CHAPTER

16

Fire-resistant Design

Introduction

The increase in the population of urban settlements coupled with the rising cost of urban land has resulted in a mushroom growth of multi-storied buildings in most of the Indian cities. Though the safety requirements of these buildings for strength and stability have been clearly defined in our codes, the information regarding fire safety is not adequate. While the municipal corporation authorities are stringent about the structural design and architectural layout of the buildings, they do not insist on strict compliance of the fire safety regulations (Kumar 2005). This was reflected during the fires in Gopala Towers, a 13-storey building, in 1983 and in Vigyan Bhavan, in 1993, in New Delhi. Such major fires have also occurred in the past in several parts of the country (e.g., the LIC Building fire in Madras in 1975, the fire in a school building in Kumbakonam in 2004) and got attention only for a short period.

The recent collapse of the twin-tower, 110-storey World Trade Center building in New York, USA, due to the terrorist attack (crashing two Boeing 767 aeroplanes into the building triggered explosions and fire) and subsequent fire (the flame temperature of 1727°C of the aeroplane fuel melted and buckled the steel columns) has renewed the interest in the fire-resistant design of structures (Subramanian 2002).

Traditionally, building codes specify regulations for buildings to be designed in such a way that they exhibit an acceptable level of performance in the event of fire (IS: 1641-1988, IS: 1642-1989, IS: 1643-1988, IS: 1644-1988). Essentially, these regulations are only concerned with the prevention of premature collapse, the provision to evacuate occupants from the structure on fire, avoiding the spread of the fire to adjacent properties, thus reducing the risk to surrounding properties and their occupants. Thus, these regulations are not concerned with the effect of the fire on the materials of the structure—this is a matter of concern only to the insurance companies and the owner of the property.

The earliest method of fire protection to steel structures was to encase the members with concrete. However, this method is not used now, since the concrete encasing increases the dead weight of the structure and results in enlarged member sizes and foundations. Moreover, due to the time required for casting and curing of concrete, the construction schedule gets delayed, resulting in an extra expenditure to the owner. Hence alternate methods in the form of plaster or gypsum spray for beams, plasterboard encasement for columns, and intumescent paints have been developed. It has to be noted that the fire protection of steel members is expensive. The typical cost of fire protection for multi-storey office blocks is around 15%–20% of the total costs; for the steel frame, the cost is around 10%–15% (Martin & Purkiss 1992).

It is possible to avoid such *passive fire protection* if special methods of construction such as slimfloor or shelf-angle floors are used, the basic steel member is overdesigned to give sacrificial protection, or fire-resistant steel members are used. We can also use such unprotected steelwork where the temperatures developed in a fire are insufficient to cause the steelwork collapse. We will discuss these methods in this chapter. We will also discuss the methods to model a real fire and its effects on steel members.

16.1 Fire Research

Fire research and investigations of the fire performance of building elements date back to the nineteenth century, when the frequent and disastrous fires of buildings during accidental fires were first realized. Fire tests have been conducted at the National Institute of Standards and Technology, Gaithersburg, USA; Building Research Establishment, London; Lund Institute of Technology, Sweden, etc. These tests were conducted to obtain reliable data as to the exact fire resistance of building elements used in building practice and to give precise particulars regarding fire prevention, alarm, and extinguishing appliances (Petterson 1973; Petterson et al. 1976; Morris et al. 1988).

16.1.1 Characteristics of Fires

Fire is an exothermic chemical reaction, essentially one of oxidation of hydrocarbons, which is followed by generation of heat. The growth of the fire depends on the rate at which heat is liberated and the rate at which it is dissipated. The main factors that influence the temperature, magnitude, and distribution of the fire are (Petterson 1973; Roytman, 1975; Subramanian & Venugopal 1984) as follows:

1. Fire load
2. Fire load display (the position of fuel within the room)
3. Fuel type
4. The dispersion factor and particle shape of the fuel
5. Window opening area
6. The temperature, pressure, and relative humidity of air
7. The dimensions of the room
8. The thermal conductivity and diffusivity of the construction material
9. Radiation levels from both within the compartment and through the windows

16.1.2 Fire Growth

The fire in a compartment may start or initiate from within the compartment or from adjacent compartment on fire if the intervening partition fails to stop it from spreading. The decisive factors that affect the ignition, growth, and spread of fire in a building are shown in Fig. 16.1.

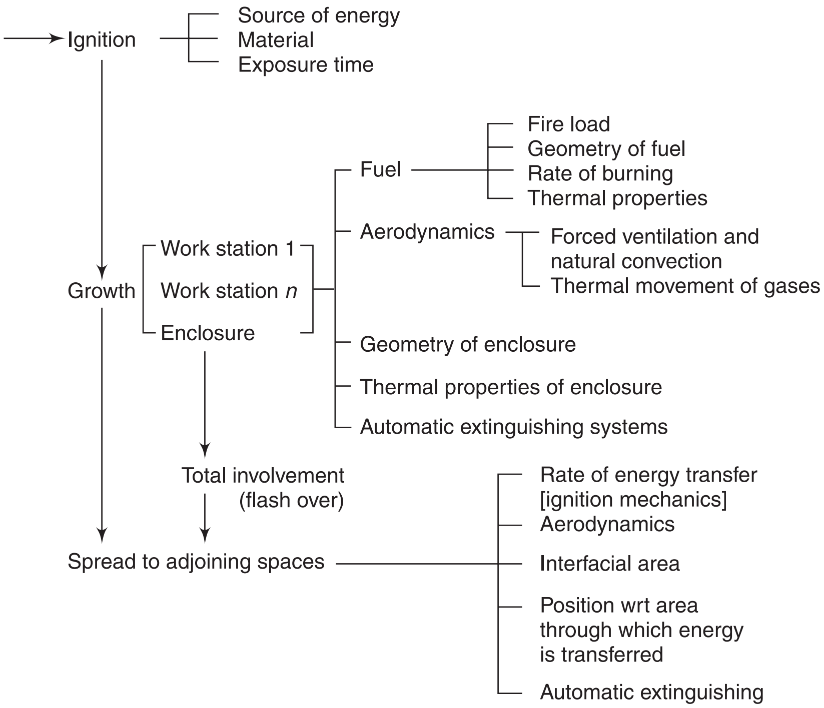


Fig. 16.1 Factors affecting ignition growth and spread of fire in building

It needs to be emphasized that the fire growth takes place mainly by radiation and surface flame spread, slowly or rapidly, leading to flashover. A model illustrating fire growth is given in Fig. 16.2, which shows the successive fire propagation from one workstation to another within a compartment. In this connection, the term *workstation* may be defined as a habitable area, usually associated with an activity that is related to a cluster of potential fuel, i.e., a furniture grouping or book shelf.

As shown in Fig. 16.3, the fire in a building can be divided into three phases: the growth or pre-flashover period, the fully developed or post-flashover period, and the decay period (ASCE/SEI/SFPE 29-99).

In the pre-flashover phase, the room temperature is low and the fire is local in the compartment. This period is critical for evacuation and fire-fighting. This phase has no significant effect on the structure. After the flashover phase, the fire enters into the fully developed phase; the temperature increases rapidly and the overall compartment is engulfed. During this phase, the highest temperature, the largest flame, and the highest rate of heating occur, leading to most structural damage and

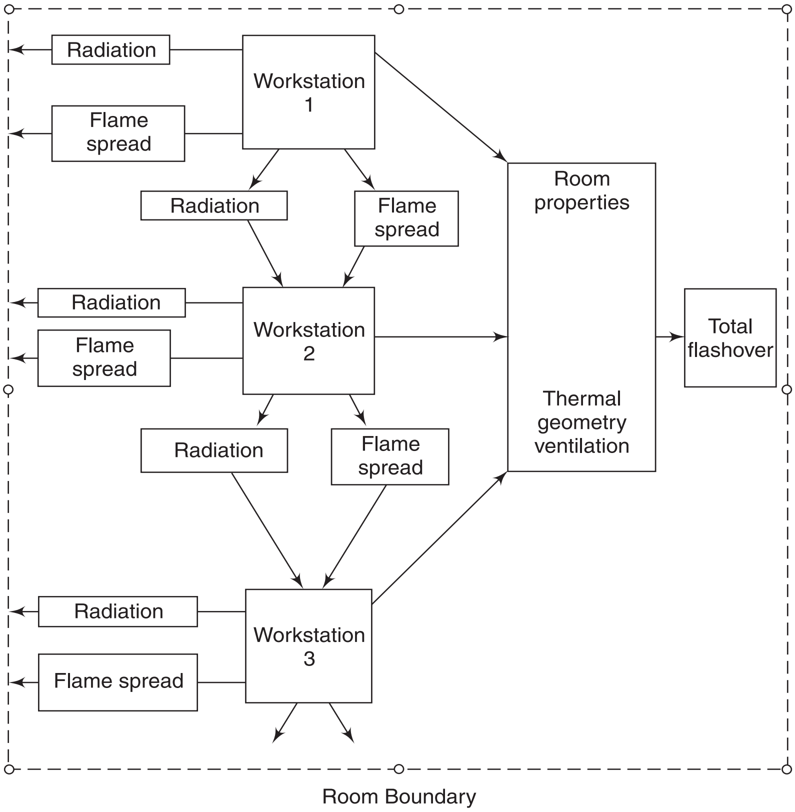


Fig. 16.2 Model of the fire growth process within a compartment

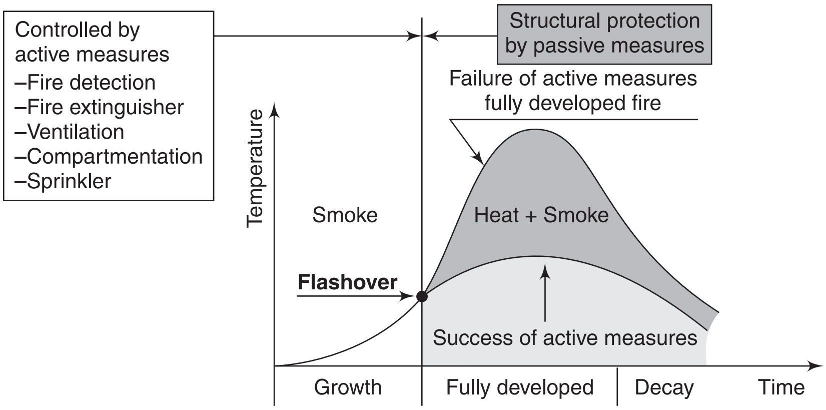


Fig. 16.3 Fire evolution and fire protection

fire spread. In the decaying period, the temperature decreases gradually. This period is important in the fire-resistant design, since the internal temperature of the cross sections of both insulated and unprotected steel members of a low section factor increase during this period (Lu & Maekelaeinen 2003).

16.2 Design Curves and Fire Models

When dealing with fire resistance, the ignition stage is generally neglected, although this stage is generally the most critical for human life. It is during this stage that toxic gases are produced and the temperature can reach 100°C and more. To select the relevant fire model, the fire scenarios need to be defined. It is a selection of a possible worst case as far as the location and the amount of fire load are concerned (Lu & Maekelaeinen 2003).

A design fire is expressed as a relationship between temperature, time, and space. (Buchanan 2001; Lu & Maekelaeinen 2003). It may be

- a nominal temperature–time curve uniform in space, or
- a 'real fire' either specified in terms of parametric time exposure, given by an analytical formula for localized fire, or obtained by computer modelling.

16.2.1 Nominal Temperature–Time Curves

The nominal temperature–time curves are a set of curves in which no physical parameters are taken into account. The main purpose of the prescription of the nominal curves is to reproduce the fire-resistance tests. The ability of the building elements to resist fire can be evaluated using such curve (Twilt et al. 1996).

Fire resistance times specified in most national building regulations relate to the test performance when heated according to an internationally agreed time–temperature curve defined in ISO 834 (or Eurocode 1 part 2-2). This standard temperature–time curve involves an ever-increasing air temperature inside the considered compartment, even when all consumable materials have been burnt and destroyed. This has become the standard design curve, used in the furnace testing of components (BS 476: 1987). This curve (see Fig. 16.4) is not really representative of the temperature build-up in a real fire and is regarded as a means of comparing performance rather than an absolute measure (Lu & Maekelaeinen 2003). Most of the European countries have standards similar to ISO 834 (e.g., BS 476 Parts 20-23) and the United States, Canada, and some other countries follow ASTM E119 fire which is similar to the ISO 834 standard fire.

