

STEEL BUILDINGS IN EUROPE

Multi-Storey Steel Buildings

Part 6: Fire Engineering

Multi-Storey Steel Buildings

Part 6: Fire Engineering

FOREWORD

This publication is the sixth part of a design guide, *Multi-Storey Steel Buildings*.

The 10 parts in the *Multi-Storey Steel Buildings* guide are:

- Part 1: Architect's guide
- Part 2: Concept design
- Part 3: Actions
- Part 4: Detailed design
- Part 5: Joint design
- Part 6: Fire Engineering
- Part 7: Model construction specification
- Part 8: Design software – section capacity
- Part 9: Design software – simple connections
- Part 10: Software specification for composite beams.

Multi-Storey Steel Buildings is one of two design guides. The second design guide is *Single-Storey Steel Buildings*.

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Contents

	Page No
FOREWORD	iii
SUMMARY	vi
1 INTRODUCTION	1
2 FIRE SAFETY ENGINEERING	3
2.1 Definition of fire safety engineering	3
2.2 Objectives of fire safety	3
2.3 Approaches to structural fire engineering	5
3 FIRE PROTECTION SOLUTIONS	14
3.1 Active fire protection	14
3.2 Passive fire protection	15
3.3 Fire resisting construction	19
4 SIMPLE CALCULATION MODELS	26
4.1 Fire behaviour and thermal actions	26
4.2 Heat transfer	29
4.3 Structural Analysis	30
4.4 Simple structural fire design methods	31
5 TENSILE MEMBRANE ACTION	37
5.1 Cardington fire test	37
5.2 FRACOF fire tests	39
6 USE OF NATURAL FIRE EXPOSURE AND ADVANCED STRUCTURAL MODELLING	42
6.1 General	42
6.2 Modelling fire severity	42
6.3 Modelling heat transfer	43
6.4 Advanced structural models	43
6.5 Validation/verification of advanced models	44
6.6 Regulatory approval	44
REFERENCES	45
FURTHER READING	45
Worked Example: Fire safety strategies and design approach of steel floor beam	49

SUMMARY

This publication provides engineers with a wide range of strategies and design approaches for the fire safety design of multi-storey buildings. It contains background information and the design basis of fire safety engineering. Forms of construction covered in this publication include unprotected and protected steel members and composite members. In terms of fire safety strategies the reader will find guidance on active and passive fire protection as well as alternative structural solutions and structural fire design.

Three different fire design approaches are covered in this guide:

- Use of established datasheets.
- Simple calculation models to EN 1993-1-2 and EN 1994-1-2.
- Advanced calculation models.

Engineers may use pre-engineered datasheets to ensure the fire safety of multi-storey buildings. More economic fire design solutions may be achieved by using the simple calculation models given in EN 1993-1-2 and EN 1994-1-2 or by carrying out advanced analyses based on engineering fundamentals and finite element techniques. The latter approach is generally applied by specialist fire engineers.

1 INTRODUCTION

Fire safety is one of the most critical issues in the design of modern multi-storey buildings. The term *fire safety* describes the precautions to minimise the likelihood and effect of a fire that may result in injury, death and loss of property. Figure 1.1 shows examples of fires in multi-storey buildings.

The general objectives of fire design are to protect life, including building occupants and fire fighters, and to minimise business disruption, damage to building property, building contents and the surrounding environment.



Figure 1.1 Fires in multi-storey buildings

To achieve the above objectives, the Construction Products Directive 89/106/EEC^[1] states in *Annex I – Essential requirements*, that for Safety in case of fire:

“The construction works must be designed and built in such a way that in the event of an outbreak of fire:

- the load bearing capacity of the construction can be assumed for a specified period of time,
- the generation and spread of fire and smoke within the works are limited,
- the spread of fire to neighbouring construction works is limited,
- the occupants can leave the works or can be rescued by other means,
- the safety of rescue teams is taken into consideration.”

To meet the regulatory requirements, engineers have to work closely with architects, contractors, manufacturers and suppliers in the design of multi-storey buildings for the fire scenario. Although many issues are primarily covered in the architectural design, engineers need to be aware of fire safety with particular reference to structural fire engineering. In some circumstances, engineers may have to use a wide range of fire safety strategies and design

approaches to ensure that a multi-storey building meets all mandatory fire safety requirements.

