwork- Roof

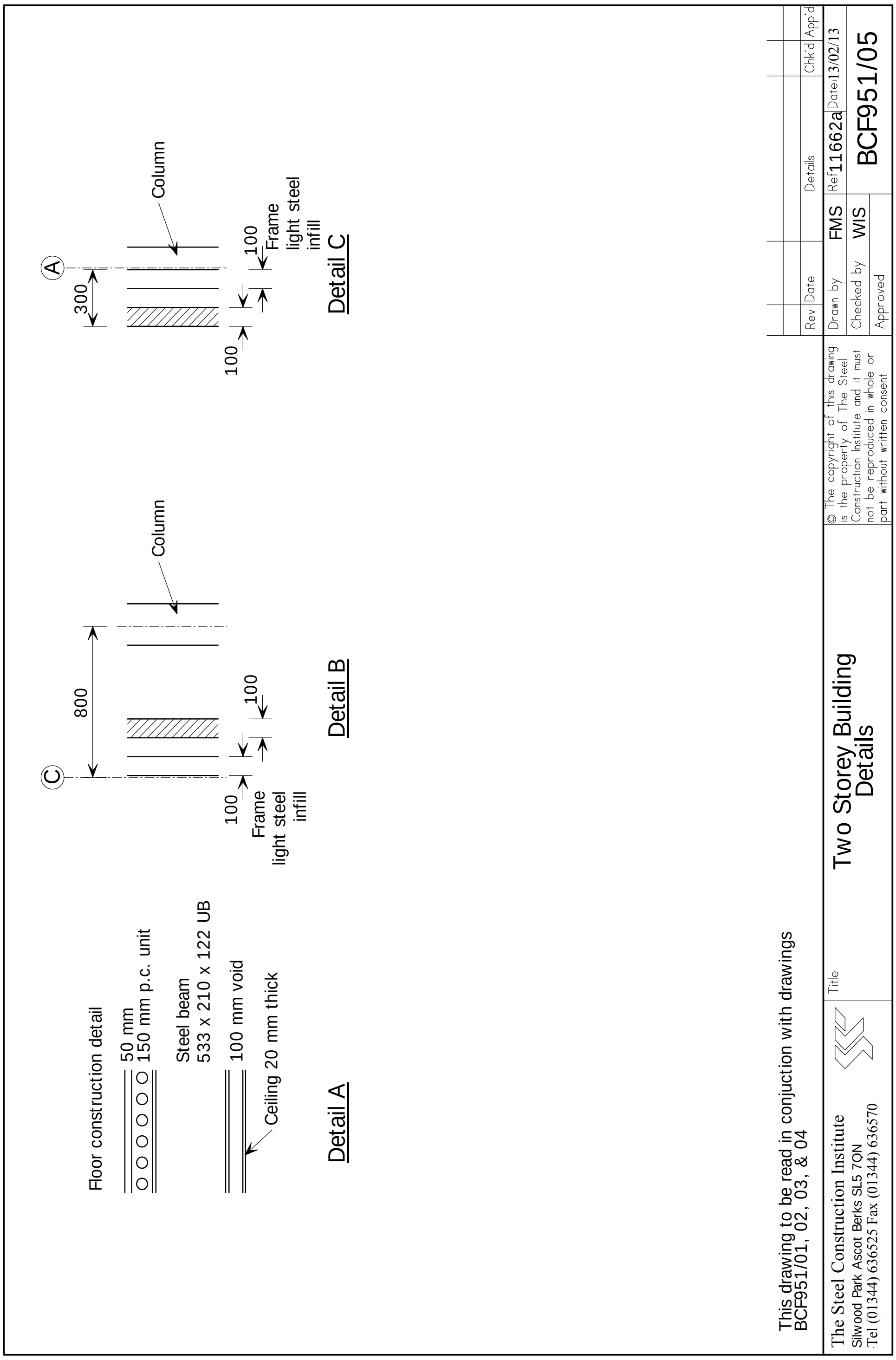

Figure 1.5 Floor and Cladding Detail

1.3 Structural fire design

The building is a two-storey office building. The floor of the top storey is 4.2 m above ground level. According to the Table A2 of Approved Document B, a 30 minute minimum fire resistance is required.

App Doc. B

1.4 Fire resistance of first floor beam

1.4.1 Normal temperature

Beam size 533 × 210 × 122 UKB, S275

Beam spacing 7.5 m

Beam span 7.5 m

Permanent action on first floor:

G_k $= 2.9 + 1.2 + 0.3 + 0.6 = 5.0$ kN/m^2

Variable action on first floor:

$Q_{k,1}$ $= 2.5 + 0.8 = 3.3$ kN/m^2

The design combination value of actions, using expression 6.10 from EN 1990 is given by:

EN 1990

f_d $= 1.35 \times 5.0 + 1.5 \times 3.3 = 11.7$ kN/m^2

Note that according to the UK National Annex to EN 1990, the design combination value of actions could be calculated as the most onerous value from expression 6.10a and 6.10b. In this instance, 6.10b is critical, resulting in a design value of 11.2 kN/m^2. If the pair of expression 6.10a and 6.10b are considered, the reduction factor, calculated in Section 1.4.2 below is also modified.

Design bending moment

$$M_{Ed} = \frac{f_d L^2}{8} = \frac{(11.7 \times 7.5) \times 7.5^2}{8} = 617 \text{ kNm}$$

Bending resistance of UKB section

From the 'Blue Book':

P363

$M_{c,y,Rd}$ $= 847$ kNm

Note that P363 also indicates that at ambient temperature, the beam is Class 1.

1.4.2 Design loading in fire:

Design moment at the fire limit state

$M_{fi,d}$ $= \eta_{fi}\, M_{Ed}$

EN 1993-1-2 2.4.2(2)

The reduction factor for design load level in the fire situation is given by:

EN 1993-1-2 2.4.2(2) Expression 2.5

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

ψ_{fi} is to be taken as $\psi_{1.1}$ according to the UK NA to EN 1991-1-2

The value of $\psi_{1.1}$ is taken from the UK NA to EN 1990, for (in this instance), office areas.

UK NA to EN 1991-1-2 NA.2.7

UK NA to EN 1990 Table NA.A1.1

$\psi_{fi} = \psi_{1,1} = 0.5$

$$\eta_{fi} = \frac{5 + 0.5 \times 3.3}{1.35 \times 5 + 1.5 \times 3.3} = 0.57$$

$M_{fi,d} = 0.57 \times 617 = 352$ kNm

If the pair of expression 6.10a and 6.10b had been used to calculate the design combination value of actions, the reduction factor is given by the smaller value as determined from expression 2.5a and 2.5b of EN 1993-1-2.

