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CALCULATION SHEET

# 1 SEVEN-STOREY OFFICE BUILDING

## 1.1 Arrangement and loading

The arrangement of the steel frame for this seven-storey office building is shown in Figure 1.1 and Figure 1.2. Details of the typical floor plan and glazing arrangement are shown in Figure 1.3 and Figure 1.4.

The steelwork has been 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 with composite floors. The initial design was carried out at ambient temperature.

**Table 1.1 Characteristic actions for design**

| **Actions on first floor** | |
|---|---|
| *Permanent Actions:* | |
| Concrete slab | 2.78 kN/m$^2$ |
| Self-weight of decking | 0.13 kN/m$^2$ |
| Allowance for self-weight of beams | 0.6 kN/m$^2$ |
| Ceiling & Services | 0.9 kN/m$^2$ |
| *Variable Actions:* | |
| Occupancy Load | 2.5 kN/m$^2$ |
| Partitions | 0.8 kN/m$^2$ |
| **Actions on roof** | |
| *Permanent Actions* | |
| Concrete slab | 2.78 kN/m$^2$ |
| Self-weight of decking | 0.13 kN/m$^2$ |
| Allowance for self-weight of beams | 0.30 kN/m$^2$ |
| Ceiling & Services | 0.90 kN/m$^2$ |
| *Variable Actions* | |
| Imposed load | 0.6 kN/m$^2$ |

**Table 1.2 Partial factors on actions**

| Factor | Value |
|---|---|
| $\gamma_G$ | 1.35 |
| $\gamma_Q$ | 1.50 |

This worked example demonstrates the verification at elevated temperature of two composite beams (primary and secondary) and a column.

Two verifications of the 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).

EN 1994-1-2 covers the structural fire design of composite members and requires that composite beams are verified for:

EN 1994-1-2 4.3.4.1.1(1)

- Resistance of critical cross-sections
- Vertical shear
- Resistance to longitudinal shear

The verification process for the protected composite beam may be summarised as:

- Under fire conditions, a reduced design value of actions is calculated.
- Determine the temperature of the protected steel beam at the required fire protection period
- Determine reduction factors for the strength of components (steel, concrete, shear studs) at the calculated temperature
- Verify the resistance of the member, based on the (reduced) design resistances of the components.

The verification process for the column is described in the accompanying example covering a two storey office.