The parts of the Eurocodes relevant to fire design of multi-storey steel-framed buildings with composite floors, include:

- EN 1991-1-2, Eurocode 1. Actions on structures exposed to fire^[2]
- EN 1993-1-2, Eurocode 3. Steel structures – Structural fire design^[3]
- EN 1994-1-2, Eurocode 4. Composite structures – Structural fire design^[4].

In addition to the general requirements, principles and rules, these documents also recommend the partial factors and design effect of loads for fire design. They provide a series of equations/models for the calculation of temperature rise, critical temperature and load bearing resistance of a structural member in fire conditions.

This guide provides engineers with a wide range of fire safety strategies for the design of multi-storey buildings. For less experienced designers this publication provides guidance on the use of datasheets to meet the legal fire safety requirements. More experienced engineers will find that the simple calculation models in the Eurocodes are straightforward to apply and will result in a more economic solution.

2 FIRE SAFETY ENGINEERING

This Section describes the basis of fire engineering design with particular reference to structural fire resistance and design approaches for multi-storey building structures to ensure adequate levels of life safety as required by national regulations.

2.1 Definition of fire safety engineering

Fire safety engineering is a multi-disciplinary science which applies scientific and engineering principles to protect people, property and the environment from fire. Structural fire engineering or design for fire resistance is a small part of fire safety engineering.

2.2 Objectives of fire safety

The objective of fire safety is to limit the risk of loss resulting from fire. Loss may be defined in terms of death or injury to building occupants or fire-fighters, financial loss due to damage of the building contents or disruption to business and environmental loss due to polluted fire fighting water or the release of hazardous substances into the atmosphere.

National regulations normally set minimum fire safety requirements to provide an adequate level of life safety, but other stakeholders such as the client, building insurer or the government's environmental protection agency may also require that the risks of financial or environmental losses are limited.

The level of safety required is specified in national regulations. Although these regulations vary between member states, they aim to fulfil the following core objectives, defined by the Construction Products Directive 89/106/EEC^[1]:

“The construction works must be designed and built in such a way that in the event of an outbreak of fire:

- The load bearing resistance of the construction can be assumed for a specified period of time
- The generation and spread of fire and smoke within the works are limited
- The spread of fire to neighbouring construction works is limited
- The occupants can leave the works or can be rescued by other means
- The safety of rescue teams is taken into consideration.”

While structural fire resistance cannot fulfil all of these core fire safety objectives, it is normally considered to be a key part of the fire safety strategy for a building.

2.2.1 Structural fire engineering

Structural fire engineering can be classed as a specific discipline within fire safety engineering, concerning the analysis of structural behaviour in fire conditions.

The basis of fire design for building structures is set out in EN 1990, which states that fire design should be based on a consideration of fire development, thermal response and mechanical behaviour. The required performance of the structure can be determined by global analysis, analysis of sub-assemblies or member analysis, as well as the use of tabular data derived from analysis or testing and individual fire resistance test results.

Considering the fire behaviour allows the designer to define the thermal actions to which the structural members will be exposed. In the prescriptive approach, the thermal action can be determined directly by use of a nominal time-temperature curve defined in EN 1991-1-2. For building structures, the standard time-temperature curve (the ISO curve) is normally used.

Having determined the thermal actions, the thermal response of the structure should be considered using an appropriate method of heat transfer analysis to determine the temperature-time history of the structure.

Finally, the mechanical behaviour of the member may be evaluated by analysis or testing to determine the resistance of the member given its temperature-time history.

Depending on the intended function of a structural element, the acceptability of its mechanical behaviour may be assessed against one or more of the following criteria, evaluated either on the basis of analysis or fire resistance test.

- *Load bearing resistance (R)* - the ability to resist specified actions without collapse during the required period of time in fire.
- *Insulation (I)* - the ability to restrict the temperature rise on the unexposed surface below the defined limits of 140°C (average) and 180°C (maximum) to prevent the ignition of a fire within the adjacent spaces.
- *Integrity (E)* - the ability to limit the development of significant sized holes through its thickness, to prevent the transfer of hot gas and spread of a fire into the adjacent spaces.

