Shelf angle floor beams in fire

Mark Lawson, Consultant to The Steel Construction Institute, discusses the resistance of unprotected shelf angle floor beams at elevated temperatures.

Composite floor slabs used in light steel construction are often supported by steel beams that are partially encased in the concrete slab. These beams may be required for longer spans or where walls do not align at different levels. They usually have side angles to support the slab and are known as 'shelf angle floor beams' and provide at least 30 minutes inherent fire resistance. These beams may also be fire protected by a plasterboard ceiling, and by intumescent coating or box protection for longer periods of fire resistance.

Shallow shelf angle floor beams are often designed for serviceability criteria, which means the design moment at the fire limit state is a relatively low proportion of the bending resistance of the beam. In these situations, it may be possible to verify an unprotected solution for 60 minutes fire resistance by calculating the reduced resistance based on a temperature profile through the cross section. Tabulated temperatures from the standard or temperatures determined from a software analysis may be used to determine the temperature profile through the section.

A typical case of a shelf angle floor beam is shown in Figure 1, in which a 170 mm deep composite slab is supported on 150 × 90 × 10 mm thick angles welded or bolted to the sides of a 254 × 254 × 107 kg/m UC beam used to minimise the overall floor depth. The decking has crushed ends in this case and so provides a solid block of concrete next to the beam web.

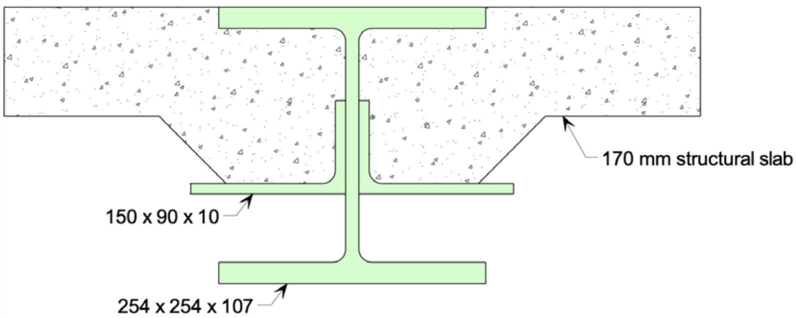

Figure 1: Cross-section through a 254 × 254 UC beam and 150 × 90 shelf angles supporting a composite slab with decking supplied with crushed ends

The fire resistance of shelf angle floor beams using UB or UC sections is given in Annex C to BS 5950-8[1]. Design guidance on the fire resistance of shelf angle floor beams in accordance with BS 5950-8 is provided in SCI publications P080[2] and P126[3]. This design process is considered to be satisfactory for Eurocode designs by taking the strength reduction factors ($k_{y,\theta}$) from the Eurocode.

The verification involves calculating the reduced plastic moment resistance of the section, including the continuous shelf angles, at elevated temperatures. The plastic moment resistance uses the strength reductions of the various elements of the cross-section. The strength reduction factors in BS EN 1993-1-2[4] may be used to replace those given in BS 5950-8, (both standards are similar). The reduction factor for the design load at elevated temperatures is also given in BS EN 1993-1-2.

In light steel construction, the top of the steel section is generally cast level with the top of the slab and most of the steel section is encased by the slab and is therefore relatively cool so that its full tensile strength can be developed. The difference with respect to solid slabs is that the outer part of the angle is exposed between the ribs of the decking, which often is supplied with crushed ends for this application. Therefore, it is recommended that the contribution of the outer 50 mm of the angle should be taken as the same as for the exposed bottom flange. There is value in performing a thermal analysis by finite element modelling to be able to predict the precise temperature distribution for a particular configuration if it is to be used regularly.