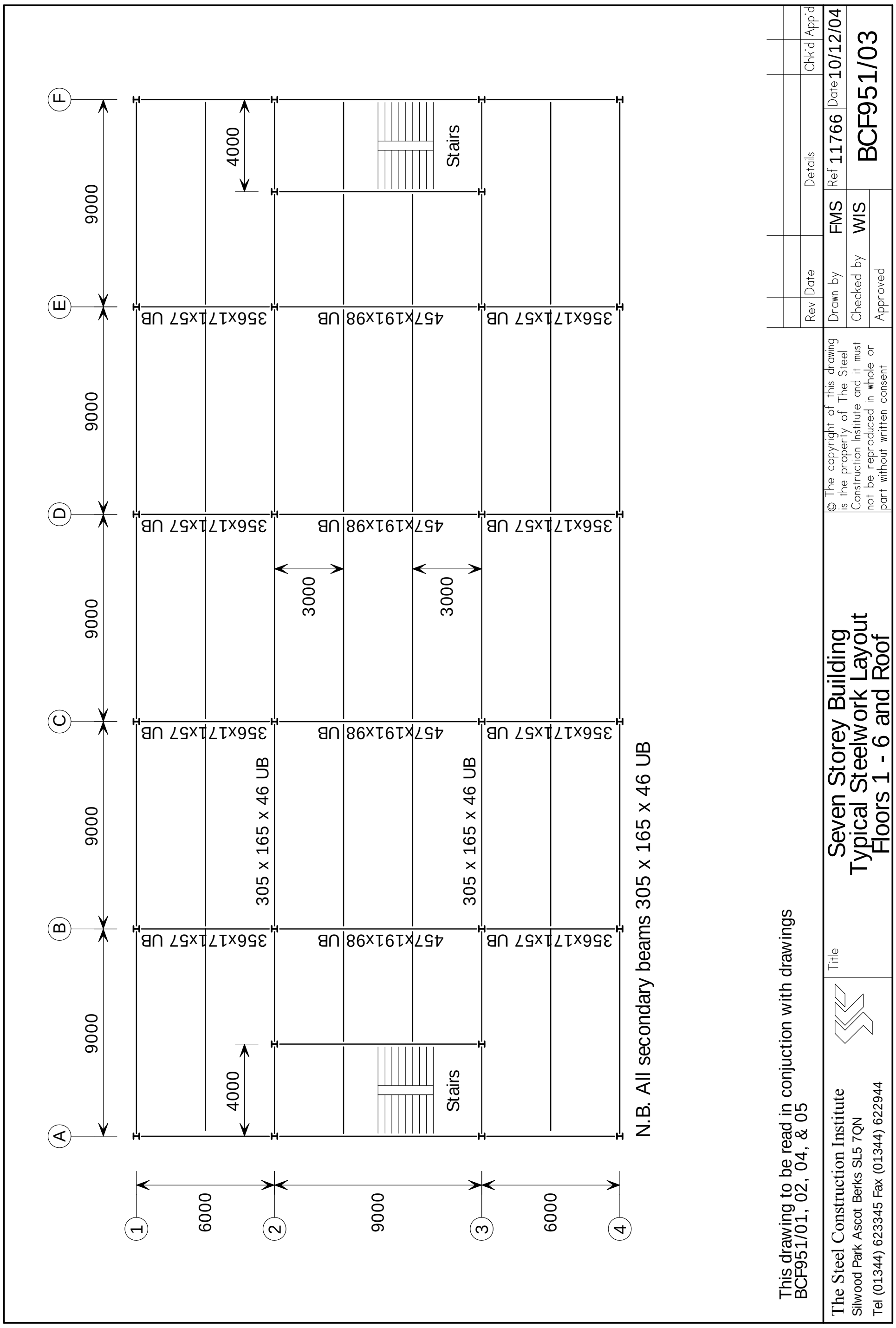


**Figure 1.1 Steelwork layout - floor levels 1 to 6 and roof level**

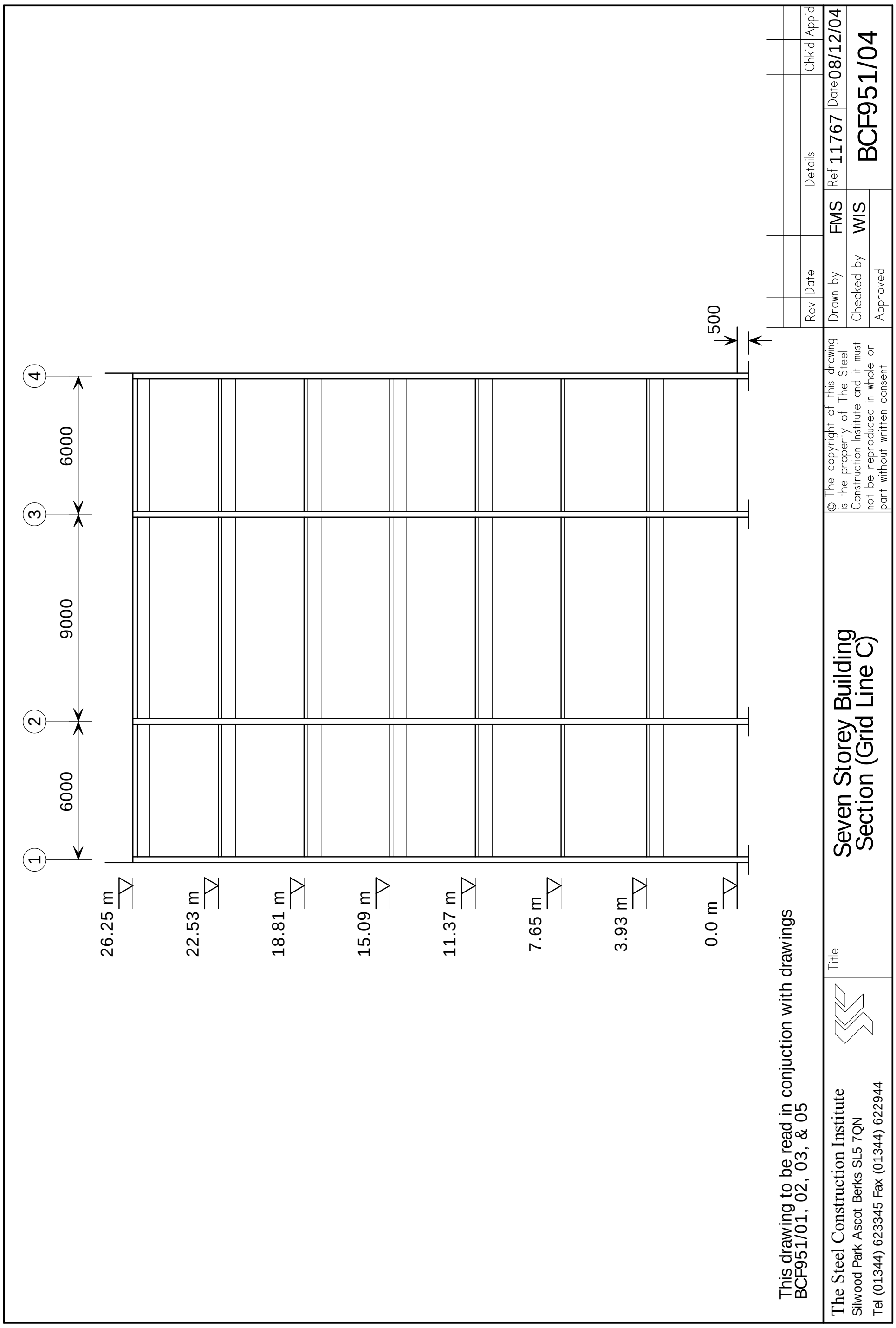


**Figure 1.2 Cross section of gridline C**

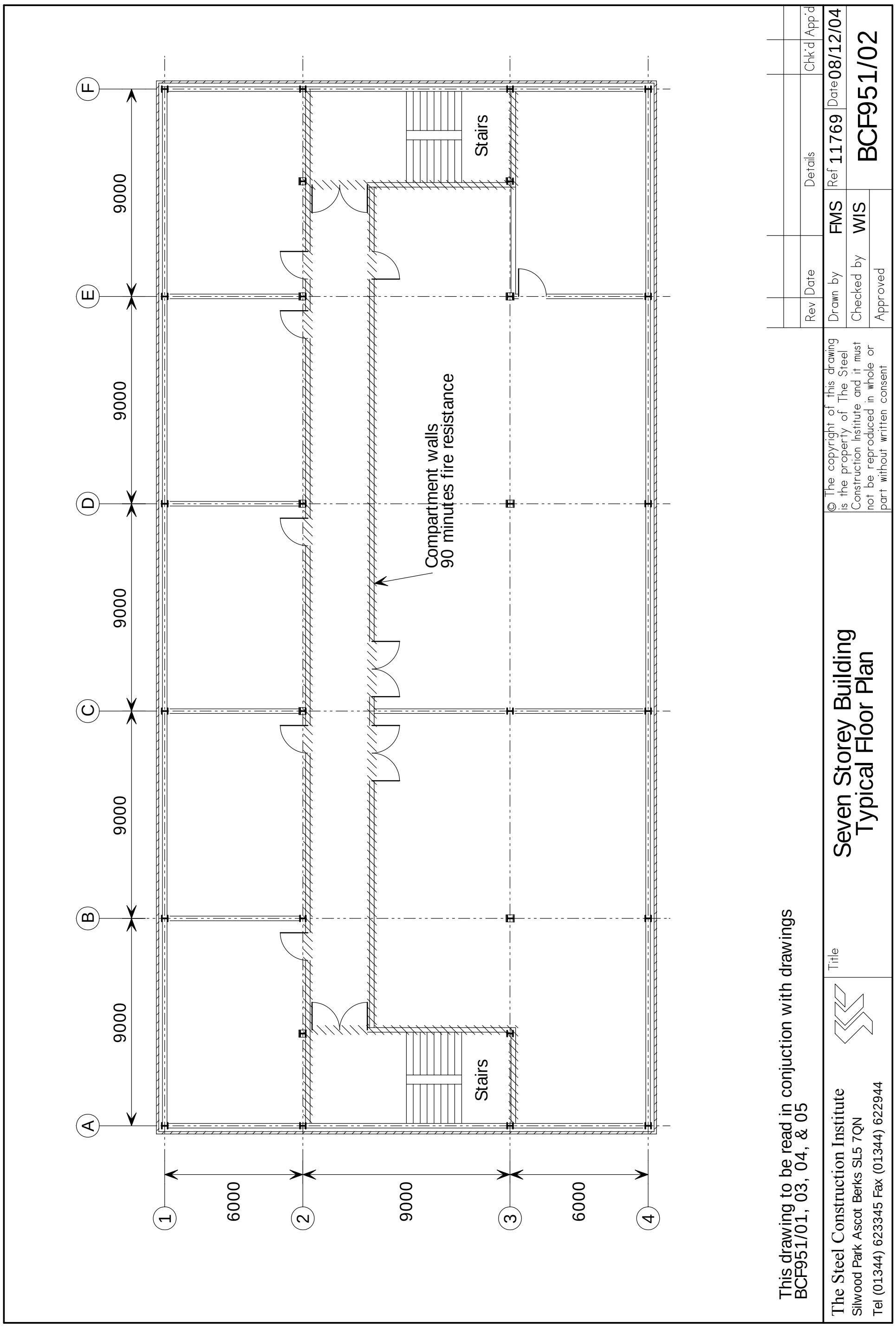


**Figure 1.3 Typical floor plan**

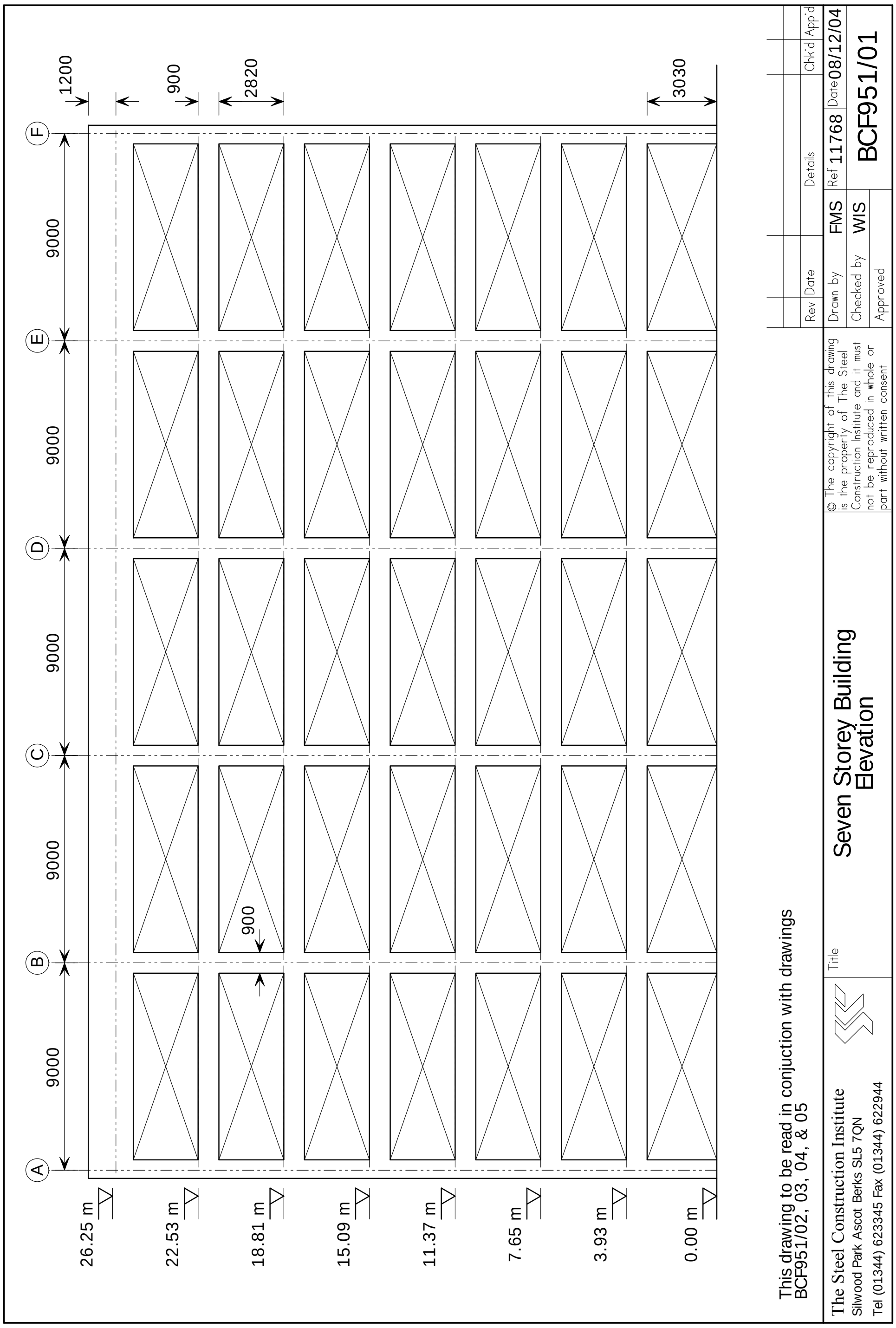


**Figure 1.4 Elevation on gridline 4**

## 1.2 Structural fire design

The structure is an office building, without sprinklers. The floor height of the top storey is 22.53 m above ground level. According to Table A2 of Approved Document B, a 90 minute minimum period of fire resistance is required.

