Fire protection of steelwork

In this first of two technical articles, David Brown of the Steel Construction Institute gives some general background on fire protection and demonstrates how the guidance used by designers to specify fire protection for beams has been developed. Part 2 will consider the protection of columns.

Critical temperature

According to BS EN 1991-2[1], the temperature of a so-called standard fire rises rapidly and continues to increase with time. Unprotected steel begins to lose strength above 400°C and at 1006°C (the standard fire temperature at 90 minutes) has only 4% of its original strength. Apart from some specific cases, steel will generally need protection to limit the reduction in strength. This article assumes the protection is an intumescent coating.

Protection is specified to limit the steel temperature to a maximum value, known as the critical temperature. A higher critical temperature will mean less protection is required; a lower critical temperature will mean more protection is required. The critical temperature therefore has an important influence on cost and time, since more protection often means more coats and longer time to cure between coats.

The critical temperature can be calculated, but many designers appear to use the tabulated values published by ASFP[2]. Others may use the tabulated values provided in the UK NA to BS EN 1993-1-2[3]. Others appear to leave the specification entirely to the coating manufacturer. For the critical temperatures tabulated by ASFP, manufacturers provide tables of required protection thickness, for different periods of fire protection and for different values of A_m/V (equivalent to H_p/A). For temperatures not given in the ASFP tables, the manufacturer must be consulted.

The values of critical temperature published by ASFP and in the UK NA differ and are presented in different formats. The background to the tabulated values is opaque. The aim of this article is to explain how the values in both documents were calculated, demonstrate that the values are generally (but not always) conservative and encourage designers to take proper responsibility for this important aspect of design.

Both ASFP and the UK NA provide values of critical temperatures for beams and columns. Beams are (or should be) more straightforward since in both documents they are assumed to be restrained. This important limitation in scope is however not mentioned.

Utilisation

Utilisation is a measure of how hard the beam is working (strength, not deflection), which might be referring to the situation at ambient temperatures, or at elevated temperatures – it is essential to know!

At elevated temperatures, three factors influence the degree of utilisation. Firstly, the design value of actions are reduced, secondly a non-uniform temperature through the section can be of benefit and finally the member may not have been fully utilised at ambient temperatures – it has spare resistance which can be used in the fire design situation.

Reduced effects of actions in the fire limit state.

In the fire condition, the design values of forces and moments are reduced by applying a factor, η_{fi}. The reduction factor represents the characteristic permanent actions and a reduced value of the characteristic variable actions – effectively implying that not all the variable action will be applied in a fire, which seems sensible.

If the original design combination had been calculated using expression 6.10 of BS EN 1990, the factor η_{fi} is given by:

$$\eta_{fi} = \frac{G_k + \Psi_{fi} Q_k}{\gamma_G G_k + \gamma_Q Q_k}$$

, which is expression 2.5 in BS EN 1993-1-2.

There are similar expressions (2.5a and 2.5b) if the load combinations had originally been determined using expressions 6.10a and 6.10b of BS EN 1990.

The UK NA to BS EN 1991-2 specifies that $\Psi_{fi} = \Psi_1$, which is to be taken from the UK NA to BS EN 1990. Typical values of Ψ_1 for different categories of loading are:

- For offices, $\Psi_1 = 0.5$
- For shopping areas, $\Psi_1 = 0.7$
- For storage, $\Psi_1 = 0.9$

Looking at the expression for η_{fi}, it is clear that the computed answer depends on the ratio Q_k:G_k and also on the value of Ψ_1. Designers might then observe that:

1. The ASFP document provides different limiting temperatures for offices, shopping and storage categories – but does not define Q_k:G_k
2. The UK NA to BS EN 1993-1-2 offers no categorisation of loading and no definition of Q_k:G_k

BS EN 1993-1-2 offers a helpful figure showing how η_{fi} varies with the ratio Q_k:G_k and the value of Ψ_1. For the three categories of loading and values of Ψ_1 = 0.5, 0.7 and 0.9, this relationship is shown in Figure 1.