Design of shelf angle floor beams in fire

The design of shelf angle floor beams in fire is presented in Annex C of BS 5950-8. Temperatures are defined for various segments of the cross-section (known as 'blocks'). These are given as:

- θ_1 - Bottom flange
- θ_2 - Exposed web of beam
- θ_3 - Bottom leg of angle
- θ_R - Angle root
- θ_4 to θ_6 - Encased web of beam and vertical leg of angle
- θ_7 - Top flange

These temperatures are presented in Table 1 as a function of B_e, which is the bottom flange width and D_e, which is the exposed depth of beam. The data for 30 and 60 minutes presented in Table 1 is reproduced from BS 5950-8 which also provides data for the 90 minutes case.

The temperature of the exposed bottom flange should be determined from Table 2 (over page), which is extracted from Table 10 of BS 5950-8 and is based on downstand beams supporting concrete slabs. This data is very conservative for shelf angle floor beams. Table 10 of BS 5950-8 only gives temperature

Table 1: Block temperatures (°C) in a shelf angle floor beam as a function of bottom flange temperature at 30 and 60 minutes fire resistance

Aspect ratio of exposed depth: width of beam	Fire resistance 30 min			Fire resistance 60 min		
	θ_2	θ_3	θ_R	θ_2	θ_3	θ_R
$D_e/B_e \le 0.6$	θ_1 - 140	475	350	θ_1 - 90	725	600
$0.6 < D_e/B_e \le 0.8$	θ_1 - 90	510	385	θ_1 - 60	745	620
$0.8 < D_e/B_e \le 1.1$	θ_1 - 45	550	425	θ_1 - 30	765	640
$1.1 < D_e/B_e \le 1.5$	θ_1 - 25			θ_1		
$D_e/B_e > 1.5$	θ_1					

θ_1 = bottom flange temperature - see Table 2 and exposed depth, $D_e = h-h_c$

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▶24 data up to 60 minutes, for longer durations or for less conservative temperature data, thermal modelling may be carried out. SCI can perform this modelling.

Flange Thickness, t_f	Fire resistance 30 min	Fire resistance 60 min
10 mm	772	938
15 mm	736	933
20 mm	714	925
25mm	676	909
30 mm	638	886

Table 2: Temperature θ_1 (°C) of the exposed bottom flange for a beam supporting a concrete slab

The temperature gradient in the web and vertical leg of the angle is given in Table C2 of BS 5950-8 as 2.3°C per mm for 30 minutes fire resistance, and 3.8°C per mm for 60 minutes fire resistance. Therefore, the depth of web with a temperature difference of 200°C over its depth is 53 mm for the 60 minute case. This may be approximated to 50 mm for analysis purposes.

The strength reduction factors for steel in Class 1 to 3 sections based on the effective yield strength at elevated temperatures are given in Table 3.1 of BS EN 1993-1-2 and are reproduced in Table 3.

Temperature (°C)	Strength reduction factor ($k_{y,\theta}$)
400	1.0
500	0.78
600	0.47
700	0.23
800	0.11
900	0.06
1000	0.04

Table 3: Strength reduction factor for steel for effective yield strength, $k_{y,\theta}$

The combined width of the two side angles should be wider than the beam in order provide a suitable bearing length of the slab. The temperature of the outer 50 mm of the exposed leg of the angle may be taken as the bottom flange temperature, θ_1, and for analysis purposes, its average temperature may be taken as $(\theta_1 + \theta_3)/2$.

The reduced bending resistance of the embedded steel section may be determined as follows:

- The plastic neutral axis depth is determined by equating the reduced tension and compression forces based on the cross-sectional areas of these elements multiplied by their strength reduction factors - see Figure 2. It is generally found that the plastic neutral axis lies at or close to the top flange of the steel section.
- The bending resistance is determined by taking moments of the reduced resistance of each element multiplied by the distance from the neutral axis. This includes the steel section and shelf angles but not the concrete.
- The reduced bending resistance in fire is then given as a ratio of the bending resistance of the steel section in normal conditions ignoring the shelf angles.