App. Doc. B

## 1.3 Fire resistance of a primary floor beam (gridline B) with fire protection

### 1.3.1 Composite beam details

Beam size: 356 × 171 × 57 UKB, S275

Beam spacing: 9.0 m

Beam span: 6.0 m

The beam has been designed at ambient temperature. The slab is 130 mm deep, using *ComFlor* 51 + deck and normal weight 25/30 concrete. The shear studs are 19 mm diameter, 95 mm long.

From the manufacturer's data, the weight of concrete is 2.78 kN/m$^2$ and the weight of the deck is 0.13 kN/m$^2$.

### 1.3.2 Design effects of actions in fire

EN 1993-1-2 2.4.2

The effects of actions in fire may be determined from EN 1990. In expression 6.11b of EN 1990, the combination factor $\psi$ to be used with the leading variable action is given in EN 1991-1-2.

UK NA to EN 1991-1-2

The UK National Annex to EN 1991-1-2 specifies that the combination factor $\psi_1$ should be used.

UK National Annex to EN 1990 defines $\psi_1$ = 0.5 for (in this instance) office loading.

UK NA to EN 1990 NA.2.2.2

As there is no accompanying variable action, the design value of actions under fire conditions is given by:

EN1990 Expression 6.11(b)

$G_k + \psi_1 \, Q_k$

Characteristic value of the permanent actions:

$G_k \quad = 2.78 + 0.13 + 0.6 + 0.90 = 4.41$ kN/m$^2$

Characteristic value of the variable actions:

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

The design value of actions under fire conditions is therefore

$G_k + \psi_1 \, Q_k \quad = 4.41 + 0.5 \times 3.3 = 6.1$ kN/m$^2$

As an alternative to using expression 6.11b or EN 1990, the effects of actions under fire conditions, $E_{d,fi}$ may be determined from:

EN 1991-1-2 2.4.2(2)

$E_{d,fi} \quad = \eta_{fi} \, E_d$

where:

$\eta_{fi}$ is a reduction factor, given in EN 1993-1-2

$E_d$ is the design value of the corresponding force or moment for normal temperature design.

The expression for $\eta_{fi}$ depends on which expression in EN 1990 has been used to calculate the design value of the combination of actions. In this example, expression 6.10 will be used, and therefore expression 2.5 of EN 1993-1-2 must be used to calculate $\eta_{fi}$.

According to expression 6.10, the design value of the combination of actions is given by: EN 1990 Expression 6.10

$E_d = 1.35 \times 4.41 + 1.5 \times 3.3 = 10.9$ kN/m$^2$

and $\eta_{fi} = \dfrac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} = \dfrac{4.41 + 0.5 \times 3.3}{1.35 \times 4.41 + 1.5 \times 3.3} = 0.556$ EN 1993-1-2 2.4.2(2) Expression 2.5

Thus the design effects in fire are given by:

$E_{d,fi} = \eta_{fi} E_d = 0.556 \times 10.9 = 6.1$ kN/m

*In this instance, the two alternative approaches to determine the design load in fire produce the same result.*

The design value of the point load applied by the secondary beams at the mid span of the primary beam is given by:

$6.1 \times 3 \times 9 = 164.7$ kN

Design bending moment at mid span of the primary beam:

$M_{Ed} = 164.7 \times 6/4 = 247.1$ kNm

Design shear force at the support:

$V_{Ed} = 164.7 /2 = 82.4$ kN

### 1.3.3 Critical temperature of the protected beam

*The critical temperature for the beam may be calculated and a protection system chosen to ensure that the steel remains below this temperature. This approach is simple and conservative. This example demonstrates the calculation of the critical temperature, but then continues to determine the actual temperature of the beam with a specific protection system and to calculate the resistance of the beam at that temperature.*

The critical temperature, $\theta_{a,cr}$ is given by: 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, $\mu_0$, is given by: EN 1993-1-2 4.2.4(3)

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

As lateral torsional buckling is not a potential failure mode, $\mu_0$ may conservatively be obtained from: EN 1993-1-2 4.2.4(4)

$$\mu_0 = \eta_{fi} \frac{\gamma_{M,fi}}{\gamma_{M0}}$$

where $\eta_{fi}$ is the reduction factor calculated above. ($\eta_{fi} = 0.556$)

Both $\gamma_{M,fi}$ and $\gamma_{M0} = 1.0$

EN 1993-1-2 2.3(2)
UK NA to EN 1993-1-1

The critical temperature for composite beams may be calculated using expression 4.22 of EN 1993-1-2 (shown above).

*Note that Table NA.1 of The UK NA to EN 1993-1-2 (which provides default values for critical temperatures) does not cover composite beams.*

The critical temperature is given by:

$$\theta_{a,cr} = 39.19 \ln\left[\frac{1}{0.9674 \times 0.556^{3.833}} - 1\right] + 482 = 567°\text{C}$$

### 1.3.4 Design resistance of protected beam in fire

The resistance of the composite beam at elevated temperature will be verified. Firstly, the temperature of the protected member must be determined.

The temperature increase in a protected member in time interval $\Delta t$ is given by:

EN 1993-1-2 4.2.5.2
Expression 4.27

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

where:

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

where:

$\lambda_p$ 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 if the fire protection material (in m)

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

$\rho_p$ 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 $\Delta t$

*Note that EN 1994-1-2 presents expression 4.27 from EN 1993-1-2 as expression 4.8, in a slightly different format, with modified nomenclature. The results from the expressions are identical.*

$\lambda_p$, $c_p$, and $\rho_p$ are taken from the manufacturer's data.

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

Thermal conductivity $\lambda_p$ = 0.2 W/mK

Thickness $d_p$ = 20 mm

Density $\rho_p$ = 850 kg/m$^3$

Specific heat $c_p$ = 1700 J/kgK

$$\frac{A_p}{V} = \frac{\text{internal surface area of boarding}}{\text{volume of member}} = \frac{172.2 + 2 \times 358}{7.26} = 122.3 \text{ m}^{-1}$$ EN 1993-1-2 4.2.5.2(4)

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, $\Delta t$ 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. In this example, $\Delta t$ has been taken as 5 seconds.

The results of this incremental procedure are shown in Figure 1.5. At 90 minutes, the temperature of the steel beam is 588°C.

*This is higher than the critical temperature calculated as 567°C and would indicate that the chosen protection is not adequate. However, the critical temperature approach in Section 1.3.3 used a conservative value of the degree of utilisation,* $\mu_0$*. Using the actual degree of utilisation would demonstrate the critical temperature to be higher.*

*This example continues in order to demonstrate that the calculated resistance of the composite beam with the selected protection system is adequate.*

For members with box protection, a uniform temperature may be assumed over the height of the profile. EN 1994-1-2 4.3.4.2.2(10)