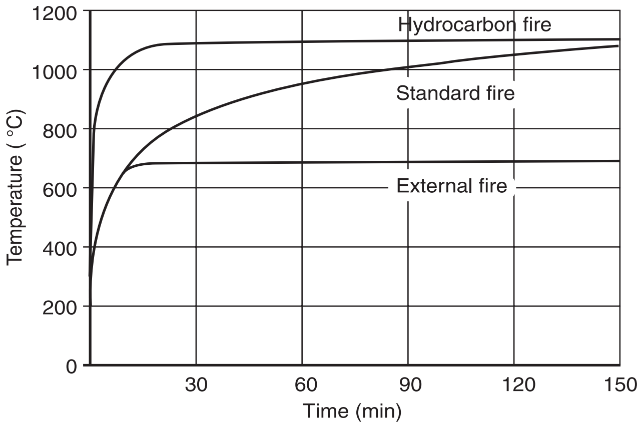


Fig. 16.4 Nominal temperature–time curves

The fire-resistance test is carried out in a furnace consisting of a large steel box lined with firebricks or ceramic fibre blanket. The furnace has a number of burners, most often fuelled with gas or fuel oil. It also has an exhaust chimney and several thermocouples for measuring gas temperature. For full-scale fire-resistance testing, a vertical wall furnace with a maximum size of 3.0 × 3.0 m is used.

The failure criteria for fire-resistance testing are stability, integrity, and insulation (Buchanan 2001). To meet the stability criteria, a structural element must perform its load-bearing function and carry the applied loads for the duration of the tests without any structural collapse. The integrity and insulation criteria are intended to test the ability of a barrier to contain a fire to prevent fire from spreading from the room of origin.

Fire resistance of building elements such as walls, beams, columns depends upon many factors, including the severity of the fire test, the material, the geometry, support conditions of the element, restraint from the surrounding structure, and the applied loads at the time of the fire. Due to the size of furnaces, the size of members that can be tested is limited. It has to be noted that tests on small members may be unrepresentative of the behaviour of larger members. Moreover, furnace tests cannot truly represent all restraint conditions (Lu & Maekelaeinen 2003).

Where the structure for which the fire resistance is being considered is external (which means the temperature of building materials will be closer to the corresponding fire temperatures), a similar external fire curve can be used. Where the storage of hydrocarbon materials makes extremely severe fires, a hydrocarbon fire curve is used, as shown in Fig. 16.4. The formula describing the standard temperature–time curve for the ISO 834 fire is

$$T_f - T_o = 345 \log (8t + 1) \tag{16.1}$$

where T_f is the furnace temperature, T_o is the ambient (room) temperature, and t is the time (in min). Formulae for the other two curves are given in Eurocode 3 (1995).

16.2.2 Natural Fire Models: Compartment Fires or Parametric Fires

Parametric fire models provide a simple means to take into account the most important physical phenomenon that may influence the development of the fire in a particular building. The parameters that are required to represent the time–temperature relationship in these models include the *fire load* present in the compartment, the openings in the walls and/or in the roofs, and the type and nature of different walls of the compartment. These models assume that the temperature is uniform in the compartment, which limits the application to post flashover fires in compartments of moderate dimensions. These models require the following data: fire load density, rate of heat release, and heat losses. Eurocode gives an equation for parametric temperature–time curves for any combination of fire load, ventilation openings, and wall-lining material. The parametric temperature–time curves considering the effect of openings and fire loads are shown in Figs 16.5 and 16.6. These curves are suitable for use as alternatives for the nominal curve for internal members of a compartment. (In these figures, A_f is the floor area, A_t is the total area

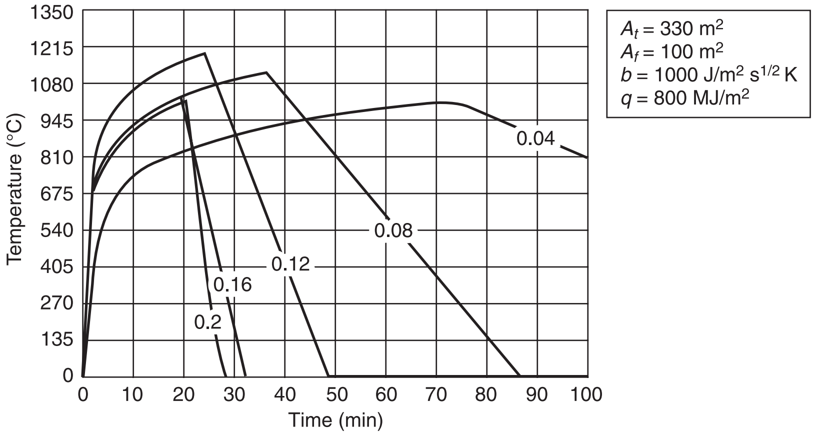


Fig. 16.5 Parametric temperature–time curves considering the effect of openings (Lu & Maekelaeinen 2003)

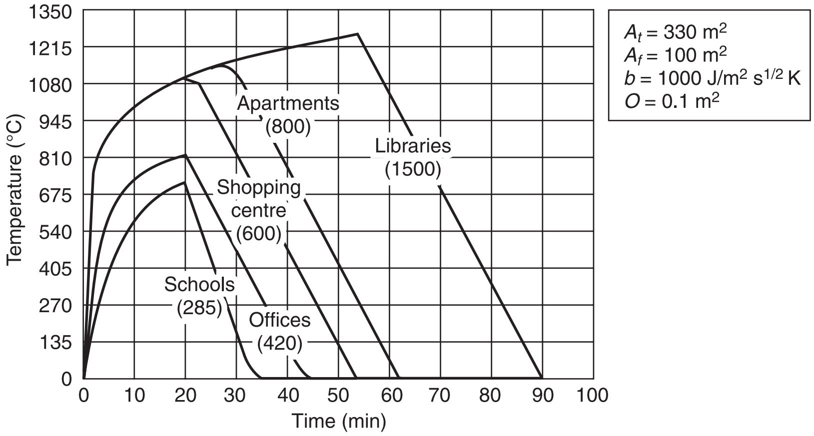


Fig. 16.6 Parametric temperature–time curves considering the effect of fire loads (Lu & Maekelaeinen 2003)

of enclosures (walls, ceiling, and floor), b is the thermal inertia of wall linings, q is the fire load, and o is the area of the opening.) The design using natural fires is normally restricted to special structures such as sports stadia, airport terminals, bus stations, and shopping malls surrounding atria, where the structure use does not change during its lifetime (Kirby 1986).

Where calculations are carried out using exposure to natural fires, it has been found that the temperatures reached in a fire are sufficiently low for the steelwork to be left unprotected (Lathan et al. 1987).

In addition to the above models, localized fire models and numerical models can be used to represent the natural fire. The details of these may be found in Lu and Maekelaeinen (2003), Twilt et al. (1996), and Eurocode 3 (1995).

The maximum temperature $T_{f,\max}$ reached in a natural fire is given by (Law 1983; Martin & Purkiss 1992)

$$T_{f,\max} = 6000\,[1 - \exp(-0.1\eta)]\eta^{-0.5} \tag{16.2a}$$

where

$$\eta = (A_t - A_w)/(A_w/H_w) \tag{16.2b}$$

where A_t is the total area of the containing compartment, A_w is the window area, and H_w is the window height.

The duration t_c of the natural compartment fire is given by (Law 1983; Martin & Purkiss 1992)

$$t_c = q_f A_f / \{0.18\, A_w \sqrt{H_w}\,(W/D)^{0.5}\,[1 - \exp(-0.36\,\eta)]\} \tag{16.3}$$

where q_f is the fire load/unit floor area expressed in terms of wood equivalent (see the next section). W/D is the width-to-depth ratio of the compartment, and A_f is the floor area. A_w and H_w have been defined earlier and η is defined as in Eqn (16.2b).

16.2.3 Fire Load

The fire load in a compartment is the maximum heat that can be theoretically generated by the combustible items and contents of the structure. It can be obtained either as an estimate for the total compartment or as a mass-weighted average for individual components within the compartment. It is calculated as the weight of the combustible material multiplied by the calorific value per unit weight and is expressed as MJ/m^2 or Mcal/m^2. The fire load is given by

$$Q_f = \Sigma m_v H_v / A_f \tag{16.4}$$

where Q_f is the fire load (MJ/m^2), A_f is the floor area (m^2), m_v is the total mass of the vth combustible material (kg), and H_v is the calorific value of the vth combustible material (MJ/kg). The fire load is also expressed in terms of the equivalent quantity of wood as kg wood/m^2 (1 kg wood = 18 MJ). Typical calorific values of common combustible materials and fire loads as per National Building Code—Part 4 and the New Zealand Code are given in Tables 16.1 and 16.2. The fire load survey conducted by Sunil Kumar and Kameswara Rao (1997) revealed that the fire load patterns of the Indian and UK office buildings were similar. They also found that the average fire load of all rooms is about 348 MJ/m^2 and the maximum fire load of 1860 MJ/m^2 occurs in storage and file rooms. It was also observed that the fire load is not dependent on the floor level of the building.

Table 16.1 Calorific values of common combustible materials (IS: 1641–1988)

Materials	Calorific Value (MJ/Kg)
Cloth (Average)	18.8
Kerosene	37.2
Leather	18.6
LPG	49.9
Paper (Average)	16.3
Plastic (Average)	22.1
Rubber	39.5
Wood (Average)	18.6

Table 16.2 Typical values of fire loads

Type of Compartment	Design fire load (MJ/m^2) as per NBC	Fire load as per New Zealand code (MJ/m^2)
Dwellings	450	400
Offices	450–900	800
Schools	450	
Assembly	450–900	
Industrial	up to 2700	
Storage and hazardous mercantile	up to 9000	
Retail occupancies	up to 4500	1200

(*Source:* Petterson et al. 1976)

The fire rating for steel structures could be calculated using

$$T_{eq}\ (\text{min}) = CWQ_f \tag{16.5}$$

where Q_f is the fire load, W is the ventilation factor, and C is a coefficient related to the thermal properties of walls, floors, and ceiling. Thus, for a building with large openings (W = 1.5) and highly insulating material (C = 0.09), T_{eq} works out to $0.09 \times 1.5 \times 150 = 20$ min.

16.3 Fire Engineering Design Problems

The study of structures for fire design and design provisions is called as *fire engineering*. The main problems encountered in the fire engineering design of structures are (Petterson et al. 1976)

1. Occupant protection
2. Personnel movement
3. Life support systems
4. Emergency communications and controls
5. Structural integrity
6. Fire protection

In this book, we are concerned only with the last two items. For the details of the other items, the readers may refer to Petterson (1973); Petterson et al. (1976) and Morris et al. (1988).

16.3.1 Fire Protection

Fire protection methods can be grouped into the following two broad groups:

1. Fire prevention designed to reduce the chance of a fire occurring.
2. Fire protection designed to mitigate the effects of a fire when it eventually occurs.

Fire prevention methods include the elimination of possible ignition sources and the protection of the structural members. According to the phase of fire development in which they are used, fire protection methods can be classified as active and passive protection.

Active protection methods include fire detection, alarm, extinguishing (fire extinguishers, fire hydrant, sprinklers), smoke control (smoke detection devices and venting features), and emergency exits (emergency staircases and lighting). The provision of adequate quantity of water for fire-fighting purposes and its delivery using suitable pumps at the required places at the required pressure is also necessary. (It has to be noted that several multi-storey buildings in India do not have emergency exits or staircase. The fire extinguishers kept in the buildings should be checked periodically; otherwise they will not be of any use. Efforts should be made to enable the fire-fighting department to be summoned promptly, to provide them with information as to the location and severity of fire and features of the building and facilities necessary to enable them quickly and safely tackle the fire.) All these methods can be operated either manually or automatically. These methods operate on the principle that early detection will lead to early fire-fighting and extinguishing of fire and reduce the risk of a large fire. For example, a combination of automatic sprinklers and smoke control systems has been used to help people to escape from fire in large multi-storey buildings. It has to be noted that these methods will not protect the structure completely from fire; these are designed mainly for the safe escape of people from buildings on fire (Roytman 1975; Nash & Young 1978).

Passive protection methods include structural fire protection, the layout of escape routes, the fire brigade access route, and control of combustible materials of construction (Bushev, 1976; Buchanan 2001). For pre-flashover fires, selection of suitable materials for the building contents and interior linings will prevent/reduce the spread of fire. In post-flashover fires, passive measures will result in sufficient fire resistance to prevent both the spread of the fire and the structural collapse.

Often combinations of the above methods are employed; though the employment of and emphasis on any one method will result in the reduced use of the remaining methods (e.g., sprinkler installation may lead to reduced overall requirement of the fire resistance). No codes allow a total trade-off for sprinklers, but many national codes allow a 50% reduction in the fire resistance of structural members if the building is provided with sprinklers. Eurocodes suggest that for calculating the fire resistance, the full load in a building provided with sprinklers be taken as 60% of the design full load.

Although steel is incombustible, its properties such as the yield strength and the modulus of elasticity are considerably reduced at high temperatures, which may result due to a major building fire. Due to the good thermal conductivity, the steel members may also ignite combustible material coming into contact with them. Hence it is necessary to provide some type of passive fire protection to the steel members if they are unable to withstand the fire loads. Some of the passive fire protection methods employed for structural steel members are discussed in Section 16.9.

16.4 Fire Engineering Design of Steel Structures

The study of steel structures under fire and their design provision is known as *fire engineering design.* The basic consideration is that the building should be capable of maintaining its structural integrity during a fire over a period of time (determined by the actual fire condition and the required safety level) and at the same time protect its occupants and provide them with a safe means to exit.

