9 FIRE DESIGN

When subject to fire steel looses both its strength and stiffness. Steel structures also expand when heated and contract on cooling. furthermore the effect of restrained to thermal movement can introduce high strains in both the steel member and the associated connections.

Fire tests on steel structures have shown that the temperature within the connections is lower compare to connecting steel members. This is due to the additional material around a connection (column, end-plate, concrete slab etc.) which significantly reduces the temperatures within the connections compared to those at the centre of supported beam.

PrEN 1993-1-2 [prEN 1993-1-2: 2003] gives two approaches for the design of steel connections. In the first approach fire protection is applied to the member and its connections. The level of protection is based on that applied to the connected members taking into account the different level of utilisation that may exist in the connection compare to the connected members. A more detailed approach is used in the second method which uses an application of the component approach in prEN 1993-1-8 together with a method for calculation the behaviour of welds and bolts at elevated temperature. By using this approach the connection moment, shear and axial capacity can be evaluated at elevated temperature [Simões da Silva et al, 2001], [Spyrou et al, 2002].

Traditionally steel beams have been designed as simply supported. However it has been shown in recent large scale fire tests on the steel building at Cardington [Moore, 1997], in real fires [SCI recommendation, 1991], and in experimental results on isolated connections [El-Rimawi et al, 1997], that joints that were assumed to be pinned at ambient temperature can provide considerable levels of both strength and stiffness at elevated temperature. This can have a beneficial effect on the survival time of the structure.

Q&A 9.1 Bolts Resistance at High Temperature

How do you calculate the resistance of bolts at high temperature?

Experimental studies [Sakumoto et al, 1992], [Kirby, 1995] have shown that the strength and stiffness of a bolt reduces with increasing temperature. In particular they show a marked loss of strength between *300* and *700ºC*. The results of this work has been included in prEN 1993-1-2, where $k_{b,\theta}$ is used to describe the strength reduction with elevated temperature. $k_{b,\theta}$ is given in Figure 9.1. Table 9.1.

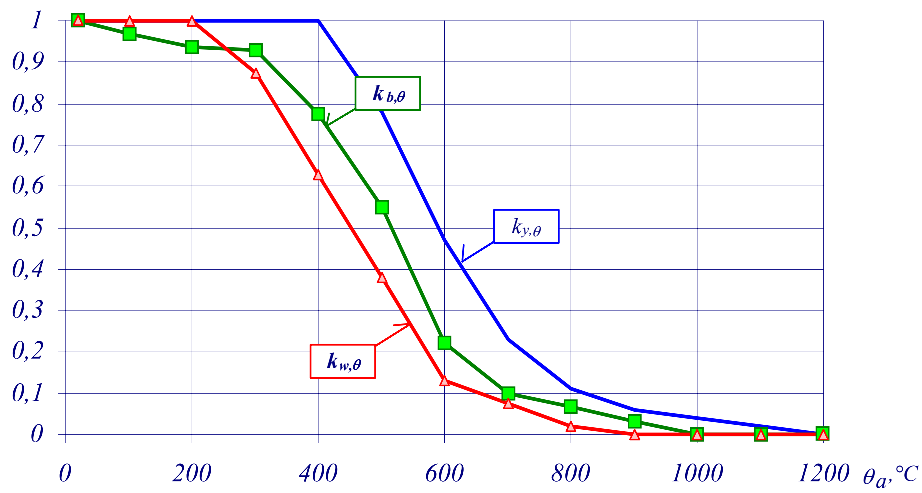


Figure 9.1 Reduction factor $k_{b,\theta}$ for bolt resistance, $k_{w,\theta}$ for weld resistance and $k_{y,\theta}$ for yield strength, prEN 1993-1-2

Table 9.1 Strength reduction factors for bolts,

Temperature θ_a	Reduction factor for bolts in tension and in shear $k_{b,\theta}$	Temperature θ_a	Reduction factor for bolts in tension and in shear $k_{b,\theta}$
20	*1,000*	500	*0,550*
100	*0,968*	600	*0,220*
150	*0,952*	700	*0,100*
200	*0,935*	800	*0,067*
300	*0,903*	900	*0,033*
400	*0,775*	1000	*0,000*