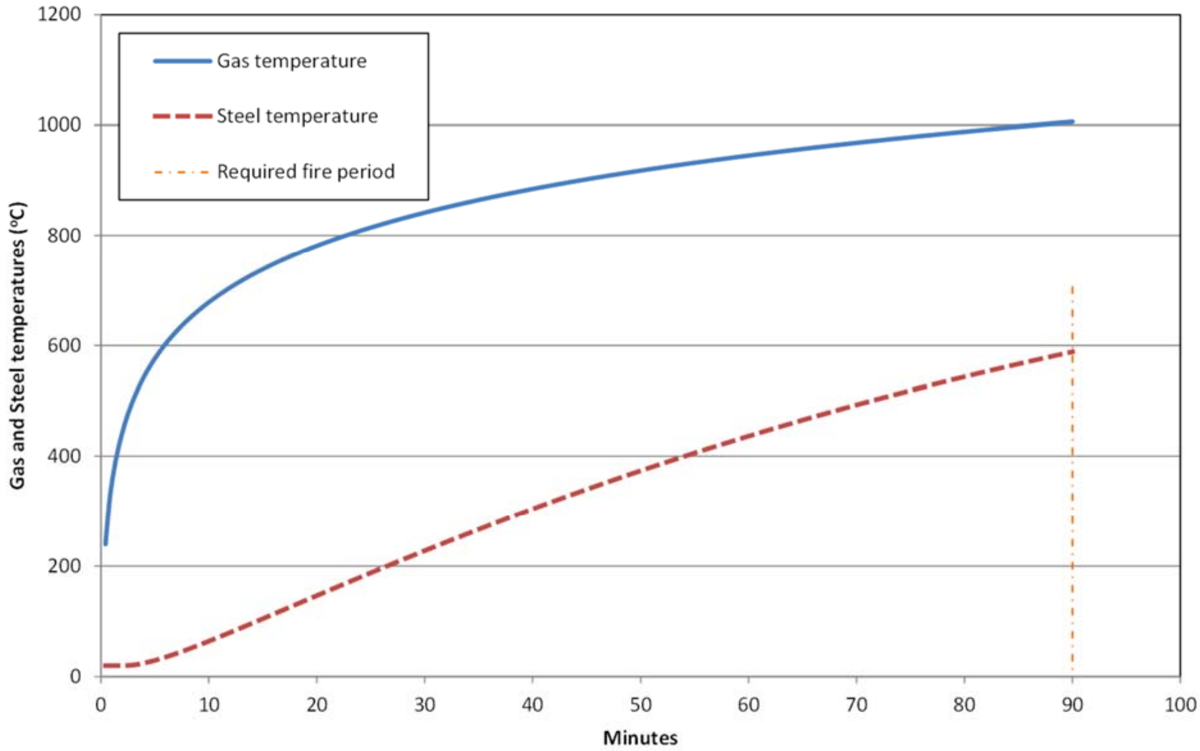


**Figure 1.5 Variation of gas and steel temperatures with time – protected beam**

## 1.4 Vertical shear resistance at elevated temperature

The resistance to vertical shear is to be taken as the resistance of the steel section alone, which may be calculated in accordance with E.4 of Annex E to EN 1994-1-2 — EN 1994-1-2 4.2.4.1.3

Clause E.4 recommends that clause 6.2.2 of EN 1994-1-1 is used to check the vertical shear resistance of a composite section, replacing $E_a$, $f_{ay}$, and $\gamma_a$ with $E_{a,\theta}$, $f_{ay,\theta}$, and $\gamma_{M,fi,a}$ respectively. — EN 1994-1-2 E.4(1)

From Table 3.2: — EN 1994-1-2 Table 3.2

$E_{a,\theta} = k_{E,\theta} \times E_a$

$f_{ay,\theta} = k_{y,\theta} \times f_{ay}$

$\gamma_{M,fi,a}$ is given in EN 1994-1-2 clause 2.3(1), and confirmed by the UK National Annex to EN 1994-1-2 as $\gamma_{M,fi,a} = 1.0$ — UK NA to EN 1994-1-2 NA.2.5

From Table 3.2, at 588°C: — EN 1994-1-2 Table 3.2

$k_{E,\theta} = 0.345$

$k_{y,\theta} = 0.507$

According to 6.2.2.2 of EN 1994-1-1, the plastic shear resistance should be calculated in accordance with EN 1993-1-1 — EN 1994-1-1 6.2.2.2(2)

The shear resistance at ambient temperature may therefore be taken from P363, — P363

$V_{c,Rd} = 501$ kN

At elevated temperature, the shear resistance is given by:

$V_{c,fi,Rd}$ $= 0.507 \times 501 = 254$ kN

The resistance (254 kN) exceeds the design effect (82.4 kN), so the shear resistance at 90 minutes is satisfactory.

## 1.5 Resistance of the composite section

The resistance of the composite section is determined after calculating the reduced resistance of the steel and concrete elements of the section.

The bending resistance of the composite section maybe calculate by plastic theory, taking into account the variation of material properties with temperature. The primary elements in the calculation are the steel resistance, the concrete resistance and the resistance of the shear studs. EN 1994-1-2 4.3.4.2.4(1)

***Steel temperature***

The temperature of the steel beam has already been calculated as 588°C at 90 minutes. Because the member has box protection, a uniform temperature maybe assumed over the height of the steel beam, as $A_p/V$ has been used in the earlier calculation of the temperature. EN 1994-1-2 4.3.4.2.2(11)

***Concrete temperature***

Although a method for determining the temperatures in a concrete slab is given in Annex D, this Annex cannot be used, according to the UK National Annex. UK NA to EN 1994-1-2 NA.3

Non-contradictory complementary information may be found at www.steel-ncci.co.uk. Resource PN005c-GB provides alternative guidance.

***Stud temperature***

When determining the resistance of a shear stud at elevated temperature, the temperature of the stud connector and of the concrete may be taken as 80% and 40% respectively of the temperature of the upper flange of the steel beam. EN 1994-1-2 4.3.4.2.5(2)

### 1.5.2 Resistance of the steel beam

The resistance of the steel beam may be calculated using the reduction factor for yield strength, $k_{y,\theta}$ as given in Table 3.2 of EN 1994-1-2 or Table 3.1 of EN 1993-1-2. *(the reduction factor is identical)*

At 588°C, the reduction factor $k_{y,\theta} = 0.507$ EN 1994-1-2 Table 3.2

The resistance of the steel beam in tension is given by: UK NA to EN 1994-1-2 NA.2.5

$k_{y,\theta} f_y A/\gamma_{M,fi}$

where $\gamma_{M,fi} = 1.0$

Thus the reduced tension resistance of the steel beam

$= 0.507 \times 275 \times 7260 \times 10^{-3}/ 1.0 = 1012$ kN

### 1.5.3 Resistance of the shear studs at ambient temperature

Firstly, the resistance of the shear studs at ambient temperature must be calculated, as given by equations 6.18 and 6.19 of EN 1994-1-1 EN 1994-1-2 4.3.4.2.5(1)

According to equation 6.18;

$$P_{\mathrm{Rd}} = \frac{0.8 f_{\mathrm{u}} \pi d^2/4}{\gamma_{\mathrm{V}}}$$

EN 1994-1-1 6.6.3.1

where

$f_{\mathrm{u}}$ is the ultimate strength of the stud, in this case 450 N/mm$^2$

$d$ is the stud diameter, in this case 19 mm

$\gamma_{\mathrm{V}}$ is the partial factor, taken as 1.25, confirmed by the UK National Annex

UK NA to EN 1994-1-1 NA.2.3

Then $P_{\mathrm{Rd}} = \dfrac{0.8 \times 450 \times \pi \times 19^2/4}{1.25} \times 10^{-3} = 81.7$ kN

According to equation 6.19;

$$P_{\mathrm{Rd}} = \frac{0.29 \alpha d^2 \sqrt{f_{\mathrm{ck}} E_{\mathrm{cm}}}}{\gamma_{\mathrm{V}}}$$

Because $h_{\mathrm{sc}}/d$ = 95/15 = 6.33 (greater than 4), $\alpha$ = 1

Then $P_{\mathrm{Rd}} = \dfrac{0.29 \times 1 \times 19^2 \sqrt{25 \times 30500}}{1.25} \times 10^{-3} = 73.1$ kN

In profiled steel sheeting, the design shear resistance must be multiplied by a reduction factor, $k_{\mathrm{l}}$

EN 1994-1-1 6.6.4.1(2)

$$k_{\mathrm{l}} = 0.6 \frac{b_{\mathrm{o}}}{h_{\mathrm{p}}} \left( \frac{h_{\mathrm{sc}}}{h_{\mathrm{p}}} - 1 \right) \text{ but } \leq 1.0$$

where

$b_{\mathrm{o}}$ is the distance between ribs, in this case 110 mm

$h_{\mathrm{p}}$ is the height of the profile, in this case 51 mm

$h_{\mathrm{sc}}$ is the height of the shear connector, in this case 95 mm

Then $k_{\mathrm{l}} = 0.6 \dfrac{110}{51} \left( \dfrac{95}{51} - 1 \right)$ = 1.11, but limited to 1.0

Thus the minimum resistance, at ambient temperature, = 1.0 × 73.1 = 73.1 kN

### 1.5.4 Resistance of the shear studs at elevated temperature

At elevated temperature,

EN 1994-1-2 4.3.4.2.5(1)

$P_{\mathrm{fi,Rd}} = 0.8\, k_{\mathrm{u},\theta}\, P_{\mathrm{Rd}}$ ($P_{\mathrm{Rd}}$ from equation 6.18 of EN 1994-1-1), or

$P_{\mathrm{fi,Rd}} = k_{\mathrm{c},\theta}\, P_{\mathrm{Rd}}$ ($P_{\mathrm{Rd}}$ from equation 6.19 of EN 1994-1-1)

In both cases, $\gamma_{\mathrm{V}}$ is replaced with $\gamma_{\mathrm{M,fi}}$

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

The temperature of the stud = 0.8 × 588 = 470°C

EN 1994-1-2 4.3.4.2.5(2)

From Table 3.2, $k_{\mathrm{u},\theta}$ = 0.846

EN 1994-1-2 Table 3.2

Thus $P_{fi,Rd} = 0.8 \times 0.846 \times \dfrac{0.8 \times 450 \times \pi \times 19^2/4}{1.0} \times 10^{-3} = 69.1$ kN

The temperature of the concrete = 0.4 × 588 = 235°C

Although clause 4.3.4.2.2(16) indicates that no reduction is required for temperatures less than 250°C, Table 3.3 indicates a small reduction.