The national regulations denote each of these three categories by using the above reference letter followed by the time requirement. For example a requirement of 60 minutes load bearing resistance for a given member would be expressed as R60.

Note that load bearing resistance (*R*) is required for all load bearing structural elements. However, insulation (*I*) and integrity (*E*) are only required for separating elements, such as floor slabs and walls which form fire resisting compartment boundaries.

2.3 Approaches to structural fire engineering

Structural fire engineering design can be carried out using a prescriptive approach, where periods of fire resistance are defined by national regulations, or a performance-based approach, where the designer must quantify the level of risk and demonstrate that it is acceptable. It should be noted that the acceptance of a performance-based approach is dependant on national regulatory authority, who should be consulted at an early stage of the design process. Table 2.1 provides a summary of the tools available for use when adopting either of these approaches.

Table 2.1 Approaches for fire design

Approach	Tools	Fire loads (Thermal actions)	Fire effects (Member temperature)	Fire resistance (Member resistance)
Prescriptive approach (standard fire methods)	Pre-engineered datasheets	Standard ISO fire tests to: <ul style="list-style-type: none"> EN 1363-1, § 5 EN 1365-2, § 5 	Relevant information covered in: <ul style="list-style-type: none"> information packages provided by fire protection manufacturers documents published in Access-Steel EN 1994-1-2, § 4.2 	
	Simple rules & models	Standard ISO fire calculations to EN 1991-1-2, §3.2	Steel members to EN 1993-1-2 § 4.2.5 Composite members to EN 1994-1-2 Annex D2, § 4.3.4.2.2	§ 4.2.3 § 4.2.4 Annex E – F, § 4.3.1 § 4.3.4.2.4 § 4.3.4.2.3
	Advanced rules & models		Physical models for heat transfer Finite element analysis	Physical models for structural response Finite element analysis
Performance-based approach (natural fire methods)	Simple rules & models	Parametric fire Localised fire	Steel members to EN 1993-1-2 § 4.2.5	§ 4.2.3 § 4.2.4
	Advanced rules & models	Consider interaction between structural members and tensile membrane action. Natural fire to EN 1991-1-2, § 3.3 Annex A to F Localised, zones, CFD	Physical models for heat transfer Finite element analysis	Physical models for structural response Finite element analysis

2.3.1 Prescriptive Approach

The prescriptive approach is the traditional and still most commonly used method for fire engineering design. It aims solely to provide adequate levels of life safety to meet the fire resistance requirements specified in the national building regulations. Prescriptive regulations will contain requirements that aim to satisfy the core objectives for fire safety set out in Section 2.2.

Structural fire resistance

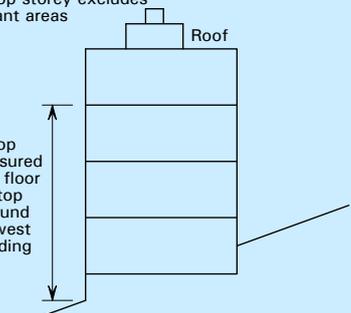
Prescriptive regulations require that buildings are subdivided by fire resisting construction. The requirements typically give limits for the maximum size of a single compartment and recommend fire resistance requirements for the

structural elements separating one compartment from another. Dividing a multi-storey building horizontally and vertically into a number of fire compartments will help to limit the spread of fire and smoke inside the building, giving occupants adequate time to escape. Some national regulations permit a relaxation in the limits of compartment size if the building is fitted with a sprinkler system.

Structural fire resistance requirements are normally defined in terms of time periods during which a structure or structural member will perform adequately when assessed against the load bearing, insulation and integrity criteria.

The requirements for fire resistance for multi-storey buildings are generally specified with regard to the use and height of the building, as shown in Table 2.2. Typically, the fire resistance requirements for multi-storey buildings range from 60 minutes (R60) to 120 minutes (R120), but some national regulations may require up to 4 hours fire resistance. If a building is fitted with a sprinkler system, the fire resistance period required for the structural elements by prescriptive regulations may be reduced.