In this instance, expression 2.5b gives the lower value, which is 0.59.

This worked example continues to use expression 6.10 of EN 1990 to calculate the design value of actions and expression 2.5 of EN 1991-1-2 to calculate the reduction factor in the fire situation.

1.5 Design resistance of unprotected beam in fire

For simplicity, the resistance of a structural section may be verified in the temperature domain.

EN 1993-1-2 4.2.4

Except when deformation criteria or when stability phenomena have to be taken into account, the critical temperature ($\theta_{a,cr}$) of carbon structural steel may be calculated as:

EN 1993-1-2 4.2.4(2)

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

where the degree of utilisation, μ_0, is given by:

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}} \text{ but not less than } 0.013$$

where:

$E_{fi,d}$ is the design effect of actions for the fire design situation (in this case the reduced bending moment), and

$R_{fi,d,0}$ is the corresponding design resistance at time $t = 0$ (i.e. the design bending resistance before any heating)

Thus, when considering bending resistance:

$$\mu_0 = \frac{M_{fi,d}}{M_{fi,t,Rd}}$$

1.5.1 Section classification

Although the beam is a Class 1 section at normal temperature, the classification at elevated temperature may differ, as according to clause 4.2.2 of EN 1993-1-2,

$$\varepsilon = 0.85\sqrt{\frac{235}{f_y}}$$

Thus $\varepsilon = 0.85\sqrt{\frac{235}{f_y}} = 0.85\sqrt{\frac{235}{265}} = 0.8$ ($f_y = 265$ N/mm^2, since $16 < t_f < 40$)

For the web;

c/t for a 533 × 210 × 122 UKB = 37.5 — P363

Class 1 limiting value = $72\varepsilon = 72 \times 0.8 = 57.6$ — EN 1993-1-1 Table 5.2

Thus the web is Class 1

For the flange;

c/t for a 533 × 210 × 122 UKB = 4.08 — P363

Class 1 limiting value = $9\varepsilon = 9 \times 0.8 = 7.2$ — EN 1993-1-1 Table 5.2

Thus the flange is Class 1

Therefore, even at elevated temperature, the beam is Class 1

As the beam is a Class 1 section with a non-uniform temperature distribution, $M_{fi,t,Rd}$, the design moment resistance at time t, may be determined from: — EN 1993-1-2 4.2.3.3 (3)

$$M_{fi,t,Rd} = \frac{M_{fi,\theta,Rd}}{\kappa_1\kappa_2}$$

in which:

$M_{fi,\theta,Rd}$ is the design moment resistance of the cross section for a uniform temperature — EN 1993-1-2 4.2.3.3(1)

$= k_{y,\theta}\,[\gamma_{M0}/\gamma_{Mfi}]M_{Rd}$

where:

$k_{y,\theta}$ is the reduction factor for effective yield strength from Table 3.1 of EN 1993-1-2

κ_1 is an adaption factor for non-uniform temperature across the cross-section — EN 1993-1-2 4.2.3.3(3)

κ_2 is an adaption factor for non-uniform temperature along the beam

At time $t = 0$ (ambient temperature design): $\theta = 20^\circ$C, $k_{y,\theta} = 1.00$ — EN 1993-1-2 Table 3.1

The material partial factor at ambient temperature is: — UK NA to EN 1993-1-1 NA.2.15

$\gamma_{M0} = 1.00$

The UK National Annex to EN 1993-1-2 suggests the use of the value for partial factors for materials at elevated temperature recommended in EN 1993-1-2 clause 2.3. Therefore: — NA to EN 1993-1-2 NA.2.3

The material partial factor at elevated temperature is: EN 1993-1-2 2.3(2)

$\gamma_{M.fi}$ = 1.00

Hence:

$M_{fi,\theta,Rd}$ = 1.00 [1.00/1.00] × 847 = 847 kNm

For an unprotected beam supporting a concrete slab

κ_1 = 0.7 (unprotected beam with concrete slab) EN 1993-1-2 4.2.3.3 (7) & (8)

κ_2 = 1.0

Therefore, at time $t = 0$

$$M_{fi,t,Rd} = \frac{847}{0.7 \times 1.0} = 1210 \text{ kNm}$$

$$\mu_0 = \frac{M_{fi,d}}{M_{fi,t,Rd}} = \frac{352}{1210} = 0.29$$

Therefore:

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0.9674 \times 0.29^{3.833}} - 1\right] + 482$$

$\theta_{a,cr}$ = 669°C

The time taken for the steel beam temperature to reach its critical value is calculated using an incremental process given in expression 4.25 of EN 1993-1-2 EN 1993-1-2 4.2.5.1

The increase in temperature of the steel beam during a time interval Δt is given by:

$$\Delta\theta_{a,t} = k_{sh} \frac{A_m / V}{c_a \rho_a} \dot{h}_{net} \; \Delta t$$ Expression 4.25

where: EN 1993-1-2 3.2.2(1)

ρ_a is the density of steel = 7850 kg/m^3

c_a is the specific heat of steel, given by EN 1991-1-2 Clause 3.4.1.2

k_{sh} is the correction factor for the shadow effect = $0.9[A_m/V]_b/[A_m/V]$: EN 1993-1-2 4.2.5.1

k_{sh} may conservatively be taken as 1 – see NOTE 2 of 4.2.5.1(2)

The design value of net heat flux per unit area, $\dot{h}_{net,d}$, is given by:

$$\dot{h}_{net,d} = \dot{h}_{net,c} + \dot{h}_{net,r}$$ EN 1991-1-2 3.1

The net radiative heat flux, $\dot{h}_{net,r}$, is given by:

$$\dot{h}_{net,r} = \Phi \, \varepsilon_m \, \varepsilon_f \, \sigma [(\theta_r + 273)^4 - (\theta_m + 273)^4]$$

The net convective heat flux, $\dot{h}_{net,c}$, is given by:

$$\dot{h}_{net,c} = \alpha_c (\theta_g - \theta_m)$$

where:

Φ = 1.0 (EN 1991-1-2 clause 3.1(7)

ε_m = 0.7 (EN 1993-1-2 clause 2.2)

ε_f = 1.0 (EN 1993-1-2 clause 4.2.5.1(3)

α_c is the coefficient of heat transfer by convection

θ_g is the gas temperature in the vicinity of the fire exposed member (see below)

θ_m is the surface temperature of the steel member

σ is the Stephan Boltzmann constant (= 5.67×10^{-8} W/m^{2}K^4) (EN 1993-1-2 clause 3.1(6))

The temperature-time curve is given by:

θ_g = $20 + 345 \log_{10} (8t + 1)$

EN 1991-1-2 3.2.1(1)

where :

t is time (in minutes)

For the beam selected and the exposure considered (3-sided) the value of $[A_m/V]$ is:

$$\left[\frac{A_m}{V}\right] = \frac{1890 - 211.9}{15.5} = 108 \text{ m}^{-1}$$

EN 1993-1-2 4.2.5 & P363

Note that the above calculation assumes the use of precast concrete planks, which protect the top surface of the beam.