From Table 3.3, $k_{c,\theta} = 0.915$ — EN 1994-1-2 Table 3.3

Thus $P_{fi,Rd} = 0.915 \times \dfrac{0.29 \times 1 \times 19^2\sqrt{25 \times 30500}}{1.0} \times 10^{-3} = 83.5$ kN

Thus the minimum resistance, at elevated temperature, = 69.1 kN

The number of studs over half the span, allowing for 300 mm at the ends of the beam = (3000-300)/150 = 18 studs

Thus the maximum force that can be transferred by the studs is
18 × 69.1 = 1244 kN

Thus the maximum force in the concrete is the minimum of 1244 kN and 1012 kN.

Therefore the force in the concrete = 1012 kN

## 1.6 Resistance of the concrete at elevated temperature

Although Annex D of EN 1994-1-2 provides information on the temperature within a concrete slab, the UK National Annex prohibits the use of Annex D. — UK NA to EN 1994-1-2 NA.3

Non-conflicting complementary information giving temperatures within a concrete slab may be obtained from resource PN005c-GB, obtained from www.steel-ncci.co.uk.

### 1.6.1 Minimum slab thickness for adequate insulation

The minimum slab thickness for a 90 minute fire resistance period is given as 110 mm for normal weight concrete. The actual slab is 130 mm, so is satisfactory. — PN005c-GB Table 3.2

Expressions for the concrete temperature for normal weight concrete and re-entrant deck are given in Equations 10, 11 and 12, covering up to 51 mm from the soffit within the ribs, at other locations within the ribs and at locations not in the rib.

Within a rib, up to 51 mm from the soffit, the concrete temperature is given by: — PN005c-GB Table 2.9

$\theta_x = 0.04x^2 - 9.5x + 1030$

where $x$ is the distance above the soffit

At other depths within a rib, the concrete temperature is given by: — PN005c-GB Table 2.10

$\theta_x = 0.016\sigma^{2.4} - 6.3\sigma^{1.1} + \theta_{51}$

where:

$$\sigma = \left(\frac{250}{d_{\text{slab}}}\right)^{0.5}$$

$\theta_{51}$ is the temperature at 51 mm within a rib

$d_{\text{slab}}$ is the slab depth

At other locations, the concrete temperature is given by:

PN005c-GB Table 2.11

$$\theta_x = 0.016\sigma^{2.8}(x - d_p) - 6.5\sigma^{1.28}(x - d_p) + 700$$

where:

$d_p$ is the height of the profile, in this instance 51 mm

The graph of these expressions is shown in Figure 1.6.

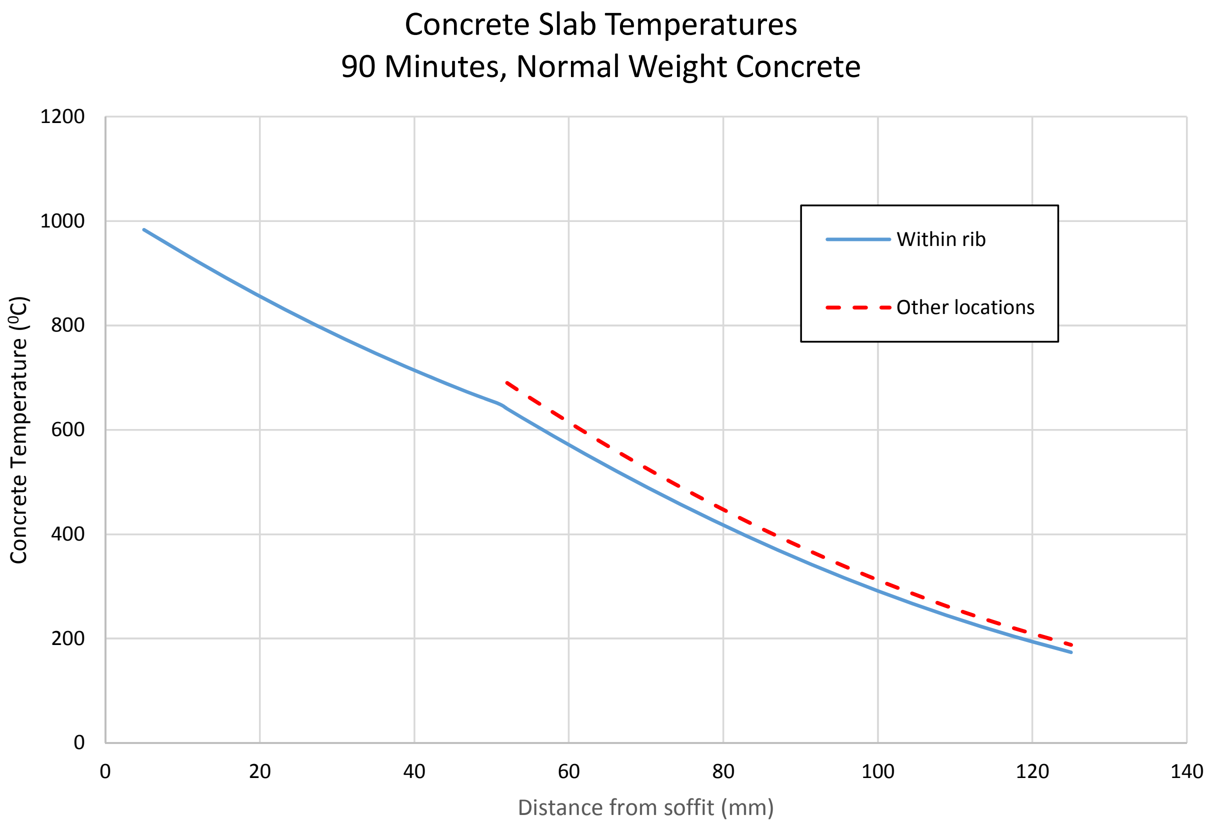


**Figure 1.6 Variation of temperature within the concrete slab**

A reduction factor, $k_{c,\theta}$ may be obtained from Table 3.3 of EN 1994-1-2

EN 1994-1-2 3.2.2(4) Table 3.3

The concrete temperature, and the reduction factor $k_{c,\theta}$, for concrete strips at distances measured from the top surface of the slab, are shown in Table 1.3. The datum used when calculating the temperature is taken as the mid-height of the strip. The temperatures in Table 1.3 are the higher values shown in Figure 1.6, indicated as "other locations". These are temperatures in the slab above the dovetail void, and correspond to positions P4 to P5 in PN005c-GB, Figure 2.4.

**Table 1.3 Concrete temperatures and reduction factors**

| Strip (from top surface) | Datum (mm) (from top surface) | Concrete temperature (°C) (See Figure 1.6) | Reduction factor $k_{c,\theta}$ (from Table 3.3) |
|---|---|---|---|
| 0 – 10 mm | 5 | 188 | 1.0 |
| 10 – 20 mm | 15 | 232 | 1.0 |
| 20 – 30 mm | 25 | 283 | 0.867 |
| 30 – 40 mm | 35 | 343 | 0.807 |
| 40 – 50 mm | 45 | 410 | 0.734 |
| 50 – 60 mm | 55 | 486 | 0.621 |
| 60 – 70 mm | 65 | 570 | 0.496 |
| 70 – 80 mm | 75 | 661 | 0.358 |

## 1.6.2 Effective width

Effective width $b_{eff} = 2 \times 6000/8 = 1500$ mm

EN 1994-1-1 5.4.1.2(5)

## 1.6.3 Resistance of the concrete slab

EN 1994-1-2 4.3.1(4)

The resistance of the concrete is given by:

$\alpha_{slab} A_j \, k_{c,\theta,j} \, f_{c,j} / \gamma_{M,fi,c}$

where:

$\alpha_{slab}$ is a coefficient accounting for the assumption of a rectangular stress block. $\alpha_{slab} = 0.85$

$A_j$ is the area of the $j^{th}$ strip

$k_{c,\theta,j}$ is the reduction factor for the $j^{th}$ strip

For example, the resistance of the strip between 50 and 60 mm is given by:

$0.85 \times 10 \times 1500 \times 0.621 \times 25 \times 10^{-3} / 1.0 = 198$ kN

The resistance of each strip, based on an effective width of 1500 mm, is shown in Table 1.4.