Table 2.2 Typical fire resistance requirements

	Fire resistance (min) for height of top storey (m)				
	<5	≤18	≤30	>30	
Residential (non-domestic)	30	60	90	120	Height of top storey excludes roof-top plant areas
Office	30	60	90	120*	
Shops, commercial, assembly and recreation	30	60	90	120*	
Closed car parks	30	60	90	120	
Open-sided car parks	15	15	15	60	
* Sprinklers are required, but the fire resistance of the floor may be 90 minutes only.					

This table is based on UK practice; other European countries may have different requirements.

As an illustration of the differences in fire requirements, the German regulations state that open multi-storey car parks in Germany do not require any fire resistance (R0).



Figure 2.1 Unprotected open multi-storey car park in Germany (R0)

When adopting a prescriptive approach, fire behaviour modelling is not required and thermal actions should be based on the standard temperature-time curve (ISO curve).

The heat transfer calculations used to determine the temperature-time history of the structure are iterative and usually require some form of automation. Simple equations for protected and unprotected steel sections are provided by EN 1993-1-2^[3] and EN 1994-1-2^[4]. Alternatively, there are a number of commercially available software packages based on finite element or finite difference analysis methods. For steel-concrete composite structural elements, the design methods provided in the Informative Annexes to EN 1994-1-2 also include tabular data which avoid the need for heat transfer analysis, but the methods are restricted in scope.

Compliance with prescriptive requirements may be demonstrated using analysis, tabulated results based on testing and/or analysis, or individual results obtained from standard fire tests.

Structural analysis may be based on simple engineering models that consider individual structural members or finite element models that allow structural sub assemblies or full structures to be analysed.

Tabular Data

Tabular data on fire performance may provide generic or product specific data. Generic tabular data for steel and steel-concrete composite structures is available in EN 1993-1-2 and EN 1994-1-2. The critical temperature method in

EN 1993-1-2 provides a table of critical temperature data expressed in terms of the degree of utilisation. Data for the design of a range of steel concrete composite beams and columns is also provided in tabular format in EN 1994-1-2.

Product-specific tabular data is commonly available for fire protection materials that can be applied to structural steelwork. The required thickness of fire protection is usually expressed as a function of fire resistance period, section factor and critical temperature. Many other building products, such as steel decking for composite floors, also come with product specific design tables.

Simple calculation models

As an alternative to tabular data, the designer may choose to carry out simple calculations to determine the resistance of a member after a given period of fire exposure. The fire action is taken as the standard temperature-time curve in the Eurocodes. The resistance of the member depends on the level of applied load and loss of material strength under fire conditions, and is calculated according to the Eurocode rules.

Simple calculation models are most practical when used to justify the design of unprotected steelwork, usually in circumstances where the fire resistance requirement is R30 or lower. Where protected steelwork is required normal tabular data provided by fire protection manufacturers is more efficient.

The disadvantage of these calculation models is that their application is limited to individual structural members and does not allow for the interaction between the structural member and the surrounding structure.

Advanced calculation models

Advanced calculation models use both engineering fundamentals and finite element techniques to carry out structural fire design. Thermal and structural responses to fire actions may be determined using advanced physical models.

This type of analysis normally leads to a more economic solution than both the prescriptive approach and the simple calculation models. This method helps designers to develop more innovative solutions for building structures and it often proves that it is safe to leave some steel members unprotected without compromising the fire resistance of the structure.

Advanced calculation models demand a considerable amount of calculations to be carried out and require significant expertise from the designer in terms of structural fire engineering and finite element techniques.

Classification of lining materials

National regulations impose controls on the materials used as lining for the walls and ceilings of buildings. These materials must resist the spread of flames across the surfaces and must not contribute significantly to the fire in terms of heat release or smoke production. Lining materials are categorised on the basis of the result of reaction to fire tests.

External fire spread

In order to control the size of a fire, national regulations normally require that the potential for fire spread between buildings is considered. In some cases, the fire resistance requirements for external walls will depend on the distance between buildings reflecting the fact that radiated heat is the main fire risk for the adjacent building. As the space between the buildings increases the intensity of the heat flux diminishes.

Means of escape

Prescriptive regulations also require that escape routes are provided to allow occupants to move to a place of safety outside the building in the event of fire. Depending on the building usage and the likely number of occupants the regulations may prescribe the number of exits required, the width of the stairs or corridors used as fire escapes and the maximum distance that needs to be travelled from the most remote point in the building.