As the radiative heat flux ($\dot{h}_{net,c}$) is a function of the surface temperature of the beam (θ_m), which is dependant on the gas temperature and time, a spreadsheet may be used to calculate the steel temperature at intervals (Δt).

According to EN 1993-1-2 clause 4.2.5.1(4) Δt should not be taken as more than 5 seconds.

A design tool to undertake this verification is available at www.steelconstruction.info

If k_{sh} is conservatively taken as 1.0, the incremental calculation of the unprotected steel temperature shows that the beam reaches its critical temperature of 669°C after 18.8 minutes.

If k_{sh} is calculated (the value is 0.698), the incremental calculation of the unprotected steel temperature shows that the beam reaches its critical temperature of 669°C after 22.7 minutes.

Because the critical temperature is reached at 22.7 minutes, earlier than the required 30 minutes of fire resistance, an unprotected solution is unsatisfactory.

Figure 1.6 shows the variation of the temperature of gas and steel temperature with time, showing when the critical temperature is reached.

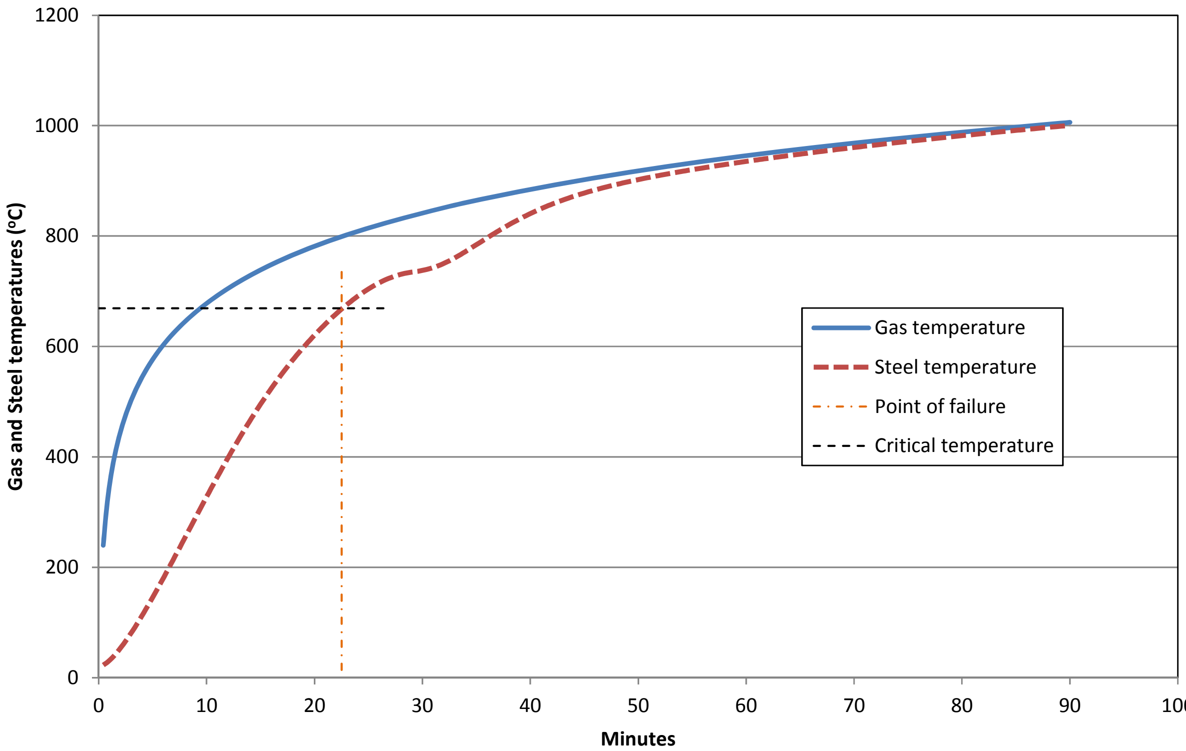

Figure 1.6 Variation of gas and steel temperatures with time – unprotected beam

1.6 Design resistance of protected beam in fire

Try encasing the beam with 10 mm fire protection board.

The temperature increase in a protected member in time interval Δt is given by:

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p / V}{d_p c_a \rho_a} \frac{(\theta_{g,t} - \theta_{a,t})}{1 + \phi/3} \Delta t - \left(e^{\phi/10} - 1\right) \Delta\theta_{g,t}$$

EN 1993-1-2 4.2.5.2 Expression 4.27

where:

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

where:

EN 1993-1-2 4.2.5.2

λ_p is the thermal conductivity of the fire protection system

A_p is the appropriate area of fire protection per unit length of the member

d_p is the thickness of the fire protection material (in m)

c_p is the temperature independent specific heat of the fire protection material

ρ_p is the unit mass of the fire protection material

$\theta_{g,t}$ is the gas temperature at time t

$\theta_{a,t}$ is the steel temperature at time t

$\Delta\theta_{g,t}$ is the increase of the ambient gas temperature during the time interval Δt

(other nomenclature as previously defined)

λ_p, c_p, and ρ_p are taken from the manufacturer's data.

For this fire protection board selected, the manufacturer provided the following data:

Thermal conductivity	λ_p	= 0.2 W/mK
Thickness	d_p	= 10 mm
Density	ρ_p	= 800 kg/m^3
Specific heat	c_p	= 1700 J/kgK

$$\frac{A_p}{V} = \frac{\text{internal surface area of boarding}}{\text{volume of member}} = \frac{211.9 + 2 \times 544.5}{15.5} = 84 \text{ m}^{-1}$$

EN 1993-1-2
4.2.5.2(4)

For a protected beam supporting a concrete slab: κ_1 = 0.85

EN 1993-1-2
4.2.3.3 (7) & (8)

κ_2 = 1.0

Therefore, at time $t = 0$

$$M_{\text{fi,t,Rd}} = \frac{M_{\text{fi},\theta,\text{Rd}}}{\kappa_1 \kappa_2} = \frac{847}{0.85 \times 1.0} = 996 \text{ kNm}$$

$$\mu_0 = \frac{M_{\text{fi,d}}}{M_{\text{fi,t,Rd}}} = \frac{352}{996} = 0.35$$

Therefore, the critical temperature of the protected beam is:

$$\theta_{\text{a,cr,p}} = 39.19 \ln \left[\frac{1}{0.9674 \times 0.35^{3.833}} - 1 \right] + 482$$

$$\theta_{\text{a,cr,p}} = 639^\circ\text{C}$$

An incremental procedure must be used to determine the gas temperature at time t and therefore the temperature of the steel. When undertaking the incremental process, Δt should not be taken as more than 30 seconds. In this example, a spreadsheet has been used to calculate the gas and steel temperatures as they vary with time. Δt has been taken as 5 seconds.