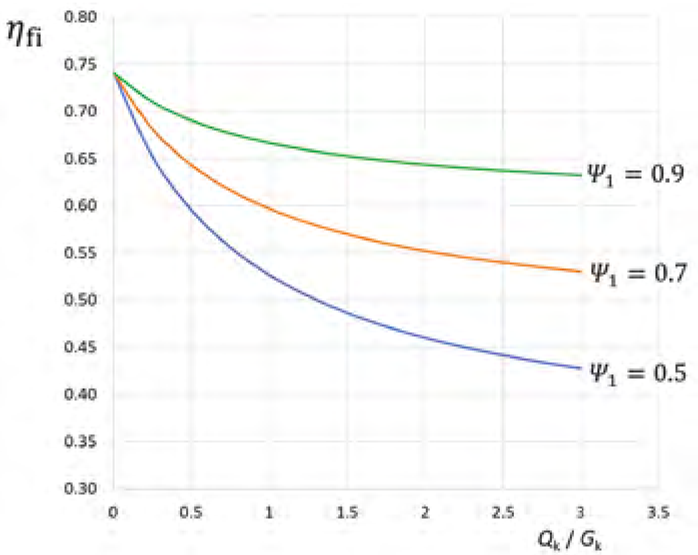

Table 1 – Reduction factor η_{fi}

NOTE 2 to Figure 2.1 in BS EN 1993-1-2 allows the use of $\eta_{fi} = 0.65$ except for storage. This conservative value should not used as it will result in unnecessary protection being specified. For a typical Q_k:G_k ratio of 1:1 and Ψ_1 (office), the value of η_{fi} is 0.53.

Non-uniform temperature through the cross section.

If a beam supports a slab, the top flange is protected to some degree. BS EN 1993-1-2 allows for this by introducing an adaptation factor, κ_1. The values are:

- For a beam exposed on four sides (i.e. no slab), κ_1=1.0
- For an unprotected beam exposed on three slides and a slab on side four, κ_1 = 0.7
- For a protected beam exposed on three slides and a slab on side four, κ_1 = 0.85

An additional factor κ_2, will generally be 1.0.

In the fire condition, the moment resistance of a beam is the moment resistance at ambient temperature, divided by $\kappa_1\kappa_2$. When κ_1< 1, this produces an enhanced value of the moment resistance.

In Table NA.1 of the UK NA to BS EN 1993-1-2, the reason for three descriptions of beams should now be clear – the three categories reflect ▸26

▸24 the three values of κ_1 above.

Confusingly, Table 16 of the ASFP guide has "non-composite beams carrying concrete floor slabs" and "composite beams supporting floor slabs", which both have a concrete slab. The difference between non-composite and composite in the ASFP table is discussed later.

Utilisation at ambient temperatures

Clearly, if a member has a surplus of resistance at ambient temperatures, those reserves of strength will be useful at elevated temperatures.

Calculation of the critical temperature

Reading BS EN 1993-1-2, designers might be tempted to use expression 4.22 to calculate the critical temperature (as it falls under the clause 4.2.4 "Critical temperature"). Once the utilisation μ_0 has been determined, the critical temperature $\theta_{a,cr}$ is given by:

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

As an alternative, both the ASFP values and those in the UK NA are based on the necessary steel strength to carry the reduced design actions in the fire condition, which will of course be less than the nominal yield strength. Having determined the reduced strength required to carry the design loads, Table 3.1 of BS EN 1993-1-2 which shows reduced steel strength vs. temperature can be interrogated to determine at what elevated temperature the calculated reduction in steel strength occurs. This temperature is presented in the ASFP guide and in the UK NA as the critical temperature.

A comparison of the two alternatives is shown in Figure 2. As can be seen, the relationship between strength reduction and temperature is almost identical. If trying to reproduce the precise values in the ASFP document or the UK NA, it is important to note that the second process involving Table 3.1 is used.

Beams in the UK NA to BS EN 1993-1-1

The relevant part of Table NA.1 is reproduced below. To help understand the tabulated temperatures, the value of κ_1 has been added to the relevant row.

Description of member		Critical temperature (°C) for utilisation factor μ_0					
	κ_1	0.7	0.6	0.5	0.4	0.3	0.2
Protected beams with slabs	0.85	558	587	619	654	690	750
Unprotected beams with slabs	0.7	594	621	650	670	717	775
Beams with no slab	1.0	526	558	590	629	671	725

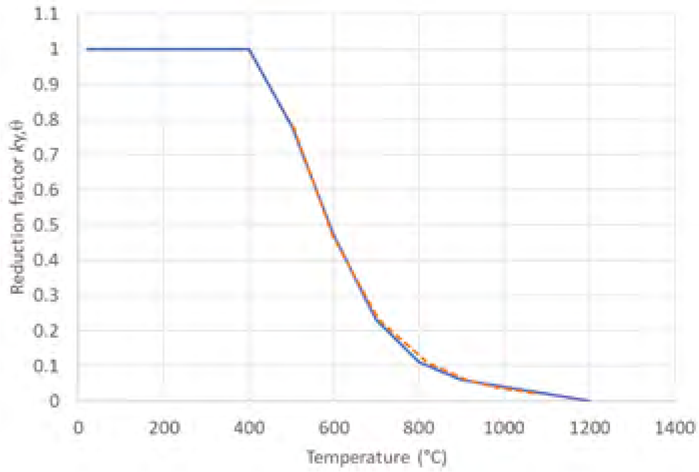

Figure 2: Reduction in steel yield strength

Example 1: Protected beam with slab, μ_0 = 0.6

In this example, κ_1 is 0.85 and enhances the moment resistance in the fire condition, which is equivalent to reducing the utilisation. The effective utilisation is therefore 0.6 × 0.85 = 0.51

From Figure 2 it can be seen that the steel reaches 51% of its original strength at a temperature just below 600°C. The precise figure, obtained by linear interpolation from the values in Table 3.1, is 587°C, as tabulated above.