16.4.1 Fire Resistance Level

The required *fire resistance level* (FRL) is prescribed in the appropriate standards or building specifications or by the client. The FRL is specified in terms of the duration (in min) of standard fire load without collapse and depends on (a) the use to which the structure is put and (b) the time taken to evacuate the structure in the case of a fire.

In India, the FRL is specified by IS: 1641-1988, IS: 1642-1989, and IS: 1643-1988 and varies from 30 min to 4 hr depending on the above two parameters.

16.4.2 Period of Structural Adequacy

The period of structural adequacy (PSA) of a member is the time (in min) for a member to reach the limit state of structural adequacy when exposed to the standard fire test. It is the time for which the structural member will support the applied loads when subjected to a standard fire test. Thus the PSA of a member is the time it takes to fail when subjected to the standard fire such as ISO 834. Full-scale testing is the most common method of obtaining fire resistance ratings (Buchanan 2001).

As per the code, the PSA shall be determined using any one of the following methods.

1. By calculation (i) by determining the limiting temperature of the steel (T_l) in accordance with Section 16.7.4 (ii) by determining the PSA as the time (t in minutes) from the start of the test to the time at which the limiting temperature is attained in accordance with Section 16.8.1 for unprotected members and with Section 16.8.2 for protected members.
2. By direct application of a single test in accordance with Section 16.8.3.
3. By calculating the temperature of the member by using a rational method of analysis confirmed by test data or by structural analysis available in specialist literature, using mechanical properties which vary with temperature in accordance with Section 16.7.

These rules are based on the member being strong enough to support its load if the temperature of the steel does not exceed a limiting temperature. This is a comparison in the time domain, where the element is considered to be structurally sound until a time when calculations estimate that the member will fail.

The calculation of PSA involves the following:

(a) Calculation of the strength of the element as a function of the temperature of the element and determination of the limiting temperature.
(b) Calculation of the thermal response of the element, i.e., variation of the temperature of the element or parts of the element with time, when exposed to fire.

(c) Determination of the PSA at which the temperature of the element or parts of the element reach the limiting temperature.

16.4.3 Structural Requirements

The code states that for protected steel members and connections, the thickness of protection material (h_i) shall be greater than or equal to that required to give a PSA greater than or equal to the required FRL. Similarly, for unprotected steel members and connections, the exposed surface area to mass ratio (k_{sm}) shall be less than or equal to that required to give a PSA equal to the required FRL.

16.5 Calculation Approach

The design procedure [once the decision whether to consider the exposure of the steel element to either a natural fire or the pseudo fire (e.g., ISO 834 fire) has been made] is essentially to calculate the temperature rise within the structural element and to assess whether the maximum temperature reached will collapse the structure. The procedure can be simplified if the structure is considered to have simple connections. (The situation for frames, which are designed as rigid, is more complex and is still under research.) The following assumptions are also made in the calculation procedure (Martin & Purkiss 1992).

1. The steel element is assumed to have a uniform mean temperature over its cross section and length. This assumption may not be correct, especially in situations in which thermal gradients exist (beam-supporting concrete slab or where column is partially protected). The protection provided to a flange or web is allowed in the calculation through the H_p/A ratio, which in any case will overestimate the mean temperature, resulting in a conservative fire performance.
2. The sections are checked for load ratio. Since the tests take into account the creep strain induced by fire (anisothermal creep tests in which the specimens are loaded at a constant load and heated to a given strain level to give a plot of resultant strain versus temperature), this assumption is validated.

The thermal properties of both the insulation and the steel are assumed constant with respect to temperature. This assumption produces conservative results (Martin & Purkiss 1992).

16.5.1 Thermal Properties of Steel

Thermal properties of steel also change with varying temperature. These properties include (1) *density* (ρ), which is the mass of the material per unit volume in kg/m^3; (2) *specific heat* c_p, which is the amount of heat required to heat a unit mass of material by 1°C with unit of J/kg K; (3) *thermal conductivity* λ, which represents the rate of heat transferred through a unit thickness of material per unit temperature difference with a unit of W/mK.

Equations are available in Eurocode 3 for specific heat, which has the largest deviation from a constant value. However, in simple calculation models, a value of 600 J/kg K is used for specific heat (Purkiss 1996). This value is quite accurate for most of the temperature ranges but does not allow for the endothermic nature of the crystal-structure phase change of steel at about 735°C. (The British Steel code recommends a constant value of 520 J/kg K.)

The density of steel is recommended by Purkiss (1996) to remain at a value of 7850 kg/m^3 for all temperatures normally experienced during a fire. For simple design calculations, the constant conservative value of 45 W/m K is recommended for the thermal conductivity of steel by Purkiss (1996), BS 5950 (1990), and Eurocode 3 (1995). In simple calculation models the relationship between thermal elongation and steel temperature may be considered to be a constant and is given by

$$\Delta l/l = 12 \times 10^{-6}(T - 20) \tag{16.6}$$

Where l is the length at 20°C and Δl is the temperature-induced expansion. In most simple fire engineering calculations, thermal expansion is neglected; but for steel members which support a concrete slab on the upper flange, the differential thermal expansion caused by shielding of the top flange and the 'heat-sink' function of the concrete slab causes a 'thermal bowing' towards the fire in the lower range of temperatures.

16.5.2 Section Factor H_p/A

An important parameter in determining the rise of the temperature of the steel section is the *section factor* H_p/A, which is defined as the ratio of the heated perimeter (H_p) to that of the cross-sectional area of the member (A). As seen in Fig. 16.7, low values of H_p/A indicate a squat section and low rate of heating. The value of the heated perimeter H_p will depend on the type of insulation (e.g., sprayed insulation, intumescent paint, or board insulation which boxes the section) and also on whether the member is heated on three or four sides. Typical H_p/A values are shown in Fig. 16.8. The same ratio can be achieved in a three-dimensional sense with the

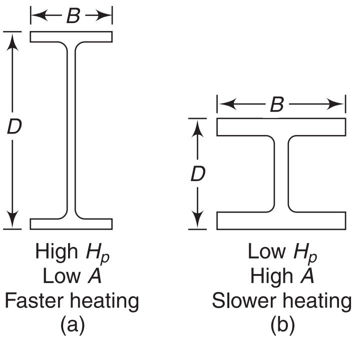
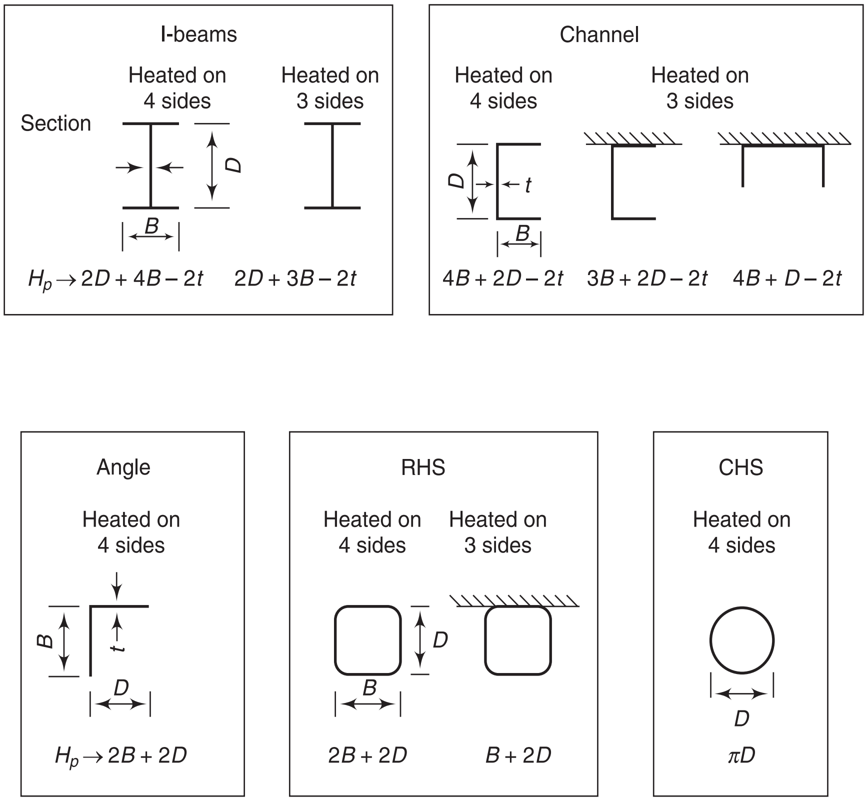
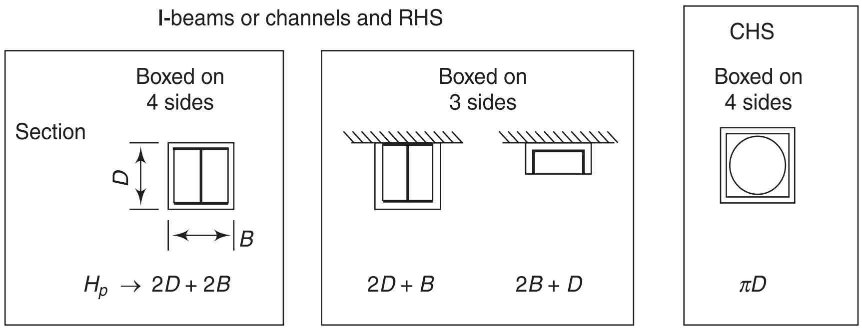


Fig. 16.7 H_p/A ratio



(a) Values of H_p for unprotected steelwork or steelwork protected by sprayed insulation or intumescent paint



(b) Values of H_p for boxed sections

Notes

1. A is always taken as the cross-sectional area of the steel member in calculating H_p/A.
2. The units of H_p/A are m^{-1} and the value is conveniently rounded to the nearest 5 m^{-1}.
3. Fillets and external radii have been ignored in calculating H_p.
4. Values for other configurations than those shown here may be calculated from analogous principles.

Fig. 16.8 Typical H_p/A values (Martin & Purkiss 1992) heated area per unit length to the volume per unit length is used instead, i.e., by using A_h/V, where A_h is the heated surface area and should not be confused with A, the cross-sectional area of the section. The ratio A_h/V gives the same information as H_p/A. Many publications give the H_p/A value for a range of beam sizes, but the calculation is not difficult and made using the width and depth of the beam, if the radii and fillets of the section are ignored as given in Fig. 16.8.

16.6 Calculation of Temperature Rise in Steel Members

This may be done either by considering the heat flow equations (as suggested in ECCS, 1983) or by fitting curves to the experimental data (as adopted in BS 5950-part 8, 1990). The heat transfer takes place in three phases: conduction, convection, and radiation. These three phases can occur separately or together depending on the circumstances.

16.6.1 Unprotected Members

For unprotected members, the temperature rise in the member, ΔT_s, due to a small increase in temperature Δt is given by

$$\Delta T_s = (1/C_s\, \rho_s)\, (H_p/A)\, (h_{\text{net}})\Delta t \tag{16.7}$$

where h_{net} is the heat flow per unit area (W/m^2) and is given by

$$h_{\text{net}} = \alpha_c(T_t - T_s) + \phi\varepsilon\sigma\,[(T_t + 273)^4 - (T_s + 273)^4] \tag{16.8}$$

where

α_c = coefficient of heat transfer by convection [normally taken as 25 W/m^2/K, if we consider the standard temperature–time curve (Lu & Maekelaeinen 2003)]

T_t = ambient gas temperature at time t

T_s = steel temperature at time t

ϕ = configuration factor, can be taken as 1. A lower value may be chosen to consider the position and shadow effect

ε = resultant emissivity of surface $\varepsilon = \varepsilon_m\, \varepsilon_f$ with $\varepsilon_m = 0.8$ and $\varepsilon_f = 1.0$. Therefore $\varepsilon = 0.8$. Note that the Eurocode recommends using a value of 0.56 for emissivity.

σ = Stefan–Boltzmann constant = 5.67×10^{-8} W/m^2 K^4

C_s = specific heat of steel taken as 600 J/kg K for simple calculations

ρ_s = density of steel = 7850 kg/m^3

H_p/A = section factor (m^{-1})

Solving the above incremental equation step by step gives the temperature development of the steel element during the fire. Normally, this is solved by using a spreadsheet as shown in Table 16.3.

Table 16.3 Spreadsheet calculation for the temperature of unprotected steel section

Time	Steel temperature T_s	Fire temperature T_t	Temperature change in steel ΔT_s
$t_1 = \Delta t$	Initial steel temperature T_0	Fire temperature halfway through time step (at $\Delta t/2$)	Calculated using Eqn (16.8) with T_s and T_t from this row
$t_2 = t_1 + \Delta_t$	$T_s + \Delta T_s$; T_s = temperature from the previous row	Fire temperature halfway through time step (at $t_1 + \Delta t/2$)	Calculated using Eqn (16.8) with T_s and T_t from this row

(*Source:* Buchanan 2001)

In order to ensure the numerical convergence of the solution, some upper limit is often prescribed for the time increment Δt. For example, ECCS suggests the upper limit as

$$\Delta t < 25{,}000/(H_p/A) \tag{16.9}$$

ECCS (1983) recommendations use the same formula for four-sided and three-sided exposure, but with an altered H_p/A value to allow for three-sided exposure to fire. The formula is

$$t = 0.54(T_s - 50)\,(A/H_p)^{0.6} \tag{16.10}$$

which can be rewritten as

$$T_s = 50 + t(H_p/A)^{0.6}/0.54 \tag{16.11}$$

ECCS limits this formula to the steel temperature ranging from 400°C to 600°C. The time when these temperatures may occur are limited by the time period between 10 and 80 min and the formula is valid for members with a section factor H_p/A value between 10 and 300 m^{-1}. It has been found that the upper temperature limit specified in the ECCS formula, Eqn (16.10), can be increased to 800°C without affecting the accuracy (Lewis 2000).