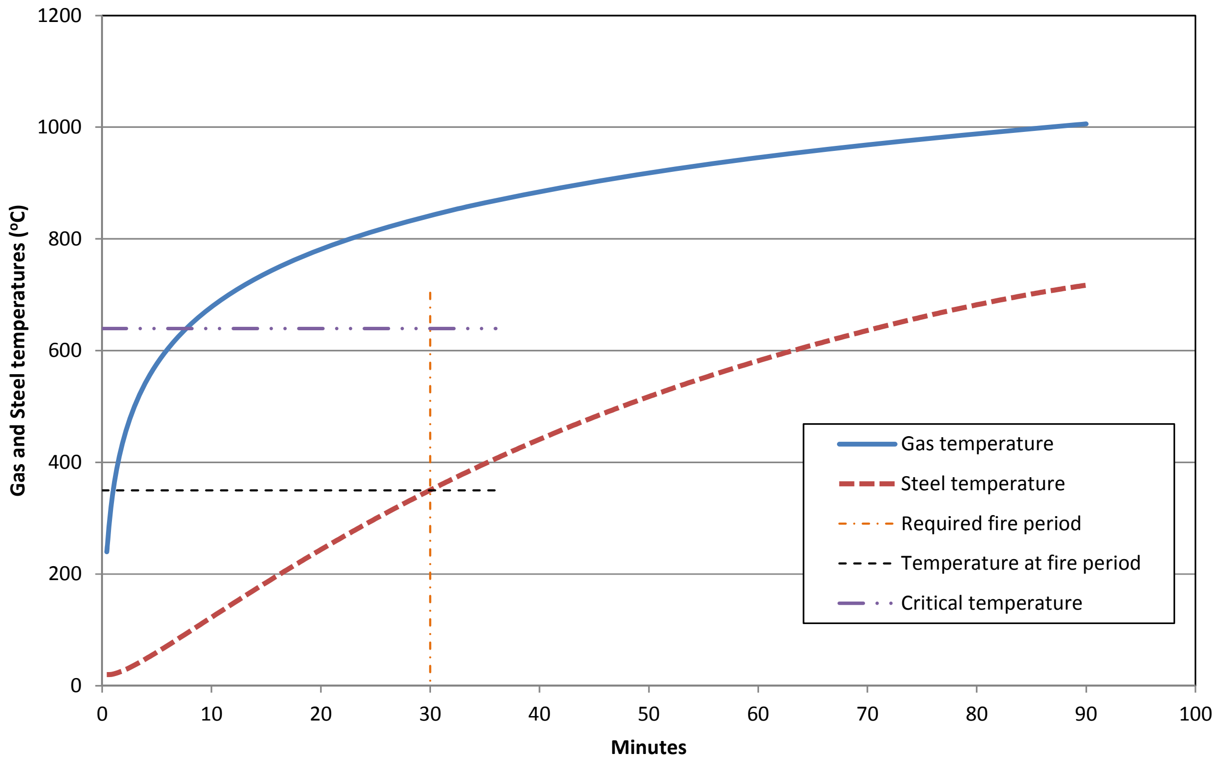

Figure 1.7 Variation of gas and steel temperatures with time – protected beam

The incremental calculation demonstrates that at the required fire resistance period of 30 minutes, the steel temperature is 350°C. This is less than the critical temperature of 639°C, and thus the protection selected is satisfactory. The salient points are highlighted in Figure 1.7.

Figure 1.7 illustrates that the selected solution would also be satisfactory for a fire resistance period of 60 minutes, as the steel temperature is 582°C, less than the critical temperature.

1.7 Fire resistance of a column (ground to first floor)

This section of the example demonstrates the verification of an internal column (203 × 203 × 60 UKC S355) at the lower level, in accordance with the simplified calculation model described in EN 1993-1-2. The column is firstly considered unprotected, and then with the addition of board protection.

1.7.1 Verification at normal temperature

Note that according to the UK National Annex to EN 1990, the design combination value of actions could be calculated as the most onerous value from expression 6.10a and 6.10b. If the pair of expression 6.10a and 6.10b are considered, the reduction factor, calculated in Section 1.7.2 below is also modified.

Axial force due to permanent actions, G_k = 327 kN

Axial force due to variable actions, Q_k = 219 kN

Design combination value of actions, using expression 6.10 from EN 1990, is given by:

EN 1990 Partial factors from UK NA to EN 1990

$N_{\mathrm{Ed}} = 1.35 \times 327 + 1.5 \times 219 = 770$ kN

The chosen column section, a 203 × 203 × 60 UKC S355 is at least a Class 2 Section at ambient temperature.

P363

This may be verified by inspecting the "n limit" given on page D-204 of P363. The selected column section is at least Class 2 at all levels of axial load, and therefore the resistance is based on the gross area.

Design resistance of the cross-section:

EN 1993-1-1 6.2.4
P363

$N_{\mathrm{c,Rd}} = N_{\mathrm{pl,Rd}} = Af_{\mathrm{y}} / \gamma_{\mathrm{M0}}$

$= 2710$ kN $> N_{\mathrm{Ed}}$ therefore OK

Design buckling resistance of the cross-section:

EN 1993-1-1 6.3.1.1

$N_{\mathrm{b,Rd}} = \chi A\, f_{\mathrm{y}} / \gamma_{\mathrm{M1}}$

From sheets 4 and 7, the length of the bottom storey column is estimated to be 4.2 + 0.5 – 0.15 – 0.05 – (0.5 ×0.533) = 4.23 m, so say 4.25 m. Assuming a buckling length of 1.0 × system length, the buckling resistance (interpolated from P363) is 1350 kN

P363

$N_{\mathrm{b,Rd}} = 1350$ kN $> N_{\mathrm{Ed}}$ therefore OK

1.7.2 Design loading at elevated temperature

Design compression force at elevated temperature, $N_{\mathrm{fi,Ed}}$ is given by:

EN 1993-1-2 2.4.2(2)

$N_{\mathrm{fi,Ed}} = \eta_{\mathrm{fi}}\, \mathrm{N_{Ed}}$

The reduction factor for design load level in the fire situation is given by:

EN 1993-1-2 2.4.2(3) Expression 2.5

$$\eta_{\mathrm{fi}} = \frac{G_{\mathrm{k}} + \psi_{\mathrm{fi}} Q_{\mathrm{k,1}}}{\gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}}}$$

ψ_{fi} is to be taken as $\psi_{1.1}$ according to the UK NA to EN 1991-1-2

UK NA to EN 1991-1-2 NA.2.7
UK NA to EN 1990 Table NA.A1.1

The value of $\psi_{1.1}$ is taken from the UK NA to EN 1990, for (in this instance), office areas.