Table 2.3 gives an appreciation of the importance of travel distances in the design of escape stairs. Note that the value of the maximum travel distance in different countries may vary.

Table 2.3 Typical maximum travel distances (m) to fire-protected areas or escape stairs

Type of building	One direction	Multiple directions
Residential	9	18
Office	18	45
Shop	18	45

The design of means of escape influences the layout of the building plan in terms of arrangements of doors, corridors and, in particular, the number and locations of staircases in the building.

Access for emergency services

Prescriptive regulations require that access and facilities are provided for the fire service, so that they can carry out fire-fighting and rescue operations. In multi-storey buildings there may be a requirement for a fire-fighting shaft, including a staircase and/or lifts to allow fire fighters a means of getting personnel and equipment to the floor affected by fire. The fire-fighting shaft may also contain a wet or dry water main running vertically up the building which can be used to provide the necessary water supply for fire fighting operations. The fire fighting shaft is designed to given fire-fighters a place of relative safety from which they can launch fire-fighting operations into the fire-affected compartment. Fire-fighting shafts usually take up more floor space than a normal stairway and lift shaft, so these requirements can have a significant impact on building design and need to be considered at an early stage. The area around the building must also allow access for emergency vehicles, usually to within reasonable proximity of the wet or dry water main inlets.

2.3.2 Performance-based approach

A performance-based fire design procedure should be clearly documented so that the philosophy and assumptions can be clearly understood by a third party. The procedure may include the following main steps:

- Review the architectural design of the building
- Establish fire safety objectives
- Identify fire hazards and possible consequences
- Establish possible fire safety strategies
- Identify acceptance criteria and methods of analysis
- Establish fire scenarios for analysis.

Review of architectural design

This review should aim to identify any architectural or client requirements that may be significant in the development of a fire safety solution, for example:

- The future use of the building and the anticipated building contents, as well as the anticipated permanent, variable and thermal actions
- The type of structure and building layout
- The presence of smoke ventilation systems or sprinkler systems
- The characteristics of the buildings occupants, the number of people likely to be in the building and their distribution
- The type of fire detection and alarm system
- The degree of building management throughout the life of the building (for example, maintaining active fire safety measures or ensuring that combustible materials are not allowed to accumulate in vulnerable areas).

Fire safety objectives

At an early stage of the design process, the fire safety objectives should be clearly identified. This process will be undertaken ideally in consultation with the client, regulatory authority and other stakeholders.

The main fire safety objectives that may be addressed are life safety, control of financial loss and environmental protection.

Life safety objectives are already set out in prescriptive regulations, but should include provisions to ensure that the building occupants can evacuate the building in reasonable safety, that fire-fighters can operate in reasonable safety and that collapse does not endanger people who are likely to be near the building.

The effects of fire on the continuing viability of a business can be substantial and consideration should be given to minimising damage to the structure and fabric of the building, the building contents, the on-going business viability and the corporate image. The level of precautions that are deemed necessary in a particular building will depend on the size and nature of the business undertaken in it. In some cases, it may be easy to relocate the business to

alternative premises without serious disruption; in other cases, business may stop until the building is reinstated. Many businesses that experience fires in their premises go bankrupt before resuming business following a fire.

A large conflagration which releases hazardous materials into the environment may have a significant impact on the environment. The pollution may be air borne or water borne, as a result of the large volumes of water required for fire fighting operations.

Identification of fire hazards and possible consequences

A review of potential fire hazards may include consideration of ignition sources, the volume and distribution of combustible materials, activities undertaken in the building and any unusual factors. When evaluating the significance of these hazards, consideration needs to be given to the likely consequences and their impact on achieving the fire safety objectives.

Possible fire safety strategies

In order to quantify the level of fire safety achieved, one or more possible safety strategies should be proposed for the building. These will normally be the most cost-effective strategies that satisfy the fire safety objectives.