The shear resistance of bolts in fire may be evaluated using the following expressions

$$F_{v,t,Rd} = F_{v,Rd}\; k_{b,\theta} \frac{\gamma_m}{\gamma_{m,fi}}\,, \tag{9.1}$$

where γ_M is the partial safety factor for the resistance and $\gamma_{M,fi}$ is the partial safety factor for fire. The bearing resistance of bolts in fire may be predicted using

$$F_{b,t,Rd} = F_{b,Rd}\; k_{b,\theta} \frac{\gamma_m}{\gamma_{m,fi}} \tag{9.2}$$

and the tension resistance of a single bolt in fire is given by

$$F_{ten,t,Rd} = F_{t,Rd}\; k_{b,\theta} \frac{\gamma_m}{\gamma_{m,fi}}\,. \tag{9.3}$$

Q&A 9.2 Weld Resistance at High Temperature

How do you calculate the resistance of a welded connection under fire conditions?

The question consists of two parts: calculating the temperature distribution in the joints (see the answer into next question Q&A 9.3, [Franssen, 2002]) and calculating the weld resistance at high temperature. The design strength of a full penetration butt weld, for temperatures up to *700ºC*, should be taken as equal to the strength of the weaker part of the joint using the appropriate reduction factors for structural steel. For temperatures higher than *700ºC* the reduction factors given in prEN 1993-1-2 for fillet welds can be applied to butt welds. Design strength per unit length of a fillet weld in a fire may be calculated as

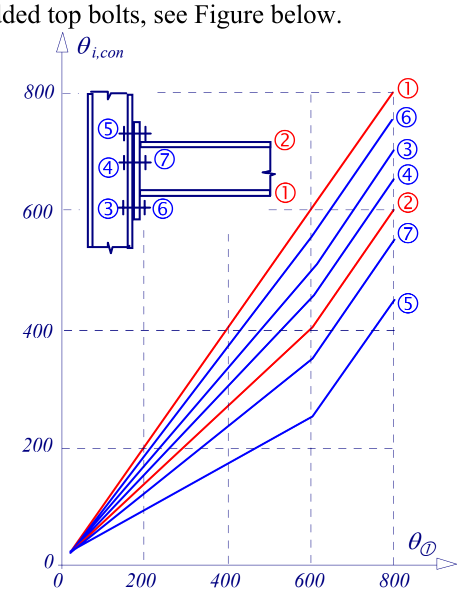
$$F_{w,t,Rd} = F_{w,Rd}\; k_{w,\theta} \frac{\gamma_m}{\gamma_{m,fi}}\,. \tag{9.4}$$

Q&A 9.3 Temperature Distribution with Time within a Joint

Can the simplified formulas of temperature distribution, indicated in prEN 1993-1-2, be used for all joints?

The thermal conductivity of steel is high. Nevertheless, because of the concentration of material within the joint area, a differential temperature distribution should be considered within the joint. Various temperature distributions have been proposed or used in experimental tests by several authors. According to prEN 1993-1-2, the temperature of a joint may be assessed using the local massivity value (*A/V*) of the joint components. As a simplification, a uniform distributed temperature may be assumed within the joint; this temperature may be calculated using the maximum value of the ratios *A/V* of the adjacent steel members. For beam-to-column and beam-to-beam joints, where the beams are supporting any type of concrete floor, the temperature may be obtained from the temperature of the bottom flange at mid span.