$\psi_{\mathrm{fi}} = \psi_{1,1} = 0.5$

$$\eta_{\mathrm{fi}} = \frac{327 + 0.5 \times 219}{1.35 \times 327 + 1.5 \times 219} = 0.57$$

Hence:

$N_{\mathrm{fi,Ed}} = 0.57 \times N_{\mathrm{Ed}} = 0.57 \times 770 = 437$ kN

1.8 Design buckling resistance of unprotected column at elevated temperature

The design buckling resistance in fire is given by:

EN 1993-1-2 4.2.3.2(1)

$N_{\mathrm{b,fi,t,Rd}} = \chi_{\mathrm{fi}}\, A\, k_{\mathrm{y},\theta}\, f_{\mathrm{y}} / \gamma_{\mathrm{Mfi}}$

The UK National Annex to EN 1993-1-2 suggests the use of the value for partial factors for materials at elevated temperature recommended in EN 1993-1-2 clause 2.3. Therefore:

NA to EN 1993-1-2 NA.2.3

γ_{Mfi} = 1.0

EN 1993-1-2 2.3(2)

f_y = 355 N/mm^2

$k_{y,\theta}$ is the reduction factor for effective yield strength from Table 3.1 of EN 1993-1-2

EN 1993-1-2 Table 3.1

The area to be used in the preceding calculation depends on the section classification, which may vary at elevated temperature.

1.8.1 Section classification

Although the column is at least Class 2 at normal temperature, the classification at elevated temperature may differ, as according to clause 4.2.2 of EN 1993-1-2,

$$\varepsilon = 0.85\sqrt{\frac{235}{f_y}}$$

Thus $\varepsilon = 0.85\sqrt{\frac{235}{f_y}} = 0.85\sqrt{\frac{235}{355}} = 0.69$ ($f_y = 355$ N/mm^2, since $t_f < 16$)

For the web;

c/t for a 203 × 203 × 60 UKC = 17.1 P363

Class 1 limiting value = $33\varepsilon = 33 \times 0.69 = 22.7$ EN 1993-1-1 Table 5.2

Thus the web is Class 1

For the flange;

c/t for a 203 × 203 × 60 UKC = 6.2 P363

Class 1 limiting value = $9\varepsilon = 9 \times 0.69 = 6.21$ EN 1993-1-1 Table 5.2

Thus the flange is Class 2

Therefore, at elevated temperature, the column is Class 2.

For a Class 2 section, the gross area is used in design.

A = 7640 mm^2 P363

1.8.2 Column slenderness

For intermediate storeys of a braced frame with separate fire compartments, the buckling length may be taken as $l_{\mathrm{fi}} = 0.5L$. Therefore:

EN 1993-1-2 4.2.3.2(5)

Buckling length, $L_{\mathrm{cr}} = L_{\mathrm{fi}} = 0.5 \times 4250 = 2125$ mm

The non-dimensional slenderness (at ambient temperature)

EN 1993-1-1 6.3.1.3

$$\bar{\lambda} = \frac{L_{\mathrm{cr}}}{i}\frac{1}{\lambda_1}$$

where:

$\lambda_1 = 93.9\varepsilon = 93.9\sqrt{235/355} = 76.4$

For this UKC section, the radius of gyration is i_z = 52 mm — P363

$$\bar{\lambda} = \frac{2125}{52} \times \frac{1}{76.4} = 0.535$$

The non-dimensional slenderness at an elevated steel temperature, $\bar{\lambda}_\theta$ is given by: — EN 1993-1-2 4.2.3.2(2)

$$\bar{\lambda}_\theta = \bar{\lambda}\left[k_{y,\theta} / k_{E,\theta}\right]^{0.5}$$

Reduction factor χ_{fi}

The reduction factor for buckling at elevated temperature, χ_{fi}, is given by:

$\chi_{fi} = \min(\chi_{y,fi}, \chi_{z,fi})$

For this section, with the same buckling length and restraint conditions in both axis, it is clear by inspection that buckling in the minor axis will be critical and only this axis needs to be considered. To determine the reduction factor: — EN 1993-1-2 4.2.3.2

1. The non-dimensional slenderness $\bar{\lambda}$ is calculated (at ambient temperature).
2. The reduction factors $k_{y,\theta}$ and $k_{E,\theta}$ are determined from Table 3.1 of EN 1993-1-2.
3. The non-dimensional slenderness at elevated temperature is calculated: $\bar{\lambda}_{z,\theta} = \bar{\lambda}_z\left[k_{y,\theta} / k_{E,\theta}\right]^{0.5}$
4. ϕ_θ is calculated, given by: $\phi_\theta = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right]$ where $\alpha = 0.65\sqrt{235/f_y}$
5. The reduction factor χ_{fi} is calculated, given by: $$\chi_{fi} = \frac{1}{\phi_\theta + \sqrt{\phi_\theta^2 - \bar{\lambda}_\theta^2}}$$

Because $\bar{\lambda}_{z,\theta}$ and therefore χ_{fi} are temperature dependant, an incremental process is required to calculate the design resistance at each temperature. The time to failure is determined as the time when the design resistance falls below the design effect (which in this example is 437 kN)

1.8.3 Steel temperature

The change in steel temperature in time interval Δt is given by: — EN 1993-1-2 4.2.5.1

$$\Delta\theta_{a,t} = k_{sh}\frac{A_m/V}{c_a\rho_a}\dot{h}_{net}\,\Delta t$$

(nomenclature as previously defined)

For an unprotected member, the reduction factors for the materials are as defined for the beam design.

For the selected column 203 × 203 × 60 UKC, exposed on four sides,

$$\frac{A_m}{V} = \frac{1210}{7.64} = 158.4 \ \text{m}^{-1}$$

P363

$\left[\frac{A_m}{V}\right]_b$ is the box value of the section factor (the value for a rectangular box that surrounds the profile).

$$\left[\frac{A_m}{V}\right]_b = \frac{(2b + 2h)}{A} = \frac{(2 \times 205.8 + 2 \times 209.6)}{7.64} = 108.7 \text{ m}^{-1}$$

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

$= 0.9 \times 108.7/158.4 = 0.62$

EN 1993-1-2 4.2.5.1(2)

1.8.4 Design buckling resistance of an unprotected column at elevated temperature

A spreadsheet may be used to calculate the gas temperature, the steel temperature and therefore the design resistance at elevated temperature. Figure 1.8 shows the results of the process. The design resistance (plotted on the right hand axis) falls to the design action (437 kN) at a time of 21.1 minutes. As this is less than the required fire resistance period, (30 minutes), the unprotected solution is unsatisfactory. The critical temperature when the column resistance falls below the design action is 691°C.