**Table 1.4 Resistance of the concrete slab, in strips from the upper surface**

| Strip | Resistance (kN) | Cumulative resistance (kN) |
|---|---|---|
| 0 – 10 mm | 318.8 | 318.8 |
| 10 – 20 mm | 318.8 | 637.5 |
| 20 – 30 mm | 276.3 | 913.8 |
| 30 – 40 mm | 257.3 | 1171.1 |

In this example, only 33.8 mm of the concrete slab is required to equate to the tension force previously determined as 1012 kN.

Thus the plastic neutral axis is at 33.8 mm from the top surface of the slab.

The design moment resistance of the composite beam at elevated temperature is given by the summation of the design forces, multiplied by their lever arms, as shown in Figure 1.7.

EN 1994-1-2 4.3.1(5)

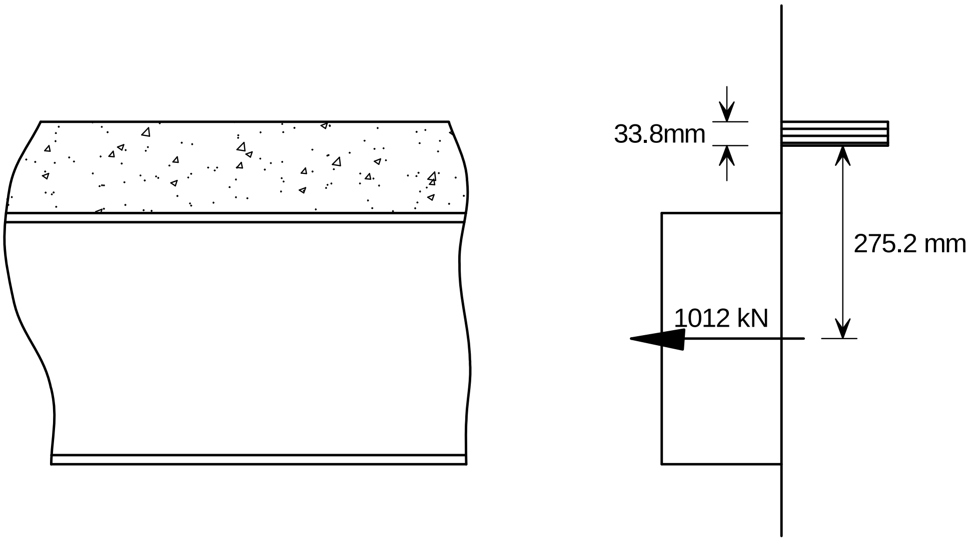


**Figure 1.7 Design forces in composite beam**

The design moment of resistance is given by:

1012 × 275

+ 98.2 × 3.8/2

+ 276.3 × (5 + 3.8)

+ 318.8 × (15 + 3.8)

+ 318.8 × (25 + 3.8)

= 296,093 kNmm, = 296 kNm

Thus the design resistance (296 kNm) is greater than the design action (247.1 kNm), so the resistance of the composite beam at elevated temperature is satisfactory.

# 2 Fire resistance of a secondary floor beam with fire protection

Most of the required information has been determined for the primary beam, which is utilised within the following verification of a secondary beam.

### 2.1.1 Composite beam details

Beam size: 305 × 165 × 46 UKB, S355

Beam spacing: 3.0 m

Beam span: 9.0 m

From Section 1.3.2, the design value of actions in fire is 6.1 kN/m$^2$.

The design bending moment is therefore $(6.1 \times 3) \times 9^2/8 = 185.3$ kNm

The design shear force is $(6.1 \times 3) \times 9 / 2 = 82.4$ kN

Following the pattern of Sections 1.3.2 and 1.3.4, an incremental process may be used to determine the temperature of the steel beam after 90 minutes. Like the primary beams, 20 mm of identical fire protection has been assumed. The results of this incremental process are shown in Figure 2.1. At 90 minutes, the temperature of the steel beam is 606°C.

*This is higher than the critical temperature calculated in Section 1.3.3 as 567°C and would indicate that the chosen protection is not adequate. However, the critical temperature approach is conservative, and this example continues in order to demonstrate that the calculated resistance of the composite beam with the selected protection system is adequate.*

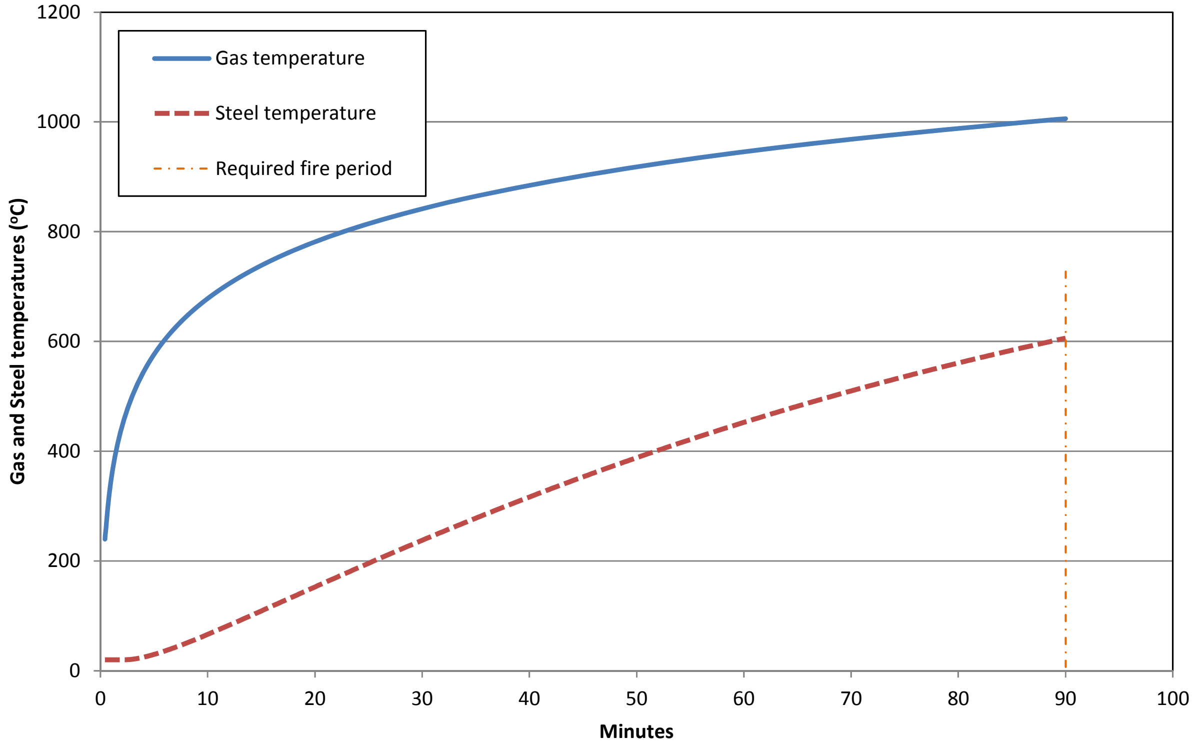


**Figure 2.1 Variation of gas and steel temperatures with time – protected beam**

## 2.2 Vertical shear resistance at elevated temperature

At 606°C, the reduction factor $k_{y,\theta} = 0.46$

EN 1994-1-2 Table 3.2
P363

Thus the vertical shear resistance at elevated temperature is
$0.46 \times 461 = 212$ kN

The resistance (212 kN) exceeds the design effect (82.4 kN), so the shear resistance at 90 minutes is satisfactory.