The fire safety strategy is an integrated package of measures in the design of multi-storey buildings. The following should be considered when developing a fire safety strategy:

- Automatic suppression measures (e.g. sprinkler systems) to limit the likelihood of the spread of fire and smoke
- Automatic detection systems, which provide an early warning of fire
- Compartmentation of the building with fire resisting construction and the provision of fire resisting structural elements, to ensure structural stability
- Means of escape-provision of adequate numbers of escape routes (of reasonable travel distances and widths) considering the number of people likely to occupy the building at any time
- Automatic systems, such as self-closing fire doors or shutters, to control the spread of smoke and flames
- Automatic smoke control systems, to ensure smoke free escape routes
- Alarm and warning systems, to alert the building occupants
- Evacuation strategy
- First aid fire fighting equipment
- Fire service facilities
- Management of fire safety.

Establishing acceptance criteria and methods of analysis

The acceptable performance criteria for performance-based design will be based on the global analysis of a given fire strategy. The criteria have to be established and evaluated following a discussion between designers and clients, using comparative, deterministic or probabilistic approaches.

A comparative approach evaluates the level of fire safety obtained from performance-based design in relation to that from a prescriptive approach, to ensure an equivalent level of fire safety is achieved. A deterministic approach aims to quantify the effects of a worst case fire scenario and to demonstrate that the effects will not exceed the acceptance criteria defined. A probabilistic approach aims to show that the fire safety strategy makes the likelihood of large losses occurring sufficiently small.

Establishing fire scenarios

The number of possible fire scenarios in any building can become large and resources to analyse all of them are usually not available. Therefore detailed analysis must be confined to the most significant fire scenarios or the worst creditable case as it is sometimes referred to. The failure of protection systems should also be included in the fire scenarios considered. In most buildings more than one fire scenario will require detailed analysis.

2.3.3 Choice of optimum approach

The choice of an optimum approach for the fire design of multi-storey buildings depends on various parameters, such as geometry, structural features, service function and designers' knowledge of fire design. Table 2.4 provides some suggestions as to which approach may result in a more economic solution.

Overall, for low-rise buildings with a small floor area, the pre-engineered datasheet approach may be the optimum choice. For high-rise buildings with a large floor area or to take into account the benefits of active protection measures, economic benefits may be obtained using advanced calculation models. For most medium size buildings, simple calculation models may result in the most economic solution.

Table 2.4 Choice of optimum approach for fire engineering design

Features of building	Datasheets	Simple calculation models	Advanced calculation models
1 Building size - floor area per storey, <i>A</i>			
Small: $A < 200 \text{ m}^2$	✓✓		
Medium: $200 \text{ m}^2 < A < 2000 \text{ m}^2$	✓✓		
Large: $A > 2000 \text{ m}^2$	✓	✓	✓
2 Building height – number of storeys, <i>n</i>			
Low: $n \leq 5$	✓✓	✓	
High: $n > 6$	✓	✓	✓

- ✓✓ Most economic solution
- ✓ Probably an economic solution

The use of active fire protection measures, such as detectors, alarms or sprinklers, is beneficial if advanced calculation methods are used. It is noted

that some national regulations and/or local authorities allow a reduction in the fire loads when these measures are present.

The ease of access to specialists in fire design will facilitate the use of advanced methods. However, for those designers with no experience in the field, the use of simple methods or the datasheets is likely to make the project more economic, as important savings can be made in the design costs.

In this regard, Table 2.5 outlines the recommended approach to be used depending on the skills of the designer and indicates which Section of this guide covers the relevant information.

Table 2.5 Impact of the skills of the designer in the fire solution

Knowledge in fire engineering design	Recommended design approach (section of this guide which covers the method)
Non-specialist	Active fire protection (3.1) Passive fire protection (3.2) Alternative fire resisting systems (3.3)
Limited knowledge	Simple calculation methods (4) Tensile membrane action (5)
Specialist	Advanced calculation methods (6)

3 FIRE PROTECTION SOLUTIONS

3.1 Active fire protection

Active fire protection measures include installation of detectors, alarms and sprinklers, which can detect fire or smoke and suppress the fire at its earliest stage.

These fire protection systems can have a significant influence on the level of life safety and property protection that can be achieved in a building. Prescriptive regulations usually require detection and alarm systems to be installed and these, along with sprinklers, often form an important part of the fire safety strategy in performance-based designs.

3.1.1 Detectors and alarms

As part of a strategy based on active fire protection measures, a number of detectors have to be installed in multi-storey buildings. These devices can detect heat, smoke and flames. Fire alarm systems are designed to alert occupants of the need to evacuate the building due to a fire outbreak. Figure 3.1 shows a typical detector and alarm device.