Example 1: Protected beam with slab

In this example, the value of μ_0 = 0.53, as calculated above for an office with Q_k:G_k = 1:1

If it is assumed that for some reason, the beam is not fully utilised at ambient temperature, but is only utilised 90%, the effective utilisation becomes 0.53 × 0.85 × 0.9 = 0.41

In this case, the critical temperature is 651°C, so the requirement for protection is reduced compared to example 1.

ASFP critical temperatures

The background to the ASFP critical temperatures is quite different to the approach in the UK NA. The UK NA requires the designer to calculate the utilisation, μ_0. In contrast, for different loading categories, ASFP have already calculated what is considered to be an appropriate value of μ_0 (although the value is not tabulated). The ASFP approach assumes that the beam is fully utilised at ambient temperatures – there is no opportunity to allow for any under-utilisation.

The relevant part of Table 16 from the ASFP Yellow Book is shown below (critical temperature in °C).

Building type	Non-composite beams carrying concrete floor slabs	Composite beams supporting concrete floor slabs
Office / Domestic	603	576
Storage	576	544
Shopping / Congregational / Car Park	583	553

ASFP utilisations

In the "office" loading category, ASFP assume Q_k:G_k = 1:1 and use expression 2.5b for η_{fi} (which includes ξ = 0.925).

The utilisation is therefore $\dfrac{1 + 0.5 \times 1}{0.925 \times 1.35 \times 1 + 1.5 \times 1} = 0.546$

If the beam is protected and has a slab, then κ_1 = 0.85 and the effective utilisation becomes 0.546 × 0.85 = 0.464

From Figure 2, the critical temperature can be seen to be approximately 600°C. The precise value is 603°C, as tabulated above, under the heading "Non-composite beams carrying concrete floor slabs".

For composite beams, ASFP adopt the guidance in clause 4.3.4.2.3 of BS EN 1994-1-2, which indicates that the temperature in the steel section is assumed to be uniform, meaning that κ_1=1.0

If κ_1 is set to 1.0, the tabulated value of 576°C is calculated.

The UK NA sees no need to discriminate between composite and non-composite beams. It does seem rather odd that in the ASFP guidance a beam designed compositely is considered a more onerous condition than a non-composite design when both are supporting a slab.

Designers should note the assumed value of Q_k:G_k = 1:1 for the "office category". If the ratio was, say 0.8:1, the critical temperature reduces from 603°C to 595°C. If the ratio changes in the opposite direction ($Q_k > G_k$), the value of 603°C is conservative. The ratio assumed for shopping areas is also Q_k:G_k = 1:1.

For storage, the assumed ratio is Q_k:G_k = 1:2 and the calculation for η_{fi} uses expression 2.5a from BS EN 1993-1-2, since this is more onerous than the result from expression 2.5b.

The utilisation is therefore $\dfrac{1 + 0.9 \times 2}{1.35 \times 1 + 1.5 + 1.0 \times 2} = 0.644$

(the utilisation according to expression 2.5b is 0.659)

If κ_1 = 0.85, the tabulated value of 575°C is calculated and if κ_1 = 1.0, the tabulated value of 544°C.

Conclusions from Part 1

Consider a composite beam in a multi-storey office building (a very typical example).

The ASFP guidance leads to a critical temperature of 576°C. This approach has the benefit of simplicity. As demonstrated above, the (unstated) utilisation is 0.546. The UK NA invites the designer to determine the utilisation. If the same utilisation is used, interpolation in Table NA.1 leads to a less onerous critical temperature of 602°C. The difference is because ASFP assume a uniform temperature through the cross section (κ_1 = 1.0) and the UK NA takes the benefit of a protected top flange (κ_1 = 0.85).

If the ratios Q_k:G_k assumed by ASFP reduce, the critical temperatures are not conservative.

Neither the ASFP nor UK NA values are appropriate for unrestrained beams.

Best practice is to calculate the actual utilisation – including any overdesign at ambient temperatures – and the critical temperature, which is not at all difficult. Alternatively, sufficient information must be provided so that the critical temperature can correctly determined by others. This must include the Q_k:G_k ratio, the loading category and the utilisation at ambient temperature. ■

References

1. BS EN 1993-1-2:2005 Incorporating Corrigenda Nos 1 and 2 Eurocode 3: Design of steel structures – Part 1-2: General rules – Structural fire design BSI, 2006
2. ASFP Yellow Book Fire protection for structural steel in buildings 5th Edition (Volume 1 of 2) ASFP, 2018
3. UK NA to BS EN 1993-1-2:2005 UK National Annex to Eurocode 3: Design of steel structures Part 1-2: General rules – Structural fire design, BSI, 2008