Table 9.2 Temperature distribution with time within joint

Reference	Temperature distribution
[Kruppa, 1976]	<u>Six joints types</u> The principal aim was to establish the performance of high strength bolts. The beams were not attached to the concrete slab. The results indicated that the bolt failure does not occur before large deformation of the others members.
[Leston-Jones et al, 1997]	<u>Double-sided joint with flush end-plate</u> Beam: 254x102x22; column: 152x152x23; 3 bolts M16 - 8.8. Furnace to follow a linear steel temperature path, reaching 900ºC in 90 min. average temperature profile for all tests: Lower beam flange $1{,}000\ \theta_{l,fb}$; Upper beam flange $0{,}677\ \theta_{l,fb}$; Beam centre web $0{,}985\ \theta_{l,fb}$; Top bolt $0{,}928\ \theta_{l,fb}$; Middle bolt $0{,}987\ \theta_{l,fb}$; Bottom bolt $0{,}966\ \theta_{l,fb}$; Column flange $1{,}036\ \theta_{l,fb}$; End plate $0{,}982\ \theta_{l,fb}$; $\theta_{l,fb}$ temperature of the lower beam flange.
[Al-Jabri et al, 1998] and [Al-Jabri et al, 1997]	<u>Steel and composite beam-to-column connections</u> Furnace to follow a linear steel temperature path, reaching 900ºC in 90 min. The hottest of the connection elements was the column web. Its temperature ranged between 8%-26% higher than beam flange temperature. The presence of the concrete slab above the connections caused a 20%-30% reduction in the beam top flange temperatures.
[Lawson, 1990]	<u>Eight beam-to-column connections</u> Typical of those used in modern framed buildings (steel and composite connections). Average temperature profile for all tests: $\theta_{lower\ beam\ flange} = 650\ ºC$ and $750\ ºC$ $\theta_{upper\ bolts} = 150\ ºC$ to $200\ ºC$ lower than $\theta_{lower\ beam\ flange}$ $\theta_{lower\ bolts} = 100\ ºC$ to $150\ ºC$ lower than $\theta_{upper\ bolts}$.
[Liu, 1996]	<u>Double-sided composite joint with extended end-plate connection</u> Numerical modelling, temperature profile: *For* ≈ 45 *min:* $\theta_{lower\ bolts} \approx 650\ ºC$; $\theta_{end\ plate} \approx 550\ ºC$; $\theta_{upper\ bolts} \approx 520\ ºC$ $\theta_{column\ web} \approx 450\ ºC$; $\theta_{unexposed\ bolt} \approx 350\ ºC$.
[SCI recommend, 1990]	<u>Joint with extended end-plate</u> Considering embedded top bolts, see Figure below. 
[El-Rimawi et al, 1997]	Assumptions: $\theta_{upper\ beam\ flange} = 0{,}7\ \theta_{lower\ beam\ flange}$; $\theta_{upper\ beam\ flange} = 0{,}7\ \theta_{beam\ web}$

Applying the expressions referred to in prEN 1993-1-2, see Figure 9.2, the temperature of the joint components may be determined as follows:
The depth of the beam is less than *400 mm*

$$\theta_h = 0{,}88\,\theta_0\left[1 - 0{,}3\,(a/h)\right], \tag{9.5}$$

where θ_0 is temperature of the lower beam flange at mid span. The depth of the beam is greater than *400 mm*

$$\theta_h = 0{,}88\,\theta_0 \qquad a \text{ is less than } h/2 \tag{9.6}$$

$$\theta_h = 0{,}88\,\theta_0\left[1 + 0{,}2\,(1 - 2a/h)\right] \qquad a \text{ is grater than } h/2 \tag{9.7}$$

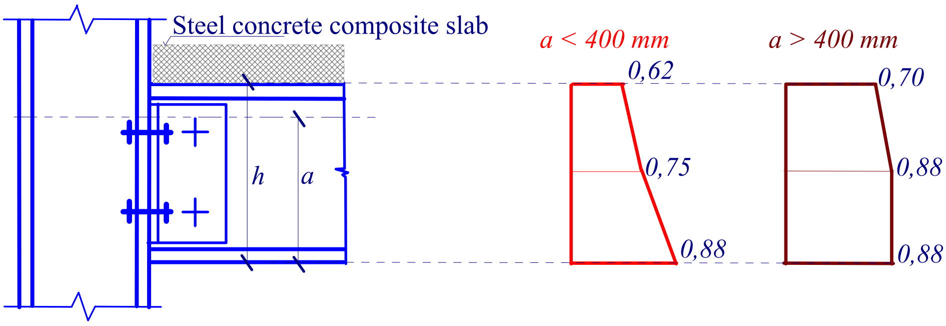


Figure 9.2 Thermal gradient within the depth of connection

With the aim to quantify the temperature distribution within a joint, some tests have been done in several joint typologies. Table 9.2 summarizes the results, showing that for deeper beams a web temperature similar to bottom flange temperature is observed while for small beams a smaller web temperature is observed. Additionally, the presence of the concrete slab above the joint cause a reduction in the beam top flange temperatures. A detailed description can be found in the literature [Kruppa, 1976], [Leston-Jones et al, 1997], [Al-Jabri et al, 1998], [Al-Jabri et al, 1997], [Lawson, 1990], [Liu, 1996], [SCI recommendation, 1990], [El-Rimawi et al, 1997]. The values proposed in prEN 1993-1-2, are in agreement with these experimental results. However, these values are based on the standard fire curve ISO834; if the fire follows other curves, such as hydrocarbon, external fire or a natural fire scenario, it is necessary to analyse the particular case, using a numerical or experimental study.