Note that in Figure 1.8, the design resistance is limited to 1350 kN, the design resistance at ambient temperature.

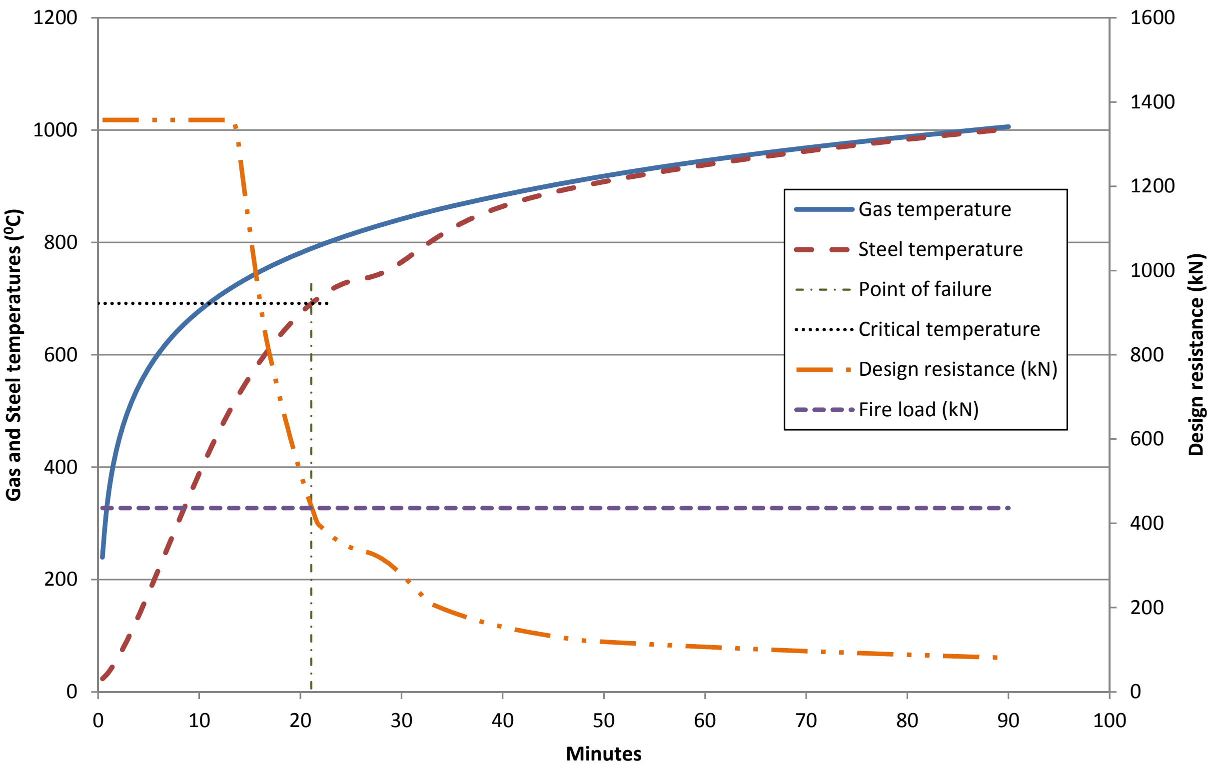

Figure 1.8 Variation of gas temperature, steel temperature and design resistance with time – unprotected column

An identical calculation process demonstrates that an unprotected 203 × 203 × 86 UKC in S355 would be satisfactory (time to failure of 32.9 minutes), if an unprotected solution was required.

1.9 Design resistance of protected column at elevated temperature

Try encasing the column with 10 mm of fire protection board.

The temperature increase in a protected member in time interval Δt is given by:

EN 1993-1-2 4.2.5.2 Expression 4.27

$$\Delta\theta_{\mathrm{a,t}} = \frac{\lambda_{\mathrm{p}} A_{\mathrm{p}}/V}{d_{\mathrm{p}} c_{\mathrm{a}} \rho_{\mathrm{a}}} \frac{(\theta_{\mathrm{g,t}} - \theta_{\mathrm{a,t}})}{(1+\phi/3)} \Delta t - \left(e^{\phi/10} - 1\right) \Delta\theta_{\mathrm{g,t}}$$

where:

$$\phi = \frac{c_{\mathrm{p}} \rho_{\mathrm{p}}}{c_{\mathrm{a}} \rho_{\mathrm{a}}} d_{\mathrm{p}} A_{\mathrm{p}}/V$$

(all nomenclature as previously defined)

λ_{p}, c_{p}, and ρ_{p} are taken from the manufacturer’s data.

For this fire protection board selected, the manufacturer provided the following data:

Thermal conductivity λ_{p} = 0.2 W/mK

Thickness d_{p} = 10 mm

Density ρ_{p} = 800 kg/m^3

Specific heat c_{p} = 1700 J/kgK

$$A_{\mathrm{p}}/V = \frac{\text{Internal surface area of boarding}}{\text{volume of member}}$$

In this instance, A_{p}/V is equal to the box value of the section factor = 108.7 m^{-1}.

An incremental procedure must be followed to determine the gas temperature at time t and therefore the temperature of the protected steelwork. Once the steel temperature is calculated, the resistance calculations follow the same process described for the unprotected column.

The incremental calculation procedure demonstrates that at the required fire resistance period of 30 minutes, the resistance of the column has not been reduced. The resistance of the column is 1350 kN. The steel temperature at 30 minutes in 404°C, less than the critical temperature of 691°C. As shown in Figure 1.9, the resistance of the protected column only reduces after 42 minutes. Note in Figure 1.9, the resistance has been limited to 1350 kN, the design resistance at ambient temperature.

Thus the selected solution is satisfactory.