16.6.2 Protected Members

For members with passive protection, the basic mechanisms of heat transfer are identical to those of unprotected steelwork, but the covering material of very low conductivity reduces the heating rate of steel considerably. Also, the insulating layer itself has the capacity to store a small amount of heat. It is acceptable to assume that the exterior surface of the insulation is at the same temperature as the fire. The change in the temperature of protected steel over time can be written as (Martin & Purkiss 1992)

$$\Delta T_s = [(\lambda_i/d_i)/C_s\rho_s]\,(H_p/A)(T_t - T_s)\,\Delta t \tag{16.12}$$

For members where the insulation has a substantial heat capacity,

$$\Delta T_s = [(\lambda_i/d_i)/C_s\rho_s](H_p/A)\,(T_t - T_s)\,\Delta t/(1 + \xi) - \Delta T_t/(1 + 1/\xi) \tag{16.13}$$

where

$$\xi = C_i\rho_i' d_i\, H_p/(2C_s\,\rho_s\,A) \tag{16.14}$$

This additional modification is needed only when $\xi > 0.25$.

Here H_p/A is the section factor for the protected steel member (see Fig. 16.8), d_i is the thickness of insulation (in m), ρ_i' is the effective density of the insulation (in Kg/m^3), λ_i is the effective thermal conductivity of insulation (in W/m K), and C_i is the specific heat of the insulation material (in J/KgK).

A spreadsheet similar to that of the unprotected member can be set up to calculate the temperature of the steel section. The value of ρ_i' is given as

$$\rho_i' = \rho_i(1 + c) \qquad (16.15)$$

where c is the percentage moisture content of the protection material.

ECCS (1983) recommendations suggest the use of the following empirical formula for predicting the temperature of a steel member protected with light, dry insulation in time t (in min) as

$$t = 40(T_s - 140)\,[(d_i/\lambda_i)/(A/H_p)]^{0.77} \qquad (16.16)$$

where λ_i is the thermal conductivity of insulation (W/mK) and d_i is the thickness of insulation (m). This equation is valid in the range of t from 30 to 240 min, T_s from 400°C to 600°C, H_p/A from 10 to 300 m^{-1}, and d_i/λ_i from 0.1 to 0.3 m^2 K/W. Lewis (2000) shows that the range of applicability can be extended to 800°C.

For insulation containing moisture, the time delay t_v (in min) can be added to the time t calculated from Eqn (16.16), using

$$t_v = c\rho_i d_i^2/(5\lambda_i) \qquad (16.17)$$

where ρ_i is the density of the insulation (kg/m^3) and c is the moisture content of the insulation (%).

16.7 Mechanical Properties of Steel at Elevated Temperatures

When structural components are exposed to fire, they experience temperature gradients and stress gradients, which are both varied with time.

16.7.1 Stress–Strain Behaviour

The stress–strain behaviour of steel at elevated temperatures can be assessed using either of the following two methods (Nethercot 2001):

1. Using ISO thermal testing in which temperature is held constant and the applied strain (stress) is increased.
2. Using anisothermal testing in which the applied stress is held constant and the temperature is increased.

The second method is considered to represent the situation of building fires. The deformation of steel at elevated temperatures is described by assuming that the changes in strain consist of three components: mechanical or stress-related strain, thermal strain, and creep strain.

The stress–strain curves obtained by British Steel for Grade 275 steel for various temperatures using the second method are shown in Fig. 16.9. From this figure, it is seen that at temperatures in excess of about 550°C, steel loses about 50% of its strength at room temperature with partial load factors of 1.5 (adopted for LL + DL.).

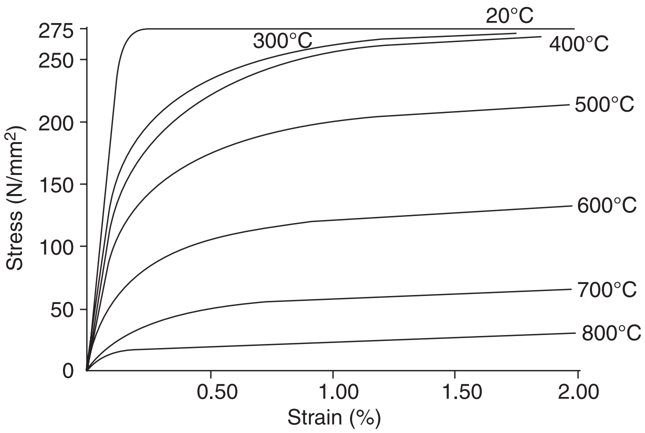


Fig. 16.9 Typical stress–strain curves at elevated temperature for steel with f_y = 275 MPa

This suggests that under conditions of uniform heating and full design load, failure of the members (in the form of excessive deflections) may occur at about this temperature. This drop in strength is equivalent to the erosion of safety provided by the load factor of 1.5. Hence a temperature of 550°C is referred to as the critical temperature for uniform heating. Note that in certain situations of thermal shielding (e.g., composite beam and slab, columns shielded by brick work) and at low load levels, the hottest part of steel may reach 800°C without the member collapsing. Although melting does not take place till about 1500°C, only 23% of the ambient strength remains at 700°C, 11% at 800°C, and 6% at 900°C.

It is also important to note from Fig. 16.9 that at elevated temperatures the stress–strain curves are more rounded with no well-defined yield stress. Hence the yield stress at elevated temperatures is taken at 0.2% of the strain level (often termed as 0.2% proof stress).

Since the slope of the initial stress–strain curves (E) also decreases with increasing temperature, the structures will deflect more at the same load level due to the temperature increase. Due to this the performance of the members at elevated temperatures will be much reduced.

This discussion shows that the mechanical properties such as the modulus of elasticity and yield stress vary with the temperature, generally decreasing with the increasing temperature. Thus, at a certain temperature, the strength of the member will decrease to virtually zero.

For the fire design of individual structural members such as simply supported beams that are free to expand during heating, the stress-related strain as discussed earlier is the only component that needs to be considered. If the reduction in strength with temperature is known, the member strength at elevated temperatures can easily by calculated using simple formulae. Note that the stress-related strains in fire-exposed structures may be well above yield levels, resulting in excessive plastification, especially in buildings with redundancy or restraint to thermal expansion.

Thermal strain *Thermal strain* is the thermal expansion ($\Delta L/L$) that occurs due to the increase in temperature. Thermal strain is not important for the fire design of simply supported members, but must be considered for frames and complex structural systems, since it can induce large internal forces (Buchanan 2001).

Creep strain The term *creep* is used to describe the long-term deformation of materials under sustained constant load. It is relatively insignificant in structural steel at normal temperatures. However, it becomes very significant at temperatures above 400–500°C and highly depends upon the stress level. At high temperatures, the creep deformation in steel can accelerate rapidly, leading to plastic behaviour. The effects of creep are usually allowed implicitly in fire engineering calculations by using stress-strain relationships, which include an allowance for the amount of creep that might be expected (Buchanan 2001).

16.7.2 Modulus of Elasticity

The modulus of elasticity is needed for buckling calculations and for elastic deflection calculations, but these are rarely attempted under fire conditions, since elevated temperatures lead rapidly to plastic deformations. The equations for the variation in the modulus of elasticity with respect to temperature adopted in IS 800 : 2007 are those recommended by the French Technical Center for Steel Construction (Proe et al. 1986). The advantage of using these equations is that they cover a large range of temperatures, from 0°C to 1000°C.

The variation in the modulus of elasticity with temperature is given by

$$E(T)/E(20) = 1.0 + T/[2000\ (\text{In}\ T/1100)] \text{ when } 0°\text{C} < T < 600°\text{C} \tag{16.18a}$$

$$= 690\ (1 - T/1000)/(T - 53.5) \text{ when } 600°\text{C} < T \leq 1100°\text{C} \tag{16.18b}$$

where $E(T)$ is the modulus of elasticity of steel at T°C and $E(20)$ is the modulus of elasticity of steel at 20°C, (room temperature).

It has to be noted that the Indian code, Australian code AS 4100 (1990) (Indian code provisions are based on the Australian code) and the New Zealand code NZS 3404 (1997) show that the modulus of elasticity reduces to zero at a temperature of 1000° C, which has been confirmed by recent studies by Poh (see Lewis 2000).

16.7.3 Yield Stress

The variation of yield stress with temperature is generally considered independent of the steel grade (Purkiss 1996). The formulae given in the Indian Code are based on a regression analysis of data from elevated temperature tensile tests conducted in Australia and Great Britain (Proe et al, 1986a; 1986b; Kirby & Preston 1988) and are given as

$$f_y(T)/f_y(20) = 1.0 \text{ when } 0 < T \leq 215°\text{C} \tag{16.19a}$$

$$= (905 - T)/690 \text{ when } 215 < T \leq 905°\text{C} \tag{16.19b}$$

where f_y(T) is the yield stress of steel at T°C, $f_y(20)$ is the yield stress of steel at 20°C (room temperature), and T is the temperature of steel (in °C).

Relationships of Eqns (16.18) and (16.19) are shown in Fig. 16.10. Note that the temperature at which the proportion of the yield stress at the elevated temperature is considered to have dropped to zero differs from that of the modulus of elasticity. To rectify this discrepancy and to give more reasonable results (as a zero yield stress cannot occur when the modulus of elasticity has a value), Lewis (2000) proposed the following relationship between yield strength and temperature:

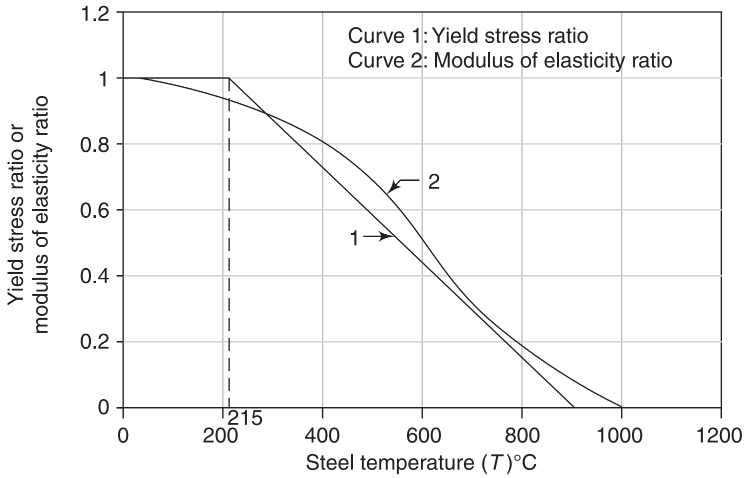


Fig. 16.10 Variation of mechanical properties of steel with temperature

$$f_y(T)/f_y(20) = 1.0 \quad 0 < T \leq 215°\text{C} \qquad (16.20a)$$
$$= (905 - T)/690 \quad 215 < T \leq 850°\text{C} \qquad (16.20b)$$
$$= 0.08\,(1000 - T)/150 \quad 850 < T \leq 1000°\text{C} \qquad (16.20c)$$

For special steel with a higher temperature resistance, such as TMT steels (see Section 16.10), the IS code suggests that the manufacturers' recommendation shall be used to obtain the variation of the yield strength and the modulus of elasticity with respect to the temperature.

16.7.4 Determination of Limiting Steel Temperature

The formula to determine the temperature at which the member being analysed will fail is a direct rearrangement of the formula for the variation of the yield stress of steel for temperature over 215°C [Eqn (16.19b)]. This implies that the only factor affecting the steel strength at the elevated temperature is the yield stress. Thus, as per the code,

$$T_l = 905 - 690 r_f \qquad (16.21)$$

where

r_f = ratio of the design action on the member under the design load for fire to the design capacity of the member at room temperature = R_f/R_d

R_d = design strength of the member at room temperature = R_u/γ_m

R_u = ultimate strength of the member at room temperature

γ_m = partial safety factor for strength

The design capacity of the member is based on the yield stress and the cross-sectional area of the beam, assuming that the cross section of the beam remains constant and uniform temperature is maintained throughout the steel. Thus Eqn (16.21) is valid for four-sided exposure to fire. Hence for three-sided exposure, which will result in significant temperature differences across the cross section of the member, a finite element approach that accounts for the temperature gradient in the steel is recommended (Lewis 2000).

The code suggests that for special steels the limiting temperature may be appropriately calculated using the thermal characteristics supplied by the manufacturers.