## 2.3 Resistance of the composite section

### 2.3.1 Resistance of the steel beam

The reduced tension resistance of the steel beam

$= 0.46 \times 355 \times 5870 \times 10^{-3} / 1.0 = 959$ kN

### 2.3.2 Resistance of the shear studs at ambient temperature

Following the calculation process in Section 1.5.3, the basic stud resistances are identical (81.7 kN and 73.1 kN from expression 6.18 and 6.19 respectively) — EN 1994-1-2 4.3.4.2.5(1)

When the decking is transverse to the span, the calculated resistances must be multiplied by the reduction factor $k_t$, as given by: — EN 1994-1-1 6.6.4.2

$$k_\mathrm{t} = \frac{0.7}{\sqrt{n_\mathrm{r}}} \frac{b_\mathrm{o}}{h_\mathrm{p}} \left( \frac{h_\mathrm{sc}}{h_\mathrm{p}} - 1 \right)$$

where:

$n_r$ is the number of studs in one rib (assumed here to be 1)

$b_o$ is the clear distance between ribs, given by the manufacturer as 110 mm

$h_{sc}$ is the height of the stud (95 mm in this instance)

$h_p$ is the height of the profile, given by the manufacturer as 51 mm

Then $k_t = \frac{0.7}{\sqrt{1}} \frac{110}{51} \left( \frac{95}{51} - 1 \right) = 1.3$

$k_t$ is also limited by the maximum value given in Table 6.2, and for the deck (assumed to be less than 1 mm thick), for through-deck welding, the maximum value is 0.85 — EN 1994-1-1 Table 6.2

Thus the revised values of resistance are:

From expression 6.18, $P_{Rd} = 81.7 \times 0.85 = 69.5$ kN

From expression 6.19, $P_{Rd} = 73.1 \times 0.85 = 62.1$ kN

### 2.3.3 Resistance of the shear studs at elevated temperature

The temperature of the stud = 0.8 × 606 = 485°C — EN 1994-1-2 4.3.4.2.5(2)

From Table 3.2, $k_{u,\theta} = 0.813$ — EN 1994-1-2 Table 3.2

Thus based on equation 6.18, — EN 1994-1-2 4.3.4.2.5(1)

Thus $P_{fi,Rd} = 0.8 \times 0.813 \times 69.5 \times 1.25 = 56.5$ kN

The temperature of the concrete = 0.4 × 606 = 242°C — EN 1994-1-2 Table 3.3

From Table 3.3, $k_{c,\theta} = 0.908$

Thus based on equation 6.19,

Thus $P_{fi,Rd} = 0.908 \times 62.1 \times 1.25 = 70.5$ kN

Thus the minimum resistance, at elevated temperature, = 56.5 kN

The number of studs over half the span, allowing for 150 mm at the ends of the beam = (4500-150)/150 = 29 studs

Thus the maximum force that can be transferred by the studs is 29 × 56.5 = 1639 kN

Thus the maximum force in the concrete is the minimum of 1639 kN and 959 kN.

Therefore the force in the concrete = 959 kN

## 2.4 Resistance of the concrete at elevated temperature

The calculations to determine the temperature in the concrete and the reduction factors are identical to the process described in Section 1.6. The concrete temperatures and reduction factors are given in Table 1.3.

### 2.4.1 Effective width

EN 1994-1-1 5.4.1.2(5)

Effective width $b_{\mathrm{eff}} = 2 \times 9000/8 = 2250$ mm

The resistance of each strip, based on an effective width of 2250 mm, is shown in Table 2.1

**Table 2.1 Resistance of the concrete slab, in strips from the upper surface**

| Strip | Resistance (kN) | Cumulative resistance (kN) |
|---|---|---|
| 0 – 10 mm | 478.1 | 478.1 |
| 10 – 20 mm | 478.1 | 956.3 |
| 20 – 30 mm | 414.5 | 1370.7 |

In this instance only 20 mm of the concrete slab is required to equate to the tension force previously determined as 959 kN. The difference between 956 kN and 959 kN may be ignored.

Thus the plastic neutral axis is at 20 mm from the top surface of the slab.

The design moment resistance of the composite beam at elevated temperature is given by the summation of the design forces, multiplied by their lever arms, as shown in Figure 2.2.

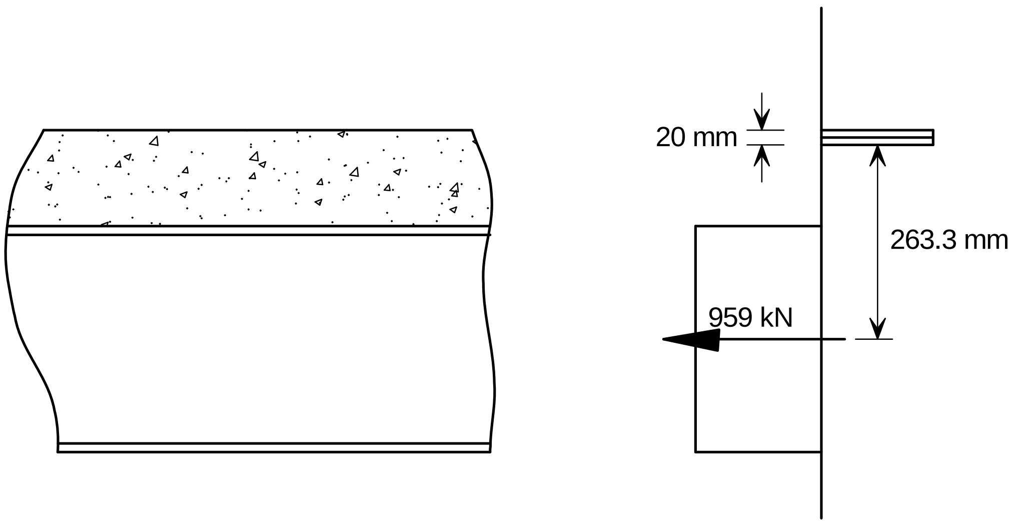


**Figure 2.2 Design forces in composite beam**

The design moment of resistance is given by:

959 × 263.3

+ 478.1 × 5

+ 478.1 × 15

= 262,067 kNmm = 262 kNm

Thus the design resistance (262 kNm) is greater than the design action (185.3 kNm), so the resistance of the composite beam at elevated temperature is satisfactory.

## 2.5 Fire resistance of a column (ground to first floor)

This section of the example demonstrates the verification of an internal column (305 × 305 × 158 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.

### 2.5.1 Verification at normal temperature

Axial force due to permanent actions, $G_k$ = 1751 kN

Axial force due to variable actions, $Q_k$ = 1377 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

$N_{Ed} = 1.35 \times 1751 + 1.5 \times 1377 = 4429$ kN

The chosen column section, a 305 × 305 × 158 UKC S355 is at least a Class 2 Section at ambient temperature.

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*This may be verified by inspecting the "n limit" given on page D-200 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:

$N_{c,Rd} = N_{pl,Rd} = Af_y / \gamma_{M0}$

$= 6930$ kN $> N_{Ed}$ therefore OK

EN 1993-1-1 6.2.4
P363

Design buckling resistance of the cross-section:

EN 1993-1-1 6.3.1.1

$N_{b,Rd} = \chi A f_y / \gamma_{M1}$

The length of the bottom storey column is estimated to be 4430 mm. Assuming a buckling length of 1.0 × system length, the buckling resistance (interpolated from P363) is 4930 kN

P363

$N_{b,Rd} = 4930$ kN $> N_{Ed}$ therefore OK

### 2.5.2 Design loading at elevated temperature

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

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

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(3) Expression 2.5

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

$\psi_{fi}$ 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

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

$$\eta_{fi} = \frac{1751 + 0.5 \times 1377}{1.35 \times 1751 + 1.5 \times 1377} = 0.55$$

Hence:

$N_{fi,Ed} = 0.55 \times N_{Ed} = 0.55 \times 4429 = 2436$ kN

## 2.6 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)

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

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

$\gamma_{Mfi} = 1.0$

EN 1993-1-2 2.3(2)

$f_y = 345$ N/mm$^2$ (as $t_f > 16$ mm)

$k_{y,\theta}$ 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.

### 2.6.1 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 $\varepsilon = 0.85\sqrt{\frac{235}{f_y}} = 0.85\sqrt{\frac{235}{345}} = 0.7$ ($f_y = 345$ N/mm$^2$, since $16 < t_f < 40$)

For the web;

P363

$c/t$ for a 305 × 305 × 158 UKC = 15.6

Class 1 limiting value = $33\varepsilon = 33 \times 0.7 = 23.1$

EN 1993-1-1 Table 5.2

Thus the web is Class 1

For the flange;

$c/t$ for a 305 × 305 × 158 UKC = 4.22 — P363

Class 1 limiting value = $9\varepsilon$ = 9 × 0.7 = 6.3 — 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.