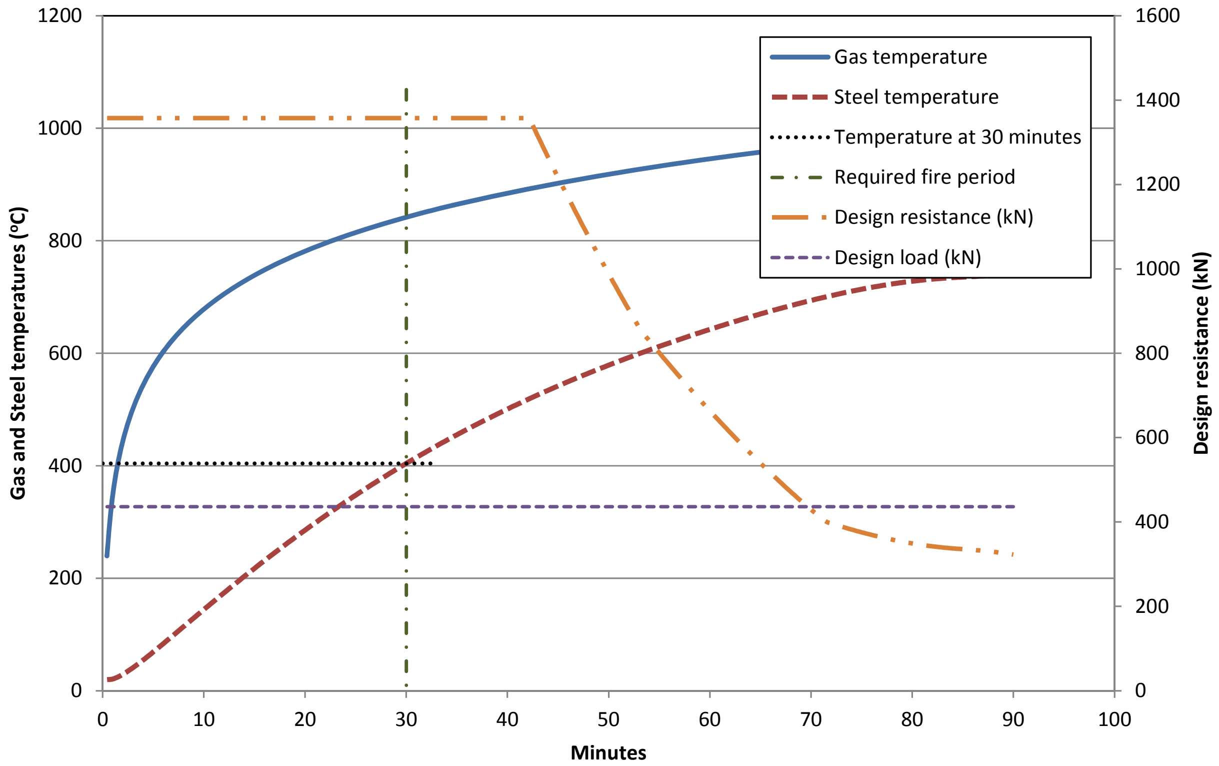

Figure 1.9 Variation of gas temperature, steel temperature and design resistance with time – protected column

1.10 Fire resistance of end-plate beam to column connection

The resistance of the connection at elevated temperature is based on the resistance of the components (the bolts, welds and steel elements) and is compared to a reduced design value of actions (in this case the shear force). The resistance of the connection components at elevated temperature is taken as the resistance at ambient temperature, multiplied by a reduction factor. The resistance of components at ambient temperature follows the guidance in P363. The calculation of the resistance of the key connection components at ambient temperature is demonstrated in Section 1.10.2 and the resistance at elevated temperature in Sections 1.10.4 to 1.10.6.

1.10.1 Arrangement of connection

Figure 1.10 shows the partial depth end plate beam to column connection at the first floor level.

The connection is a 'standardised' detail, taken from the 'Green Book' of simple joints, P358. P358

The connection details are given on page T-11 of P358. There are 6 rows of M20, 8.8 bolts at 70 mm vertical pitch. The end plate is 430 × 200 × 12 in S275 material. 8 mm fillet welds are provided to the beam web.

From page T-11, the vertical shear resistance is given as 752 kN, and the critical check is identified as check 4. Check 4 covers the resistance of the beam web in shear.

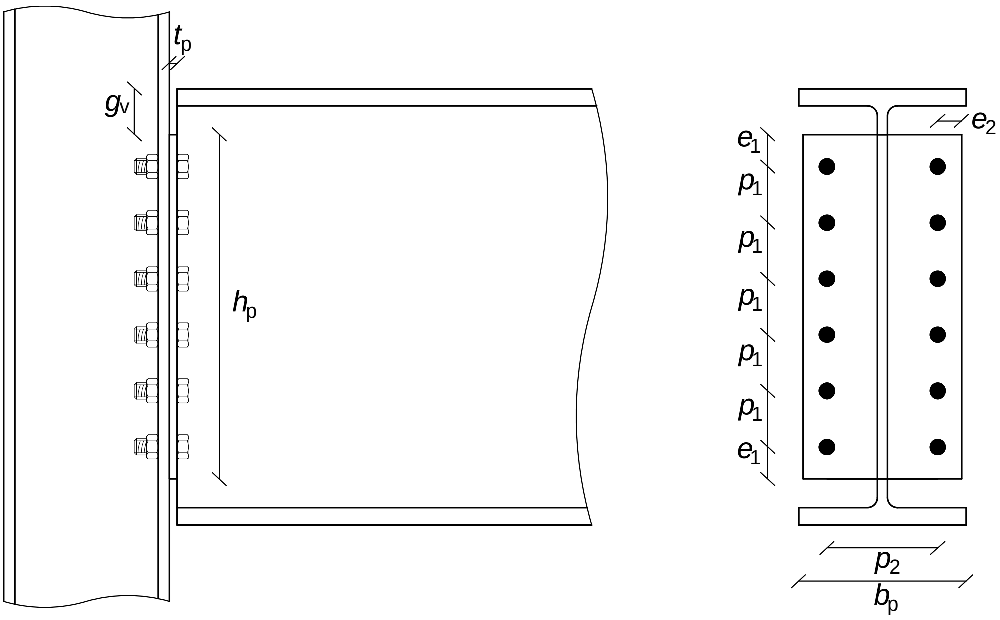

Figure 1.10 The beam to column connection detail at the first floor

1.10.2 Connection resistance at ambient temperature

Bolt group

From page C-381 of P363, the shear resistance of an M20, 8.8 bolt, is 94.1 kN — P363

In P358, check 8, this resistance is factored by 0.8, to allow for the inevitable tension that the bolts experience, even in a nominally pinned joint.