Q&A 9.4 Component Method under High Temperatures

At ambient temperature, besides presenting rules for the calculation of the resistance of bolts loaded in shear, in bearing and in tension and the design resistance per unit length of a fillet weld, prEN 1993-1-8 presents design rules to determinate the behaviour of the complete joint. Can the method be used at high temperature?

The component method, see [Zoetemeijer, 1974] and [prEN 1993-1-8: 2003], that consists of the assembly of extensional springs and rigid links, may be adapted and applied to the evaluation of the behaviour of steel joints under elevated temperatures. Depending on the objective of the analysis, a simple evaluation of resistance or initial stiffness may be pursued or, alternatively, a full non-linear analysis of the joint may be performed [Simões da Silva et al, 2001], taking into account the non-linear load deformation characteristics of all the joint components, thus being able to predict the moment-rotation response, see Figure 9.3.

To evaluate the non-linear response of steel joints in fire, knowledge of the mechanical properties of steel with increasing temperature is required. In the context of the component method, this is implemented at the component level. The elastic stiffness, K_e, is directly proportional to the Young's modulus of steel and the resistance of each component depends on the yield stress of steel. Equations (9.8) to (9.10) illustrate the change in component force-deformation response with increasing temperature for a given temperature variation θ of component i.

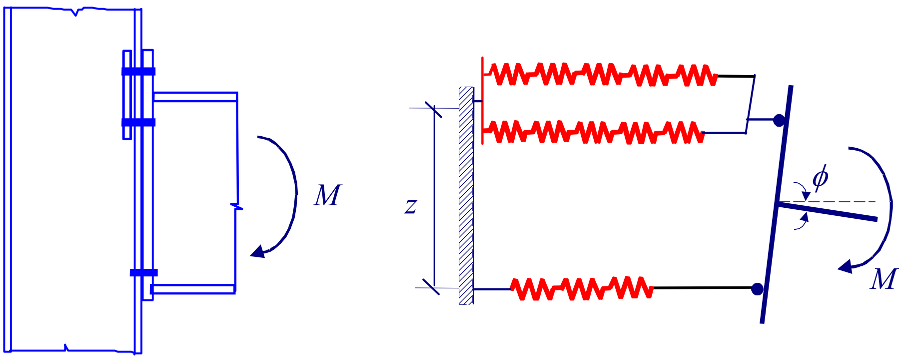


Figure 9.3 Component method applied to a typical beam-to-column joint, a) joint, b) component model

$$F_{i,y,\theta} = k_{y,\theta}\ F_{i,y,20°C}\ , \tag{9.8}$$

$$K_{i,e,\theta} = k_{E,\theta}\ K_{i,e,20°C}\ , \tag{9.9}$$

$$K_{i,pl,\theta} = k_{E,\theta}\ K_{i,pl,20°C}\ . \tag{9.10}$$

Introducing Equations (9.8) to (9.10) for the corresponding (constant) values of K_e, K_{pl} and F_y in any evaluation of moment-rotation response of steel joints at room temperature yields the required fire response. Implementation of this procedure allows the fire resistance to be established in any of two domains:

- Resistance - find reduced resistance at design temperature.
- Temperature - find critical temperature for loading and compare with design temperature.