$A$ = 20100 mm$^2$ — P363

For intermediate storeys of a braced frame with separate fire compartments, the buckling length may be taken as $l_{fi}$ = 0.5$L$. Therefore: — EN 1993-1-2 4.2.3.2(5)

Buckling length, $L_{cr} = L_{fi}$ 0.5 × 4430 = 2215 mm

The non-dimensional slenderness (at ambient temperature) — EN 1993-1-1 6.3.1.3

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

where:

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

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 $\chi_{fi}$*

The reduction factor for buckling at elevated temperature, $\chi_{fi}$, 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. $\phi_\theta$ is calculated, given by: $\phi_\theta = \frac{1}{2}\left[1 + \alpha\bar{\lambda}_\theta + \bar{\lambda}_\theta^2\right]$ where $\alpha = 0.65\sqrt{235/f_y}$

The reduction factor $\chi_{fi}$ 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 $\chi_{fi}$ 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 2436 kN)

## 2.6.2 Steel temperature

The change in steel temperature in time interval $\Delta t$ is given by:

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

EN 1993-1-2 4.2.5.1

(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 305 × 305 × 158 UKC, exposed on four sides,

$$\frac{A_m}{V} = \frac{1840}{20.1} = 91.5 \ \text{m}^{-1}$$

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

$$\left[\frac{A_m}{V}\right]_b = \frac{(2b + 2h)}{A} = \frac{(2 \times 327.1 + 2 \times 311.2)}{20.1} = 63.5 \text{ m}^{-1}$$

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

$= 0.9 \times 63.5/91.5 = 0.62$

EN 1993-1-2 4.2.5.1(2)

## 2.6.3 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 2.3 shows the results of the process. The design resistance (plotted on the right hand axis) falls to the design action (2436 kN) at a time of 22.8 minutes. As this is less than the required fire resistance period, (90 minutes), the unprotected solution is unsatisfactory. The critical temperature when the column resistance falls below the design action is 608°C.

Note that in Figure 2.3, the design resistance is limited to 4930 kN, the design resistance at ambient temperature.

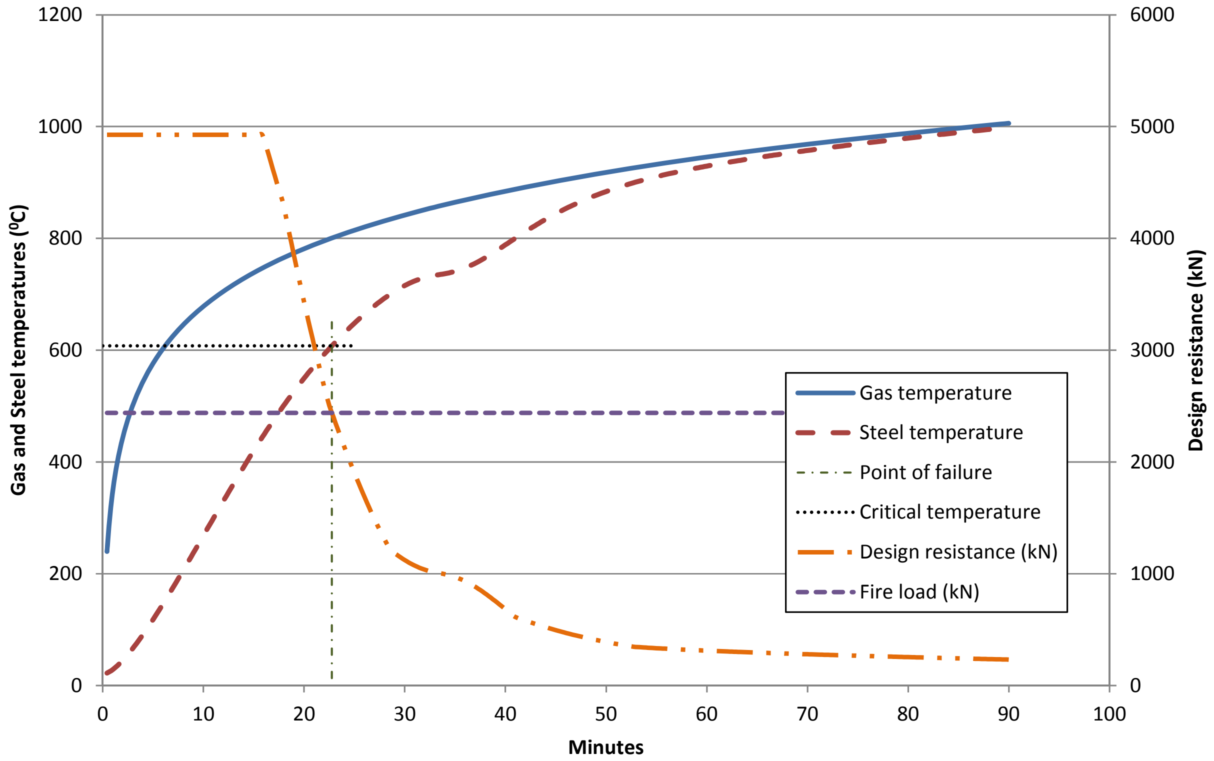


**Figure 2.3 Variation of gas temperature, steel temperature and design resistance with time – unprotected column**

## 2.7 Design resistance of protected column at elevated temperature

Try encasing the beam with 15 mm of fire protection board.

The temperature increase in a protected member in time interval $\Delta t$ 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:

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

(all nomenclature as previously defined)

$\lambda_p$, $c_p$, and $\rho_p$ are taken from the manufacturer's data.

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

| | | |
|---|---|---|
| Thermal conductivity | $\lambda_p$ | = 0.2 W/mK |
| Thickness | $d_p$ | = 15 mm |
| Density | $\rho_p$ | = 800 kg/m$^3$ |
| Specific heat | $c_p$ | = 1700 J/kgK |

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

In this instance, $A_p/V$ is equal to the box value of the section factor = 63.5 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 90 minutes, the resistance of the column is 3716 kN. The steel temperature at 90 minutes is 534°C, less than the critical temperature of 608°C.

As shown in Figure 2.4, the resistance of the protected column only reduces after 70 minutes. Note in Figure 2.4, the resistance has been limited to 4930 kN, the design resistance at ambient temperature.

As the design resistance (3716 kN) exceeds the design effect (2436 kN) the selected solution is satisfactory.

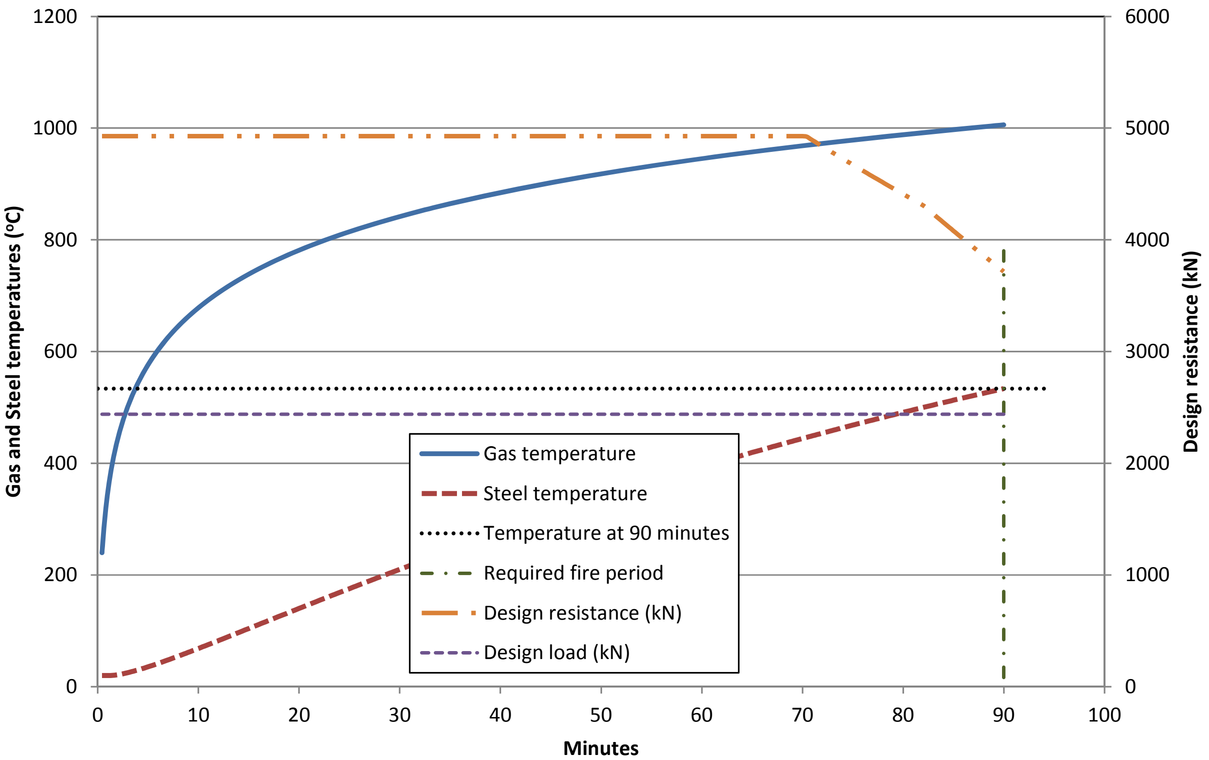


**Figure 2.4** Variation of gas temperature, steel temperature and design resistance with time – unprotected column