Thus the design shear resistance per bolt is $0.8 \times 94.1 = 75.3$ kN

Because the column flange is 14.2 mm in S355, and the end plate is 12 mm in S275, bearing in the end plate will be critical. — P363

The bolt group has the following geometry:

Edge distance, e_2 = 30 mm

End distance, e_1 = 40 mm

Pitch, p_1 = 70 mm

Gauge, p_2 = 140 mm

As all the above dimensions are the same or larger than those in the middle table on page C-381 of P363, the bearing resistance in 12 mm, S275 plate will be at least 101 kN. — P363

Thus, bolt shear is critical. The resistance of the bolt group is therefore
$12 \times 75.3 = 903.6$ kN

Beam web

In P358, check 4, the resistance of the beam web, $V_{pl,Rd}$ is given by:

$$V_{pl,Rd} = A_v \frac{f_{y,b1}/\sqrt{3}}{\gamma_{M0}}$$

$f_{y,b1} = 265$ N/mm^2, as $16 < t_f < 40$
$A_v = 0.9 \times 430 \times 12.7 = 4915$ mm^2

$$V_{pl,Rd} = 4915 \times \frac{265/\sqrt{3}}{1.0} \times 10^{-3} = 752 \text{ kN (as T-11, P358)}$$

1.10.3 Connection resistance at elevated temperature

Although Annex D of EN 1993-1-2 covers the resistance of joints at elevated temperature, this Annex should not be used, according to the UK National Annex. Non-contradictory information (NCCI) to replace Annex D may be found at www.steel-ncci.co.uk, where resource SN004a-GB covers the strength of bolts and welds in fire situations.

UK NA to EN 1993-1-2 NA.3.2

When calculating the connection resistance, the bolts, welds and steel elements (end plate, beam web etc.) should be verified, against a reduced design action.

The reduction factor for design load level in the fire situation is given by:

EN 1993-1-2 2.4.2(2) Expression 2.5

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

Note that the use of expression 2.5 is associated with the use of expression 6.10 from EN 1990. If expressions 6.10a and 6.10b from EN 1990 had been used, the calculation of η_{fi} should be taken as the smaller value from expressions 2.5a and 2.5b from EN 1993-1-2.

ψ_{fi} is to be taken as $\psi_{1.1}$ according to the UK NA to EN 1991-1-2

UK NA to EN 1991-1-2 NA.2.7

The value of $\psi_{1.1}$ is taken from the UK NA to EN 1990, for (in this instance), office areas.

UK NA to EN 1990 Table NA.A1.1

$$\psi_{fi} = \psi_{1,1} = 0.5$$

$$\eta_{fi} = \frac{5 + 0.5 \times 3.3}{1.35 \times 5 + 1.5 \times 3.3} = 0.57$$

The reduced design shear force is therefore:

$$V_{fi,d} = 0.57 \times \frac{f_d L}{2} = \frac{(11.7 \times 7.5)7.5}{2} = 188 \text{ kN}$$

Temperatures in the connection

The temperature of any part of the joint may be conservatively assumed to be equal to the bottom flange temperature at mid span of the connected beam. As calculated on sheet 15, the bottom flange of the protected steel beam will reach a temperature of 350°C at the required time of 30 minutes. Thus $\theta_b = 350$°C.

PN004a-GB Clause 5

EN 1993-1-8 6.2.6.1

1.10.4 Design resistance of bolts in shear and bearing (at elevated temperature)

At elevated temperature, the design resistance of bolts in shear $F_{v,t,Rd}$ is given by:

PN004a-GB 3.1.1

$$F_{v,t,Rd} = F_{v,Rd}\, k_{b,\theta}\, \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

At elevated temperature, the design resistance of bolts in bearing $F_{b,t,Rd}$ is given by:

$$F_{b,t,Rd} = F_{b,Rd}\, k_{b,\theta}\, \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

where:

γ_{M2} is the partial factor at normal temperature

$\gamma_{M,fi}$ is the partial factor for fire conditions

For bolts in shear and for bolts in bearing, $\gamma_{M2} = 1.25$

UK NA to EN 1993-1-8

The UK National Annex to EN 1993-1-2 suggests the use of the value for partial factors for materials at elevated temperature recommended in EN 1993-1-2 clause 2.3. Therefore:

NA to EN 1993-1-2 NA.2.3

The material partial factor at elevated temperature is:

EN 1993-1-2 2.3(2)

$\gamma_{M.fi} = 1.00$

At 350°C, the reduction factor for bolts, $k_{b,\theta} = 0.839$

PN004a-GB Table 3.1

At normal temperatures, the bolt shear resistance was critical (see sheet 22). This will remain so at elevated temperature as both the shear resistance and the bearing resistance are multiplied by the same factor, $k_{b,\theta}\, \dfrac{\gamma_{M2}}{\gamma_{M,fi}}$.

design resistance of bolts in shear $F_{v,t,Rd}$ is therefore:

$$F_{v,t,Rd} = F_{v,Rd}\, k_{b,\theta}\, \frac{\gamma_{M2}}{\gamma_{M,fi}} = (0.8 \times 94.1) \times 0.839 \times 1.25/1.0 = 79.0 \text{ kN}$$

The resistance of the bolt group is therefore:

12 × 79.0 = 948 kN, > 188 kN, satisfactory.

1.10.5 Design resistance of beam web (at elevated temperature)

Table 3.1 of EN 1993-1-2 provides reduction factors for yield strength.

Assuming all the connection components are at the temperature of the bottom flange (350°C), the reduction factor $k_{y,\theta} = 1.0$.

EN 1993-1-2 Table 3.1

Therefore, there is no reduction to the resistance of the beam web at elevated temperature.

Therefore, the resistance of the connection at elevated temperature is satisfactory.

Note that this also applies to the resistance of the end plate – no reduction is required.

More generally, at temperatures greater than 400°C, Table 3.1 indicates a reduction factor $k_{y,\theta}$ less than 1.0. The verification of connection components such as the end plate and beam web should follow the checks described in P363, using a material strength reduced by the factor $k_{y,\theta}$.

1.10.6 Design resistance of welds (at elevated temperature)

At elevated temperature, the design resistance of fillet welds $F_{w,t,Rd}$ is given by:

$$F_{w,t,Rd} = F_{w,Rd}\, k_{b,\theta}\, \frac{\gamma_{M2}}{\gamma_{M,fi}}$$

PN004a-GB 4.2

For design at ambient temperature, a 'full strength' weld has been provided. Even allowing for some coexisting nominal moment, the vertical shear resistance at ambient temperature is significantly in excess of the applied shear.

Vertical resistance (in the absence of any coexisting moment)

= (430 – 2 × 8) × 2 × 1.25 = 1035 kN

At 350°C, the reduction factor for welds, $k_{w,\theta} = 0.938$

PN004a-GB Table 3.1

Thus the resistance to vertical shear alone is 0.938 × 1035 = 971 kN

Although no coexisting moment has been allowed for, this resistance is much greater than the design shear force of 188 kN, and therefore satisfactory.

In general, the advice of Advisory Desk AD370 should be followed, to determine a required weld size at ambient temperature (which may be less than that provided). At elevated temperature, the reduced weld resistance should be compared to the reduced shear load. The reduced shear load may be increased by a factor of 1.27 to allow for the coexisting moment (see AD370).