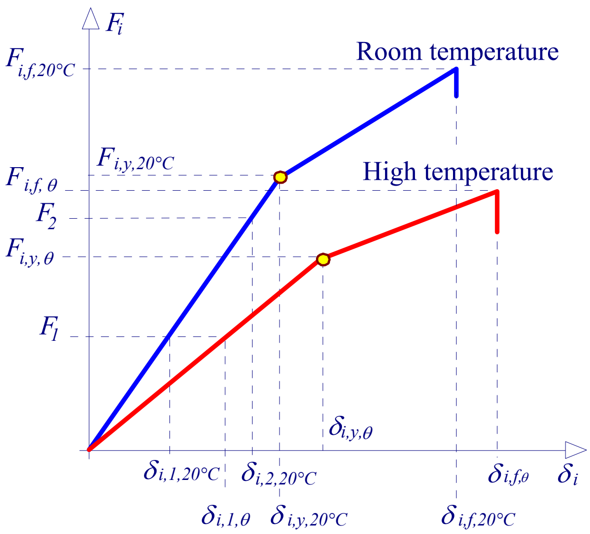


Figure 9.4 Isothermal force-deformation response of component

Resistance

With reference to Figure 9.4, for a given level of applied force F, the component deformation $\delta_{i,\theta}$ is given for $F' < F_{i,y,\theta}$ as $$\delta_{i,\theta}(F_1) = \delta_{i,1,\theta} = \frac{F_1}{K_{i,e,\theta}} = \frac{F_1}{k_{E;\theta}\, K_{i,e,20°C}} = \frac{1}{k_{E,\theta}}\,\delta_{i,20°C}(F_1)\,, \tag{9.11}$$

for $F = F_{i,y,\theta}$ as $$\delta_{i,y,\theta} = \frac{F_{i,y,\theta}}{K_{i,e,\theta}} = \frac{k_{y,\theta}}{k_{E,\theta}}\delta_{i,y,20°C}\,, \tag{9.12}$$

for $F_2 \geq F_{i,y\theta}$ as $$\delta_{i,\theta}(F_2) = \delta_{i,2,\theta} = \delta_{i,y,\theta} + \frac{1}{k_{E,\theta}}\,\frac{\delta_{i,f,20°C} - \delta_{i,y,20°C}}{F_{i;f,20°C} - F_{i,y,20°C}}\left(F_2 - F_{i,y,\theta}\right). \tag{9.13}$$

From equilibrium considerations, the bending moment for a given level of joint deformation is given by

$$M_\theta = \underset{(r=1,2)}{F_{r,\theta}}\; z = k_{y,\theta}\, M_{20°C}\,. \tag{9.14}$$

Similar expressions can be derived for stiffness and rotation of the joint, and the initial stiffness of a joint loaded in bending, at temperature θ is given by

$$S_{1,\theta} = \frac{E_\theta\, z^2}{\sum_i \frac{1}{k_{i,\theta}}} = k_{E,\theta} \times S_{1,20°C}\,. \tag{9.15}$$

The rotation at yield of the component i follows from

$$\phi'_{i,y,\theta} = \frac{M_{i,y,\theta}}{S_{i,y,\theta}} = \frac{k_{y,\theta}\, M_{i,y,20°C}}{k_{E,\theta}\, S_{i,2,20°C}} = \frac{k_{y;\theta}}{k_{E;\theta}}\,\phi_{i,y,20°C}\,. \tag{9.16}$$

Equations (9.11) to (9.16) give the generic moment-rotation curve at a constant temperature θ where the yielding sequence of the various components is identified.

Temperature
For a joint under uniform temperature distribution, the critical temperature is defined as the maximum temperature of the joint corresponding to failure of the joint,

$$M_{j,Sd} = M_{j,max,\theta}\,. \tag{9.17}$$

According to prEN 1993-1-2 the evaluation of the critical temperature requires the calculation of the degree of utilization of the joint at time $t = 0$, μ_0, defined as the relation between the design effect of the actions for the fire design situation and the design resistance of the steel member, for the fire design situation, at time t. For the present case of steel joints, the degree of utilization is explicitly given by:

$$\mu_0 = \frac{M_{j,Sd}}{M_{j,max,20°C}}\,. \tag{9.18}$$

Using Equation (9.18) allows the direct calculation of the critical temperature of the joint from Equation (9.28) [prEN 1993-1-2: 2003]

$$\theta_{cr} = 39{,}19\,ln\left[\frac{1}{0{,}967\,\mu_0^{3{,}833}} - 1\right] + 482\,. \tag{9.19}$$