16.8 Time to Reach Limiting Temperature

Formulae to arrive at the relation between the time t and the limiting temperature T_l, have been derived by fitting the experimental fire tests.

16.8.1 Unprotected Steel

The time t at which the limiting temperature is attained can be calculated from the following formula given in the code.

a. For four-sided exposure conditions

$$t = -4.7 + 0.0263T_l + 0.213T_l/k_{sm} \tag{16.22}$$

b. For three-sided exposure conditions

$$t = -5.2 + 0.0221T_l + 0.433T_l/k_{sm} \tag{16.23}$$

where

t = time from start of the test (in min)

T_l = limiting steel temperature (in °C) 500°C $\leq T \leq$ 750°C

k_{sm} = exposed surface area to mass ratio (expressed in 10^3 mm^2/kg)

= 2×10^3 mm^2/kg $\leq k_{sm} \leq 35 \times 10^3$ mm^2/kg

= $(H_p/A) \times 10^6/7.85$ mm^2/kg

For temperatures below 500°C the code suggests a linear interpolation based on the time at 500°C and an initial temperature of 20°C at $t = 0$. These formulae were obtained from the regression analysis of British temperature data for unprotected steel (Wainman & Kirby 1988). Design temperatures for different fire resistance times for beams and columns are given in the British Steel Code BS 5950: Part 8: 1990, which were used as the basis for the regression analysis (Lewis 2000).

16.8.2 Protected Steel

The temperature rise in protected steel in the code is based on the results of tests on members with appropriate protection. To evaluate the performance of a protected member, temperature data can be obtained either from a single experimental test or from regression data from a series of tests.

For all members with four-sided exposure conditions, the limiting temperature is recommended by the code to be taken as the average temperature of all results taken by thermocouples taken during the tests. For columns with three-sided exposure conditions, the limiting temperature is taken as the average temperature of the thermocouples located on the face furthest from the wall, or, alternatively, the temperature from similar members with four-sided exposure can be used for more conservative results.

The code also states that the variation in steel temperature with the time measured in the standard fire test may be used without modification, provided

(a) the fire protection system is the same as the prototype,
(b) the fire exposure condition is the same as the prototype,
(c) the fire protection material thickness is equal to or greater than that of the prototype,
(d) the surface area to mass ratio is equal to or less than that of the prototype, and
(e) where the prototype has been submitted to a standard fire test in an unloaded condition, stickability has been separately demonstrated.

When the results of a series of tests are used, the variation in temperature with time can be interpolated provided the tests meet the limitations (a) – (e) mentioned above. The code also has an option to use a regression analysis to form a relationship between temperature and time by using the following formula:

$$t = k_0 + k_1 h_i + k_2 (h_i/k_{sm}) + k_3 T + k_4 h_i T + k_5 (h_i T/k_{sm}) + k_6 (T/k_{sm}) \quad (16.24)$$

where t is the time from the start of the test (in min), $k_0 - k_6$ are the regression coefficients as given in Table 16.4, h_i is the thickness of fire protection material (in mm), T is the steel temperature ($T > 250°C$), and k_{sm} is the exposed surface area to mass ratio, (10^3 mm^2/kg).

Table 16.4 Regression coefficients

k_0	k_1	k_2	k_3	k_4	k_5	k_6
–25.90	1.698	–13.71	0.0300	0.0005	0.5144	6.633

Limitations and conditions on the use of the regression analysis are also covered in clause 16.6.2.3 of the code.

Eurocode3 uses a spreadsheet time step formula for predicting the temperature of the protected steel (ECCS-1983; Martin & Purkiss 1992).

16.8.3 Determination of PSA from Single Test

The period of structural adequacy can be determined from the results of a single standard fire test provided conditions (a)–(d) in Section 16.8.2 are met as well as the following:

(e) The conditions of support are the same as the prototype and the restraints are not less favourable than those of the prototype.
(f) The ratio of the design load for fire to the design capacity of the member is less than or equal to that of the prototype.

These conditions mean that the results from the test can only be used when the prototype gives equal to, or more severe results than, those of the member being analysed, particularly in relation to the span, load restraints, support conditions, exposed surface area to mass ratio, and the thickness of the insulation.

The code also has a clause (clause 16.9) to provide for circumstances where there is three-sided exposure with concrete densities differing by more than 25% or for slabs with a thickness that varies by more than 25%. This clause makes an allowance for the resulting effect on the steel temperature which these variations make. The effect of different concrete densities and thicknesses is small, so a considerably large difference must be present in the construction before the members must be treated as separate cases.

16.8.4 Special Considerations

A conservative approach is given for designing the fire resistance of connections and web penetrations.

Connections The code requires that the connections be protected with the maximum thickness of fire protection material required for any of the members framing into the connections. This thickness should be maintained over the entire section of the connection, including bolt heads, welds, and splice plates.

Beam web penetrations The code states that unless determined in accordance with a rational fire engineering design, the thickness of fire protection material at and adjacent to web penetrations shall be the greatest of the following (see Fig. 16.11):

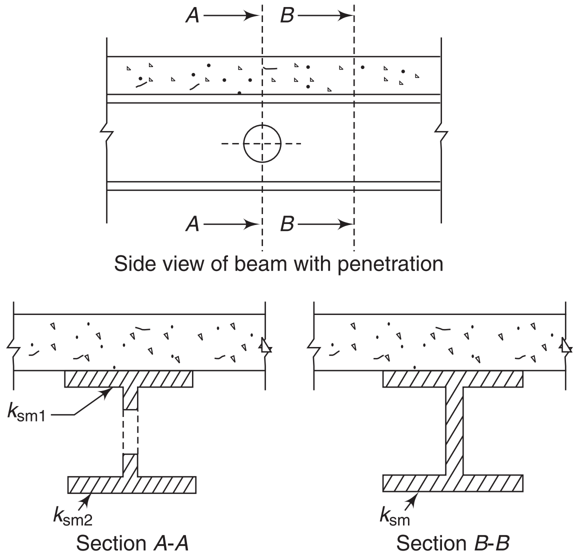


Fig. 16.11 Beams with web openings

(a) That required for the area of the beam above the penetration considered as a three-sided fire exposure condition
(b) That required for the area of the beam below the penetration considered as a four-sided exposure condition or
(c) That required for the section as a whole considered as a three-sided fire exposure condition

The thickness shall be applied over the full beam depth and shall extend on each side of the penetration for a distance at least equal to the beam depth and not less than 300 mm.

16.9 Passive Protection for Steelwork

Various passive protection methods have been tried in practice. These include cladding the members with insulating materials (using spray protection, board protection, intumescent coatings, concrete encasement, etc.), circulating water using a system of hollow structural members, composite construction (in which the concrete slab provides thermal shielding), and partial encasement using precast slabs (e.g., in slimfloor construction) or blockwork or double skin walls. These methods are discussed briefly in this section.

16.9.1 Fire Protection Systems

The traditional approach to the fire resistance of steel structures is to clad the members with insulating materials. The various alternative methods are chosen based on fire load, fire rating, and the type of structural members. These methods are discussed below briefly (Rains 1976):

Spray protection In this method of protection, sprays containing asbestos-free materials (e.g., vermiculate or mineral fibre) in a cement or gypsum binder are applied on the structural members as a coating for a prescribed thickness. They are relatively low cost and can be applied at the site rapidly, even to cover complicated shapes. They do not suffer from the problems faced in using rigid boarding around complex structural details. Mineral fibre fire proofing is generally sprayed with specially designed equipment. The equipment feeds the dry mixture of mineral fibres and various binding agents to a sprayer nozzle, where water is added to the mixture as it is sprayed onto the steel surface. This process is wet and messy and results in an undulating finish (see Fig. 16.12), which is unacceptable in public areas of buildings. The other important aspect of this insulation is the 'stickability' of the material. Hence the cohesion and

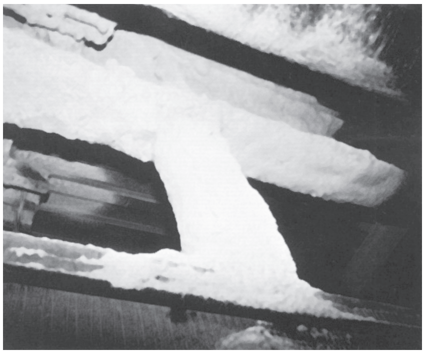


Fig. 16.12 Example of sprayed insulations

adhesion of the spray must be thoroughly tested. It is mostly used in areas that are normally hidden from public view, such as beams and connections above suspended ceilings. The required thickness to achieve fire-resistance ratings can be found in individual manufacturers' literature or trade publications. The thickness may vary from 10 to 100 mm with specific mass in the range of 240–400 kg/m^3. The coatings are difficult to repair and hence all attach-ments to the steelwork should be made prior to the installation of the fire protection coating. When exposed to fire, heat is absorbed in removing the water of hydration and absorbed water.

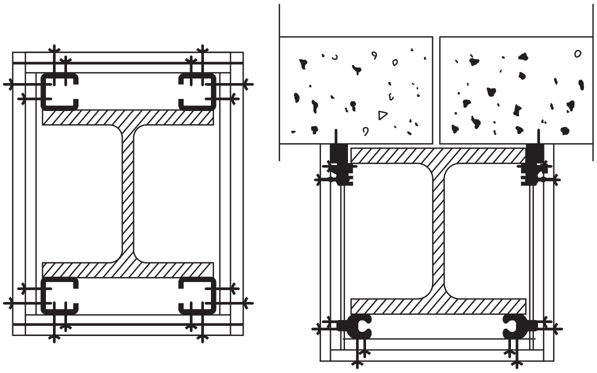
Sprayed fire protection offers the following benefits:

- It is relatively low cost.
- Its application is rapid.
- It can easily cover complex details.
- Some products offer corrosion protection.
- In most cases, blast cleaning and priming may not be necessary before applying the fire protection.

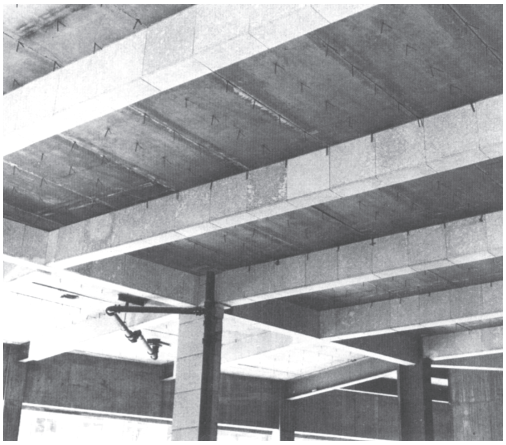
However, sprayed fire protection has the following limitations:

- The undulating finish may affect the appearance—especially the visible members.
- The process can create mess and dust while spraying and may require masking or shielding to limit overspray.
- The applied thickness may not be uniform and hence may not provide a fail-safe protection.
- It is not suitable for members exposed to atmospheric weather.

Board protection (Dry system) This kind of protection uses plasterboards or boards made of mineral fibre, calcium silicate, gypsum plaster, or vermiculate fixed around the exposed parts of the steel members. Calcium silicate boards are more expensive than gypsum boards. They are usually glued or screwed to metal or wood framings which are fastened to the steel member as shown in Fig. 16.13. The number and thickness of layers can be easily adapted to the particular application. These systems are easy to apply and effective. Also, the steel member does not require any preparation prior to applying the protection. The boards create an external profile which is aesthetically pleasing. But these boards are inflexible to use around complex details such as connections. A ceramic fibre blanket may be used as a more flexible insulating barrier in some cases. Some board products are soft or brittle and are susceptible to mechanical damage; some are prone to damage due to water and hence suitable for internal use only. This kind of protection is expensive than sprayed protection and is generally used on columns or exposed beams. Thermal properties of some insulation materials including some commercial products are given in Table 16.5. (Thermal properties of mild steel and air are also included for comparison.)



Schematic diagram
(a)



A building with beams protected with Vicuclad boards (work proceeding on columns)
(b)

Fig. 16.13 Fire insulation of columns and floor girders with two layers of 13-mm thick boards

Table 16.5 Thermal properties of insulation materials

Material	Density (kg/m^3)	Specific heat (J/kg K)	Thermal conductivity (W/m K)	Moisture content (% by weight)
Sprayed mineral fibre	250–350	1050	0.10–0.12	1.0
Fendolite by Firepro	775	1100	0.19	1.0
Vermiculite plaster	300–350	1200	0.12–0.15	15.0
Vicuclad© by Promat	400	1100	0.15	7.0
High density perlite or vermiculite plaster	550	1200	0.12	15.0
Mineral wool, fibre silicate (compressed fibre boards)	150	1200	0.20	2.0

(*contd*)

(*contd*)

Gypsum board	800	1700	0.20	20.0
Fibre silicate or fibre calcium silicate board	600	1200	0.15	3.0
Aerated concrete	600	1200	0.30	2.5
Lightweight concrete	1600	880	0.80	2.5
Dense concrete	2300	880	1.5	1.5
Polyurethane foam	20	1400	0.034	
Brick (common)	1600	840	0.69	
Mild steel	7850	460	45.8	
Air	1.1	1040	0.026	

(*Source:* Buchanan 2001; Lu & Laekelaeinen 2003)

The advantages of board fire protection are as follows,

- It is suitable for members which are visible. Also, a range of surface finishes and colours are available.
- The thickness of insulation is guaranteed.
- It offers clean and dry fixing and hence will not disturb other operations.
- No preparation of the steel surface is required prior to applying this kind of fire protection.