Simplified design of shelf angle floors beams in fire

The load ratio that may be supported in fire conditions depends on the shape and depth of the steel section, the relative cross-sectional area of the shelf angles and the depth of concrete. Lighter UB sections will benefit more from the effect of the partial encasement than heavy UC sections. For an approximate design, Table 4 may be used to obtain the maximum load ratio that may be applied at the fire limit state depending on the steel section and fire resistance period. Where data is not presented in this table, such as for the 90 minute fire resistance case with low load ratios, the precise configuration may be analysed by thermal modelling. This is cost-effective if the same or similar details are used in a large project or in other projects.

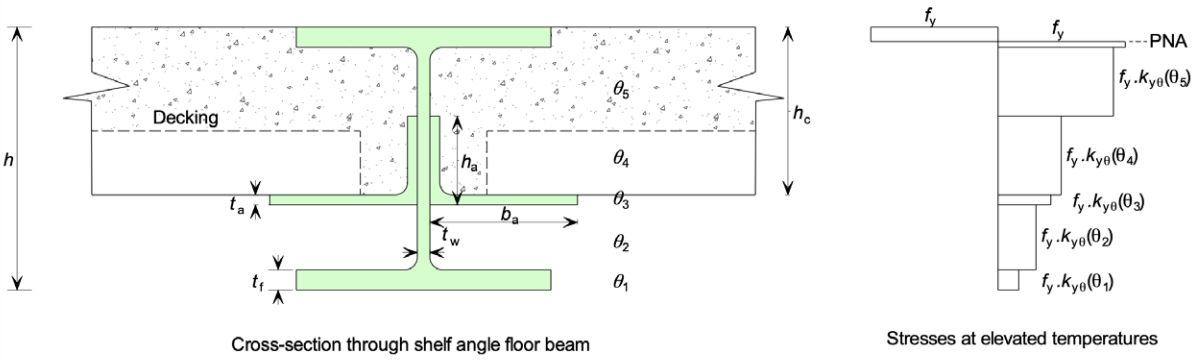

Figure 2: Plastic bending resistance of a shelf angle floor beam in fire using a UC section in this case

	UB Sections		UC Sections
Fire resistance	$h/h_c < 1.6$	$h/h_c < 2$	$h/h_c < 1.6$
	$M_{Rd,fi}/M_{Rd}$	$M_{Rd,fi}/M_{Rd}$	$M_{Rd,fi}/M_{Rd}$
30 mins	≤ 0.65	≤ 0.50	≤ 0.60
60 mins	≤ 0.35	≤ 0.25	≤ 0.30
90 mins	≤ 0.20	Not presented	Not presented

Table 4: Maximum load ratios for unprotected partially encased UB and UC sections with side angles supporting composite slabs

In Table 4:
$M_{Rd,fi}$ is the reduced bending resistance of the partially encased section in fire conditions
M_{Rd} is the bending resistance of the UB or UC section in normal conditions
h is the beam depth and h_c is slab depth

It is also generally the case that the peak temperature at the top of the flange does not exceed the limit for insulation at 60 minutes fire exposure provided the web thickness does not exceed 18 mm or as substantiated by thermal modelling.

Where, the exposed part of the steel section is fire protected, the required fire protection of a shelf angle floor beam or a partially encased beam may be determined from the section factor of the exposed part of the section.

This method does not apply for RHS sections with side angles or a welded bottom plate (known as a slim floor beam) and in this case, the temperatures should be obtained by thermal modelling as the temperatures in the web of the RHS will be higher than for a UC section. Nevertheless, 60 minutes fire resistance can often be achieved in the case of an RHS slim floor beam and 30 minutes for an RHS section with shelf angles.

1 BS 5950-8:2003. *Structural use of steelwork in building – Part 8: Code of practice for fire resistant design*, BSI, 2003

2 *Fire resistant design of steel structures – A handbook to BS 5950: Part 8*, SCI, 1990, Lawson, R, M and Newman, G, M. (available on Steelbiz)

3 *The fire resistance of shelf angle floor beams to BS 5950: Part 8* Newman, G, M., SCI, 1993, (available on Steelbiz)

4 BS EN 1993-1-2:2005. *Eurocode 3: Design of steel structures – Part 1-2: General rules – Structural fire design,* BSI, 2006