However, board fire protection has the following limitations.

- It is difficult to apply around complex details such as joints.
- This type of protection is not suitable for use either externally or in areas of high humidity.
- It is generally more expensive than spray protection.
- It may result in significant fixing time.

Intumescent coatings This type of fire protection is used in visible steelwork with moderate fire protection requirements. Intumescent paints provide decorative finish under normal conditions. However, under the action of fire they foam and swell, producing an insulating layer, which may be 50 times as thick as the original paint film. The intumescent coating (IC) system consists of three components: a primer, a base coat (the coat that foams and expands when exposed to fire), and a sealer coat. The base coat usually comprises the following (Chakraborty & Bandyopadhyay, 2005).

- A catalyst (e.g., ammonium polyphosphate), which decomposes to produce a mineral acid such as phosphoric acid.
- A carbohydrate (e.g., starch), which combines with the mineral acid to form a carbonaceous char.
- A binder or resin, which softens at a predetermined temperature.
- A spumific agent, which decomposes together with the melting of the binder to liberate large volumes of non-flammable gases. The production of these gases (e.g., carbon dioxide, ammonia, and water vapour) causes the binder to foam and expand to form an insulating char, many times the coating thickness.

The intumescent coatings are classified as thin-film and thick-film coatings. Of these, the thin-film coatings are commonly used (see Table 16.6).

Table 16.6 Typical application of intumescent coatings

Intumescent coating type	Thickness (in mm) for different fire resistance periods			
	30 min	60 min	90 min	120 min
Solvent-based thin coats	0.25–1.0	0.75–2.5	1.5	
Epoxy-resin-based thick coats	4.0–5.0	4.0–11.0	6.0–16.5	6.0–16.5

(*Source:* Chakraborty & Bandyopadhyay 2005)

The intumescent coatings may be applied by a brush, sprayer or roller in several layers till the required thickness is achieved. This system of fire protection requires blast cleaning of the surface of steelworks.

The advantages of the intumescent coating system of fire protection are the following.

- There is no appreciable increase in the overall dimensions of the member.
- IC permits easier modification to the structure; an important consideration to note is that one of the main attractions of steel structure is the relative ease with which it can be altered at a later date.
- Coating can be applied rapidly and easily to cover any complex details.
- Thicker and expensive coatings offer up to 120 min of fire resistance and are suitable for external use.

However, the use of intumescent coating has the following limitations.

- It is expensive than board or spray protection.
- Some systems are not suitable for use in either areas of high humidity or areas exposed to atmospheric conditions.
- It requires blast cleaning of the surface and a prime coat.
- Some coatings may also require a top sealer coat.

Concrete encasement Full or partial encasement of open steel sections in concrete is occasionally used as a method of fire protection [see Fig. 1.46(a) and (c)]. This method is useful when the composite action of steel and concrete (either plain or reinforced) is taken into account in the design. Thus, in addition to a higher load resistance, it can also provide fire resistance. In the case of hollow steel sections, concrete may be used to fill the sections, again with or without re-bars. During fire, the concrete acts to some extent as a heat sink and diminishes the heating of the steel section. The steel tube, even though loses its strength as its temperature rises, confines the concrete and prevents it from spalling. Such concrete-filled steel tubular columns with some reinforcement bars or fibre reinforcement enable resistance periods of up to 2 hr to be achieved (Giddings 1978; British Steel 1996; Twilt et al. 1996). However, it is essential to provide vent holes to prevent the excessive steam pressure from exploding the hollow member during heating.

Encasing steel members in concrete has the following advantages.

- It provides a robust system which can achieve a high fire resistance.
- When encased in high density concrete, no corrosion treatment of the steel is necessary.
- Visible flanges of columns allow the steel to be expressed architecturally.

- The composite action of steel and concrete can provide a higher load resistance in addition to higher fire resistance in basements and ground floors of multi-storey buildings.
- The system provides an enhanced resistance to seismic loading.

However, the use of concrete for fire resistance has the following limitations.

- The addition of concrete increases the dead weight and hence increases the foundation cost. The fully encased version requires shuttering with the concrete placed in situ (the partially encased shape can be cast off-site without shuttering).
- The on-site concreting operations also have an adverse effect on the scheduling of the work.
- Precasting of concrete results in the erection of heavy members and may delay the delivery to site.
- Connections must be designed to allow full compaction of concrete around the steel member.
- The concrete surrounding a steel member hides the steel surface. Hence the detection and rectification of any corrosion, due to the carbonation of concrete, becomes difficult.

Use of water Some buildings with hollow section columns are linked together as a system and filled with water fed from a gravity reservoir. The water absorbs the heat and, through natural circulation, carries the heat around the structure, thus allowing the heat to dissipate. Water losses due to boiling and evaporation are replenished from storage tanks. This will require imaginative detailing of the connections between individual elements. Additives may be necessary to prevent corrosion and to prevent freezing in cold climates (Bond 1975).

This system has the following advantages.

- Fire resistance is only limited by the volume of water stored.
- The structure is largely undamaged by fire.
- The use of hollow sections gives a pleasing slender appearance. The lack of external protection leaves the steel frame exposed so that it can be integrated into the building architecture, by allowing it to be finished in numerous attractive ways.

The use of water as a fire protection system has the following drawbacks.

- This method of protection is expensive from both construction and maintenance points of view.
- The system is generally economical only for large buildings.
- The system is applicable only if hollow structural sections are used for the members.
- The system is not applicable for members with an inclination less than 45°.
- In most cases, the architecture must include a huge water storage tank.

Most of the protection methods discussed so far are normally applied at site after the main structural elements are erected. Hence it results in considerable delay in the construction process and results in increased costs. Of course, the intumescent paints can be applied to steelworks at the fabrication stage, thus avoiding site work. However, care must be taken to see that the paints are not affected by abrasion or impact. It has to be noted that fire protection materials have to be routinely tested for insulation, integrity, and load-carrying capacity as per ISO 834 furnace tests. Material properties for the design are determined from the results by semi-empirical means (Lu & Maekelaeinen 2003). Design charts for insulation of steel members in fire have been provided by Wang (2005). Zicherman (2003) deals with the fire performance of foam–plastic building insulation.

16.9.2 Thermal Shielding by Partial Encasement

Most of the design codes are explicit about the fact that the structural fire resistance of a member is dependent to a large extent on its loading level in fire, and also that loading in the fire situation has a very high probability of being considerably less than the factored loads for which the strength design is performed (Lu & Maekelaeinen 2003). Designers can use this option in combination with other methods. Whichever method is used, fire protection adds to the construction and maintenance costs. Hence attention should be paid to the ways of utilizing the inherent fire resistance of several forms of construction to reduce the need for added protection.

The effect of loading level reduction in beams can be combined with a reduction in the exposed perimeter by making use of shielding and heat sink effects of the supported concrete slabs. The traditional down stand beam [Fig. 16.14(a)] gains some advantage over complete exposure by having its top flange upper face totally shielded by the slab. Supporting the slab on shelf angles welded to the beam webs [Fig. 16.14(b)] keeps the upper part of the beam web and the whole top flange protected from fire. Resistance periods of about 60 min may readily be achieved

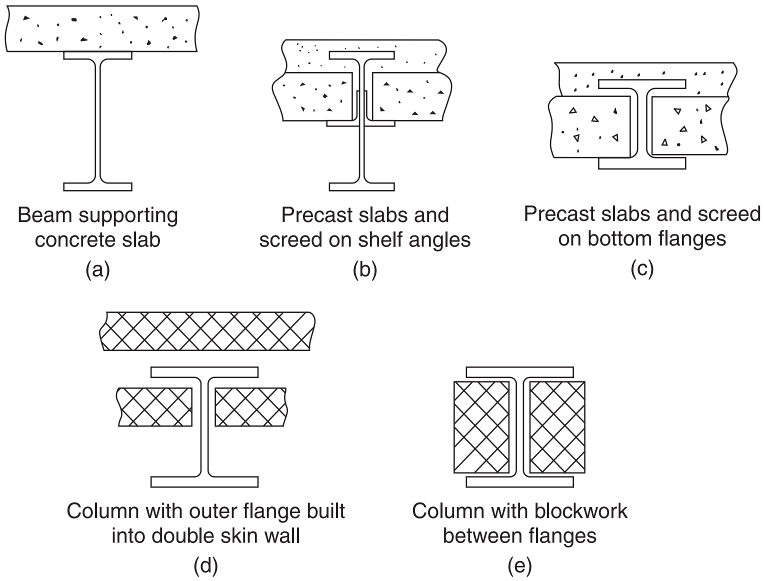


Fig. 16.14 Thermal shielding of beams and columns (Nethercot 2001)

by this method (Steel Construction Institute 1993). In 'slimfloor' construction, usually a shallow beam section is used and the slab is supported on the lower flange [Fig. 16.14(c)], either by pre-welding a plate across this flange or by using an asymmetric steel section (Mullett 1991; Lawson et al 1999). Such construction leaves only the lower face of the bottom flange exposed to fire. Though such a shallow beam will not represent an efficient structural arrangement (as a composite beam in the normal temperature condition), due to the high degree of thermal shielding offered by it, it offers a substantial resistance period in the case of fires. Due to the shallow beam sizes, the overall height of individual floors is reduced and hence results in cost savings on the completed structure.

Similarly, the outer flanges of H-section columns can be built into double skin wall [Fig. 16.14(d)] or the spaces between the flanges can be filled with concrete blocks as shown in Fig. 16.14(e). It has been shown that by such an arrangement a fire resistance of about 30 min can be easily achieved (BRE Digest 317 1986; Steel Construction Institute 1992).

The minimum thickness of material [gypsum plaster, plasterboard, asbestos insulating boards (which are banned in several countries), solid bricks of clay, aerated concrete blocks, concrete, etc.] required for encased steel columns of size 203 mm × 203 mm and steel beams of size 406 mm × 176 mm for 30 min to 4 hr of fire resistance is provided by IS 800 : 2007 (see Tables 16.7 and 16.8).

Table 16.7 Fire resistance of encased steel columns 203 mm × 203 mm (protection applied on all four sides)

Nature of construction and materials	Minimum dimensions (in mm) excluding any finish for a fire resistance period of				
	1 hr	1.5 hr	2 hr	3 hr	4 hr
Hollow protection (without an air cavity over the flanges)					
a. Metal lathing with trowelled lightweight aggregate gypsum plaster[1]	13	15	20	32	
b. Plasterboard with 1.6 mm wire binding at 100 mm pitch, finished with lightweight aggregate gypsum plaster less than the thickness specified:					
(i) 9.5-mm thick plaster board	10	15			
(ii) 19-mm thick plaster board	10	13	20		
c. Asbestos insulating boards					
(i) Single layer of board, with 6 mm cover fillets at transverse joints		19	25		
(ii) Two layers of board				38	50
d. Solid bricks of clay composition, or sand lime, reinforced in every horizontal joint unplastered	50	50	50	75	100
e. Aerated concrete blocks	60	60	60		

(*contd*)

(*contd*)

f. Solid blocks of lightweight concrete hollow protection, with an air cavity over the flanges	50	50	50	60	75
Asbestos insulating board screwed to 25 mm asbestos battens					
	12	19			
Solid protections					
a. Concrete, not leaner than 1:2:4 mix (unplastered)					
(i) Concrete not assumed to be load bearing, reinforced[2]	25	25	25	50	75
(ii) Concrete assumed to be load bearing	50	50	50	75	75
b. Lightweight concrete, not leaner than 1:2:4 mix (unplastered), concrete not assumed to be load bearing, reinforced	25	25	25	25	25

[1]So fixed or designed as to allow full penetration for mechanical bond.

[2]Reinforcement shall consist of steel binding wire not less than 2.3 mm in thickness, or a steel mesh weighing not less than 0.5 kg/m^2. In concrete protection, the spacing of the reinforcement shall not exceed 200 mm in any direction.

Table 16.8 Fire resistance of encased steel beams 406 mm × 176 mm (protection applied on three sides)

Nature of construction and materials	Minimum thickness (in mm) of protection for a fire resistance period of					
	0.5 hr	1 hr	1.5 hr	2 hr	3 hr	4 hr
Hollow protection (without an air cavity beneath the lower flanges):						
a. Metal lathing with trowelled lightweight aggregate gypsum plaster[1]	13	13	15	20	25	
b. Plasterboard with 1.6 mm wire binding[2] at 100 mm pitch, finished with lightweight aggregate gypsum plaster less than the thickness specified:						
(i) 9.5 mm plasterboard	10	10	15			
(ii) 19 mm plasterboard	10	10	13	20		
c. Asbestos insulating boards						
(i) Single layer thick board, with 6 mm cover fillets at transverse joints			19	25		
(ii) Two layers thick board					38	50
Hollow protection (with an air cavity below the lower flange):						
a. Asbestos insulating board screwed to 25 mm asbestos battens	9	12				
Solid protections						
a. Concrete, not leaner than 1:2:4 mix (unplastered):						
(i) Concrete not assumed to be load bearing, reinforced[3]	25	25	25	25	50	75
(ii) Concrete assumed to be load bearing	50	50	50	50	75	75

(*contd*)

(*contd*)

b. Lightweight concrete[4], not leaner than 1:2:4 mix (unplastered)	25	25	25	25	40	60

[1]So fixed or designed as to allow full penetration for mechanical bond.

[2]Where wire binding cannot be used, expert advice should be sought regarding alternative methods of support to enable the lower edges of the plasterboard to be fixed together and to the lower flange, and for the top edge of the plasterboard to be held in position.

[3]Reinforcement shall consist of steel binding wire not less than 2.3 mm in thickness, or a steel mesh weighing not less than 0.5 kg/m^2. In concrete protection, the spacing of the reinforcement shall not exceed 200 mm in any direction.

[4]Concrete not assumed to be load bearing, reinforced.

16.9.3 Use of Composite Slabs

One of the significant contributions to the rapid growth in the use of steel for multi-storey constructions has been the utilization of the floor system as shown in Fig. 16.15. This system uses profiled steel sheets of about 50 mm depth and 0.90 mm thickness with profiles as shown in Fig. 16.16 which span between beams and act as both permanent formwork and tension reinforcement for the slab (Lawson 1989). Two forms of composite action are employed: between the slab and the sheeting to span transversely and between the slab and the steel section to span longitudinally. *Shear studs* are site welded through the sheets to facilitate composite action of the slab and steel beam. Such metal deck composite floors provide enhanced fire resistance without the addition of fire protection to steel beams. Two methods are available for the fire protection of such systems. In the first method, if the required fire resistance period is longer than 30 min, sufficient tension reinforcement bars are provided in the slab (the slab is assumed to behave like a normal

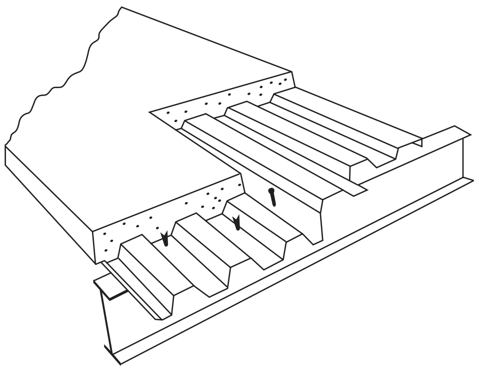


Fig. 16.15 Composite beam with profiled sheeting

Re-entrant (a) Trapezoidal (b)

Fig. 16.16 Various forms of profiled sheeting reinforced concrete slab without profile decking in the fire condition). Lawson (1985) showed that for fire resistance periods up to 90 min no additional tension reinforcement may be necessary. Alternatively, allowances may be made for the redistribution of moments by calculating plastic moment capacities at elevated temperatures (Nethercot 2001). If the slab is sufficiently thick to limit the temperature rise on the unexposed surface to an average value of 140°C, fire resistance of up to 4.0 hr can be obtained, as shown in Table 16.9. More details about the fire engineering design of composite slabs and shelf angle floors have been given by Martin and Purkiss (1992) and Der Stahlbau (1983).

Table 16.9 Insulation requirements for profile decking

Fire resistance (hr)	Minimum concrete dimension (in mm)			
	Trapezoidal decks		Re-entrant decks	
	NW	LW	NW	LW
0.5	60	50	90	90
1.0	70	60	90	90
1.5	80	70	110	105
2.0	100	80	125	115
3.0	130	110	150	135
4.0	150	130	170	150

(*Source:* Martin & Purkiss 1992)
Note: NW = Normal-weight concrete and LW = lightweight concrete.

16.9.4 Determination of Insulation Thickness

The insulation thickness t may be derived from the time to failure for an insulated member heated in a furnace test and is given by (see Melinek & Thomas 1987; Melinek 1989) for the discussion and background of the equation for time of failure)

$$t = \lambda_i I_f F_w (H_p/A) \text{ (in metres)} \tag{16.25}$$

where I_f is the insulation factor given by

$$I_f = \{t_f/[40\,(T_l - 140)]\}^{1.3} \tag{16.26}$$

where t_f is the required fire rating (in min) and T_l is limiting temperature (in °C). In Eqn (16.25), F_w is the protection material density factor and is given by

$$F_w = [(1 + 4\mu)^{0.5} - 1]/(2\mu) \tag{16.27}$$

where

$$\mu = \lambda_i\,(\rho_i'/\rho_s)\,I_f\,(H_p/A)^2 \tag{16.28}$$

where ρ_i is the dry density of insulation material (in kg/m^3), ρ_s is the density of steel = 7850 kg/m^3, λ_i is the effective thermal conductivity of the insulation (in W/mK), (H_p/A) is the section factor, and ρ_i' is the effective density of insulation material (taking into account the moisture content) c (in % by weight) and is equal to

$$\rho_i' = \rho_i(1 + c)$$

16.10 Fire-resistant Steels

Fire-resistant steels (FRS) in the form of *thermo-mechanically treated* (TMT) steels, which perform better than ordinary structural steel under fire, have been developed recently. Though these steels also have the same ferrite-pearlite microstructure, the addition of molybdenum and chromium results in a stable microstructure, up to a temperature of 600°C. The chemical composition of TMT steels is presented in Table 16.10.

Table 16.10 Comparison of the chemical composition of fire-resistant steel and mild steel

	C	Mn	Si	S	P	Mo + Cr
FRS	≤ 0.20%	≤ 1.50%	≤ 0.50%	≤ 0.040%	≤ 0.040%	≤ 1.00%
Mild steel	≤ 0.23%	≤ 1.50%	≤ 0.40%	≤ 0.050%	≤ 0.050%	

At about 600°C, the TMT steel has a minimum of 65% of its yield strength at room temperature, whereas ordinary structural steel has only about 40% of the yield strength. TMT joists, channels, and angles are readily available in the market and can be welded without any pre-heating. However, these TMT steels are very expensive and are used only in situations where a significant temperature rise is expected.

16.11 Fire Performance Assessment

As discussed already, fire resistance is a measure of the ability of building elements to resist a fire. Fire resistance is most often quantified as the time for which the element can meet certain criteria during an exposure to a standard fire test (see Sections 16.4.2 and 16.8). Structural fire resistance can also be quantified using the load capacity of a structural element exposed to fire. Thus, the verification of fire resistance can be in one of the three domains as shown in Table 16.11.

Table 16.11 Three domains for comparing fire severity with fire resistance

Domain	Units	Fire resistance	≥ Fire severity
Time	Minutes or hours	Time or failure	≥ Fire duration as calculated or specified by code
Temperature	Degree Celsius	Temperature to cause failure	≥ Maximum temperature reached during the fire
Strength	kN or kNm	Load capacity at elevated temperatures	≥ Applied load during the fire

(*Source:* Buchanan 2001)

This verification requires that the following design equation be satisfied:

$$\text{Fire resistance} \geq \text{fire severity} \tag{16.29}$$

where *fire resistance* is the measure of the ability of the structure to resist collapse or other failure during exposure to fire of specific severity and *fire severity* is a measure of the destructive impact of a fire, or a measure of the forces or temperature which can cause collapse or other failure as a result of the fire.

By far the most common procedure is for the fire severity and the fire resistance to be compared in the time domain such that

$$t_{\text{fail}} \geq t_s \tag{16.30}$$

where t_{fail} is the time to failure of the element and t_s is the fire duration specified in the code.

We have discussed the time and temperature domains in Sections 16.8 and Section 16.7.4, respectively. The methods to assess the strength are described in Buchanan (2001) and Lawson and Newman (1990).

While doing the verification of the fire resistance in the strength domain, the limit state to be considered is that of accidental loading which gives rise to a series of reduced load factors compared to those at the normal ultimate limit state (see Table 2.7 in Chapter 2). The analysis of the structure and the determination of the loading on elements in the structure for the purpose of assessing their fire resistance should be performed using the same assumptions and methods that would be used under ambient conditions except for the reduced load factors (and reduced values of the modulus of elasticity and the yield strength of steel) and the fact that snow loads are not usually considered.

The structural design at the normal temperature concerns the prevention of collapse (strength limit state) and excessive deformation (serviceability limit state). The design for fire resistance is mainly concerned with the prevention of collapse only. Large deformations are expected under severe fire conditions and, hence, are not normally calculated. Lu and Maekelaeinen (2003) give the design of a four-storey frame subjected to fire, based on Eurocode 3 provisions. The behaviour of multi-storey buildings subjected to fire is discussed by O'Corner and Martin (1998). Examples 16.3 and 16.4 give the design of unprotected and protected beams to give the required fire resistance.

Many software packages are available to estimate the temperature of steel members when exposed to elevated temperatures (CSIRO 1993; Franssen et al 2000). Some of these packages have been specifically produced for fire analysis (e.g., SAFIR, VALCAN Firecalc) while others are structural programs (NASTRAN, ANSYS, and ABAQUS) with temperature functions or options included into it. A comparison of the guidelines of fire-resistant codes of Europe, China, Japan, England, Canada, USA, Australia, and India is provided by Chakraborty and Bandyopadhyay (2005). The design of portal frames subjected to fire is discussed in Martin and Purkiss (1992) and Newman (1990).

Examples

Example 16.1 *Calculate the temperature rise on an ISMB 400 heated on four sides after exposure for 15 min to ISO 834 fire.*

Solution

For ISMB 400,

$$\text{Area} = 78.46 \text{ cm}^2$$

$$H_p = 2D + 4B - 2t = 2 \times 400 + 4 \times 140 - 2 \times 8.9 = 1342.2 \text{ mm}$$
$$= 1.342 \text{ m}$$

$$H_p/A = 1.342 \times 100^2/78.46 = 171 \text{ m}^{-1}$$

$$\Delta t = 25{,}000/(H_p/A) = 25{,}000/171 = 146$$

Use $\Delta t = 120$ s.

The governing equations are Eqns (16.7) and (16.8):

$$\Delta T_s = (1/C_s\rho_s)\,(H_p/A)\,h_{\text{net}}\Delta t$$

with

$$h_{\text{net}} = \alpha_c\,(T_t - T_s) + \phi\varepsilon\sigma\,[(T_t + 273)^4 - (T_s + 273)^4]$$
$$= 25(T_t - T_s) + 1 \times 0.8 \times 5.67 \times 10^{-8}\,[(T_t + 273)^4 - (T_s + 273)^4]$$
$$= 25(T_t - T_s) + 4.536 \times 10^{-8}[(T_t + 273)^4 - (T_s + 273)^4]$$

and

$$\Delta T_s = 1/(600 \times 7850) \times 171 \times h_{\text{net}} \times 120 = 4.36 \times 10^{-3}\,h_{\text{net}}$$

At $t = 0$, both T_o and T_s are 20°C. T_t is given by

$$T_t = T_o + 345 \log (8t + 1)$$

The values of T_s are calculated at $t = 0, 2, 4, 6, 8, \ldots$ minutes and the values of T_t, h_{net} $(T_t - T_s)$, and ΔT_s at $t = 1, 3, 5, 7, \ldots$ min. The calculations are carried out in the spreadsheet form as shown in Table 16.12.

Table 16.12 Temperature rise in an unprotected steel section

t (min)	T_t (°C)	h	ΔT_s (°C)	T_s (°C)
0				20
	349.2	14,693	64	
2				84
	502.3	26,110	113.8	
4				197.8
	576.4	30,847	134.5	
6				332.3
	625.8	30,850	134.5	
8				466.8
	662.8	26,099	113.8	
10				580.6
	692.5	18,132	79.1	
12				659.6
	717.3	10,843	47.3	
14				706.9
	738.6	6473	28.2	
16				735.1

Taking the mean of values 14 and 16 min, the steel has reached a temperature of 721°C after 15 min.

Example 16.2 *Calculate the rise in temperature in an ISMB 400 with fire protection in the form of 12.5-mm-thick gypsum board encasing the column on all four sides.*

Solution

The properties of the gypsum board (from Table 16.5) are

$$\rho_i = 800 \text{ kg/m}^3$$
$$C_i = 1700 \text{ J/kgK}$$
$$\lambda_i = 0.2 \text{ W/mK}$$
$$c = 20\%$$

Thus the effective density of the insulation [Eqn (16.15)] is

$$\rho_i' = \rho_i(1 + c)$$
$$= 800(1 + 0.0125 \times 20)$$
$$= 1000 \text{ kg/m}^3$$

Let us determine whether the insulation has a substantial heat capacity, using Eqn (16.14),

$$\xi = C_i \rho_i' d_i H_p/(2C_s \rho_s A)$$
$$H_p = 2D + 2B = 2 \times 400 + 2 \times 140 = 1080 \text{ mm}$$
$$H_p/A = 1.08 \times 100^2/78.46 = 137.6 \text{ m}^{-1}$$
$$\therefore \quad \xi = [(1700 \times 1000 \times 0.0125)/(2 \times 7850 \times 600)] \times 137.6$$
$$= 0.31$$

Since this value is greater than the limiting value of 0.25, the insulation has a substantial heat capacity. Hence, as per Eqn (16.13),

$$\Delta T_s = [(\lambda_i/d_i)/C_s\rho_s)\,(H_p/A)\,(T_t - T_s)\,\Delta_t/(1 + \xi) - \Delta T_t/(1 + 1/\xi)$$

The allowable value of Δ_t from Eqn (16. 9),

$$\Delta_t < 25{,}000/137.6 = 181.6 \text{ s}$$

Use a time step of 180 s.

Substituting these values,

$$\Delta T_s = [(0.2/0.0125)/(7850 \times 600)] \times 137.6 \times (T_t - T_s) \times 180/(1 + 0.31)$$
$$-\Delta T_t/(1 + 1/0.31)$$
$$= 0.064\,(T_t - T_s) - 0.237\Delta T_t$$

As in Example 16.1, the reference temperature is taken as 20°C and the value of T_t is calculated using Eqn (16. 1) as

$$T_t = T_o + 345 \log(8t + 1)$$

The calculations are done again in the form of a spreadsheet as shown in Table 16.13.

Table 16.13 Temperature rise in a protected steel section

T (min)	T_t (°C)	ΔT_t (°C)	$T_t - T_s$ (°C)	ΔT_s (°C)	T_s (°C)
0					20
	404.3	384.3	384.3	*	
3					20
	561.0	156.7	541.0	*	

(*contd*)

(*contd*)

6					20
	635.9	74.9	615.9	21.7	
9					41.7
	685.6	49.7	644.0	29.4	
12					71.1
	722.9	37.3	651.8	32.9	
15					104
	This calculation is continued and skipped to the final lines of the table.				
48					460.5
	916.6	9.3	456.1	27.0	
51					487.5
	925.4	8.8	437.9	25.9	
54					513.4
	933.7	8.3	420.3	25.0	
57					538.4
	941.6	7.9	403.2	23.9	
60					562.3

Note that in the very early stage, ΔT_s is found to be negative, as the heat flux is absorbed by the insulation. Hence, negative values have been set equal to zero and are shown in Table 16.13 as '*'. These results have been plotted along with the results of Table 16.13 in Fig. 16.17. It is seen that with only a 12.5-mm-thick gypsum board, the temperature of the steel section drops dramatically.

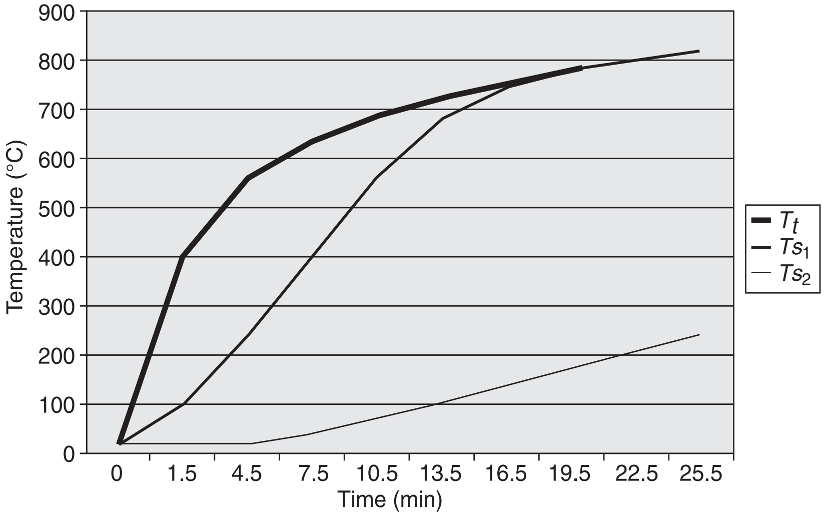


Fig. 16.17 Temperature–time curves of standard fire, protected and un-protected steel sections

Example 16.3 *An ISMB 500 supports a concrete floor. If 30 min fire resistance is required, what fraction of its capacity can the beam safely carry? Assume the live load to dead load ratio of 0.67:0.33.*

Solution

The load factor at room temperature is 1.5 (Table 5.1 of code). Assume the load factor for live load to be 0.8 and dead load to be 1.0 for the fire condition.

Combined load factor for fire limit state = $1.0 \times 0.33 + 0.8 \times 0.67 = 0.866$

Therefore, if fully loaded,

$$\text{load ratio} = 0.866/1.5 = 0.577$$

For a three-side heated beam member,

$$H_p = 2D + 3B - 2t$$
$$= 2 \times 0.5 + 3 \times 0.19 - 2 \times 0.0112 = 1.548 \text{ m}$$
$$H_p/A = 1.548 \times 100^2/132.11 = 117 \text{ m}^{-1}$$
$$k_{sm} = 117 \times 10^6/(7.85 \times 1000) = 14.9 \times 10^3 \text{ mm}^2/\text{kg} < 35 \times 10^3$$

Hence, from Eqn (16.23),

$$t = -5.2 + 0.0221T_l + 0.433T_l/k_{sm}$$
$$30 = -5.2 + 0.0221T_l + 0.433T_l/14.9$$
$$T_l = 688$$

From Eqn (16.21), the load ratio corresponding to the limiting temperature of 688°C is

$$r_f = (905 - T_s)/690$$
$$= (905 - 688)/690 = 0.31$$

∴ Proportion of full capacity available = 0.31/0.577 = 0.54

Example 16.4 *Repeat Example 16.3 to determine what thickness of insulation will be required if the beam has been designed to carry 85% of its moment capacity when subjected to fire. If the fire rating is to be increased to 120 min, by how much must the insulation thickness be increased? Assume an insulation of sprayed mineral fibre with density ρ_i = 350 kg/m^3, moisture content c = 1%, k_i = 0.10 W/mK, and density of steel ρ_s = 7850 kg/m^3.*

Solution

$$\text{Fire limit state load ratio} = 0.85 \times 0.866 = 0.736$$
$$\text{Limiting temperature } T_l = 905 - 690r_f$$
$$= 905 - 690 \times 0.736$$
$$= 397°\text{C}$$

For a limiting temperature of 397°C and 30 min fire resistance, as per Eqns (16.26)–(16.28),

$$I_f = \{t_f/[40\,(T_l - 140)]\}^{1.3}$$
$$= \{20/[40(397 - 140)]\}^{1.3}$$
$$= 299 \times 10^{-6} \text{ m}^3/\text{kW}$$
$$\mu = k_i\,[\rho_i(1 + c)/\rho_s]\, I_f\,(H_p/A)^2$$
$$= 0.10 \times [350\,(1 + 0.01)/7850] \times 299 \times 10^{-6}\,(117)^2$$
$$= 0.018$$
$$F_w = [(1 + 4\mu)^{0.5} - 1]/(2\mu)$$
$$= [(1 + 4 \times 0.018)^{0.5} - 1]/(2 \times 0.018)$$
$$= 0.983$$

∴ Required thickness of insulation, as per Eqn (16.25),

$$t = k_i\,(H_p/\mathrm{A})I_f F_w$$
$$= 0.10 \times 117 \times 299 \times 10^{-6} \times 0.983$$
$$= 0.0034 \text{ m}$$
$$= 3.4 \text{ mm}$$

If the fire rating is increased to 120 min,

$$I_f = \{120/[40(397 - 140)]\}^{1.3}$$
$$= 3071 \times 10^{-6} \text{ m}^3/\text{kW}$$
$$\mu = 0.10 \times [350\,(1 + 0.01)/7850] \times 3071 \times 10^{-6} \times 117^2$$
$$= 0.189$$
$$F_w = [(1 + 4 \times 0.189)^{0.5} - 1]/(2 \times 0.189)$$
$$= 0.860$$

The required thickness of insulation

$$t = 0.10 \times 117 \times 3071 \times 10^{-6} \times 0.860$$
$$= 0.0309 \text{ m} = 31 \text{ mm}$$

Summary

As per the building regulations, buildings in India are to be designed in such a way that there is an acceptable level of performance in the event of a fire. Essentially, this is intended to ensure public safety rather than safeguard the structure. Thus the main criteria are to prevent premature collapse, thereby permitting escape from the building, and to limit the spread of fire, thus reducing the risk of the surrounding properties and their occupants.

The basic material properties of steel used in the structural design—its strength and stiffness—are adversely affected by an increase in temperature beyond 300°C. Hence it is important to provide necessary fire resistance to steel structures. Several studies have been conducted all over the world to study the fire-resistant design. Based on these studies, some basic information about the characteristics of fire and its growth, design curves, and models is given in this chapter. Since it is very difficult to model the natural fire (due to the large number of parameters that affect the fire growth), some standard nominal temperature curves, such as the one defined in ISO 834, have been proposed based on the furnace testing of components.

Fire protection can be in the form of active protection (e.g., fire alarm, fire extinguishers, smoke control, and emergency exits) and/or passive protection (structural fire protection, layout of escape routes, fire brigade access routes, and control of combustible materials of construction).

The basic idea of the fire engineering design of steel structures is to provide structural integrity during a fire for a specified period of time (as specified in the codes for different classes of buildings). The calculation procedure for determining the temperature rise within the structural element and to access whether the maximum temperature reached will effect the collapse of a structure is given for both protected and unprotected steel members. A discussion on the effect of fire on the mechanical properties of steel is also included.

The various passive fire protection systems (spray protection, board protection, intumescent coating, concrete encasement, and water-filled structures) are discussed. Thermal shielding by partial encasement and the use of composite slabs as means of fire protection are also explained. Recent developments such as fire-resistant steels are also included. The concepts and equations presented are explained with the use of worked-out examples.

Exercises

1. Calculate the temperature rise on an ISMB 300 column heated on three sides after exposure for 20 min to ISO 834 fire.
2. Calculate the temperature rise on an ISMC 300 column heated on three sides after exposure for 15 min to ISO 834 fire.
3. Calculate the rise in temperature in an ISMB 500 column with fire protection in the form of mineral wool (with $\rho_l = 150$ kg/m^3, $C_i = 1200$ J/kgK) provided on all four sides.
4. An ISMB 400 supports a concrete floor. If 45 min of fire resistance is required, what fraction of its capacity can the beam safely carry? Assume the live load to be 75% of the dead load.
5. Repeat Exercise 4 to determine what thickness of insulation will be required if the beam is designed to carry 80% of its moment capacity when subjected to fire. If the fire rating is increased to 90 min, by how much the insulation thickness needs to be increased?

Review Questions

1. What are the aims of traditional building codes with regard to fire safety?
2. What are the factors that influence the temperature, magnitude, and distribution of fire?
3. What are the three phases of fire growth within a compartment?
4. What is the main purpose of nominal temperature–time curves?
5. Does the nominal time–temperature curve represent any natural building fire?
6. What is the name of the standard temperature–time curve which has been accepted internationally?
7. What are the three failure criteria adopted for fire resistance testing?
8. Write the temperature–time relationship as per ISO 834 fire.
9. List the parameters that are required to represent the time–temperature relationship of natural fires.
10. Write down the equation describing the time–temperature relationship in the case of natural fire.

11. Describe what is meant by fire load.
12. List the main problems that are encountered in the fire engineering design of structures.
13. What are the active and passive protection methods that are employed in practice?
14. How is the fire resistance level specified in the code?
15. What is meant by the period of structural adequacy and what are the methods employed to determine it?
16. What are the properties of steel which change with varying temperature?
17. State the values of the following properties of steel, which are commonly assumed in simple calculation models:
    a. Specific heat of steel
    b. Density of steel
    c. Thermal conductivity of steel
18. Define section factor H_p/A.
19. How is the section factor expressed in a three-dimensional model?
20. What is the value of the Stefan–Boltzmann constant?
21. Write down the equation for temperature rise in a member due to a small increase in temperature for unprotected steel members.
22. Write down the equation for the change in temperature over time for protected steel.
23. Write down the empirical formula suggested by ECCS 1983 for predicting the temperature of a steel member when
    a. It is not protected.
    b. It is protected with light, dry insulation.
24. Write short notes on the effects of elevated temperature on the mechanical properties such as the modulus of elasticity and the yield stress of steel.
25. What is meant by creep? How is the effect of creep usually taken care of in fire engineering calculations?
26. Write down the expressions for the variation in the modulus of elasticity and yield stress with temperature as given in IS 800 : 2007.
27. What is the limiting steel temperature and how is it determined?
28. List the advantages and limitations of the following fire protection systems:
    a. Spray protection
    b. Board protection
    c. Instumescent coating system
    d. Concrete encasement
    e. Water-filled structures
29. List some of the active fire protection systems.
30. List some of the thermal shielding systems.
31. Write a short note on the use of composite slabs as a means of fire protection.
32. Write a short note on fire-resistant steels.
33. Write down the equation for determining the insulation thickness.
34. Write a short note on fire performance assessment.