Figure 3.1 Fire detector and alarm device

3.1.2 Sprinklers

Sprinklers are devices that automatically suppress a small fire, either on its ignition or shortly after its ignition.

As shown in Figure 3.2, a sprinkler normally has a glass bulb, which contains a volatile liquid and seals the water nozzle. In the event of fire, the heated liquid expands, breaks the glass bulb and thus activates the sprinkler head.

Sprinklers contribute to both structural fire safety and building property protection. In some countries use of sprinklers in a multi-storey building may lead to a reduction in the required fire resistance period, but this should be checked with the relevant national regulations.



Figure 3.2 Sprinkler and its activation

3.2 Passive fire protection

The high temperatures induced in a building by the outbreak of a fire affect all construction materials, such that their strength and stiffness is reduced as the temperature increases. It is often necessary to provide fire protection to structural members in multi-storey steel buildings in order to delay the loss of load bearing capacity. Structural members may be insulated by using fire protection materials, such as boards, sprays and intumescent coatings. The performance of these fire protection materials is tested and assessed in accordance with EN 13381^[5].

The thickness of protection required in a given building will depend on the fire protection material selected, the fire resistance period required by national building regulations, the section factor of the member to be protected, and the critical temperature of the member.

3.2.1 Fire protection methods and materials

There are two types of passive protection materials, namely non-reactive and reactive. Non-reactive protection materials maintain their properties when they are exposed to a fire. Boards and sprays are the most common non-reactive materials. Reactive protection materials are characterised by a change in their properties when they are subject to fire. The most well known example of this type of protection is the intumescent coating.

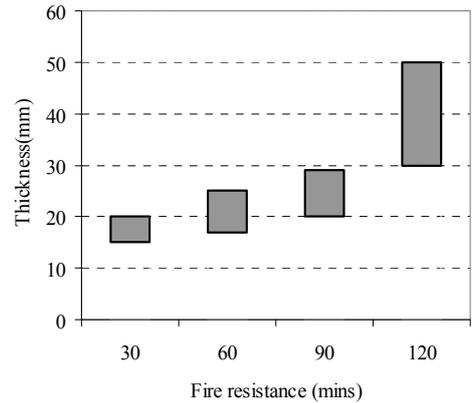
Boards

A variety of proprietary boards, with thicknesses ranging from 15 to 50 mm, are widely used to protect steel members to achieve a 30 to 120 minute fire resistance.

Boards are generally manufactured from either mineral fibres or naturally occurring plate-like materials such as vermiculite and mica, using cement and/or silicate binders. Boards may be fixed to steel members either mechanically, using screws, straps and/or galvanized angles, or glued and pinned.



(a)



(b)

Figure 3.3 Board fire protection (a) Fixing of boards to steel column; (b) Fire resistance

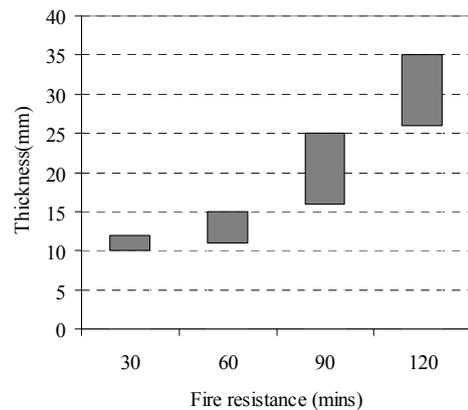
Boards are factory-manufactured and therefore, their thickness and quality can generally be guaranteed. They offer a clean, boxed appearance, which may be pre-finished or ready for further decoration. However, boards cannot easily be fitted to members with complex shapes. Boards generally cost more than sprayed and intumescent coatings, although non-decorative boards may be cheaper than decorative ones. In addition, the time required to fix the boards is significant compared to the application of intumescent coatings, which not only increases the construction costs but it also affects the construction programme of multi-storey buildings.

Sprayed non-reactive coatings

The process of applying this type of protection is shown in Figure 3.4(a). Sprayed non-reactive coatings with thicknesses of 10 to 35 mm are widely used to protect steel members, to achieve a fire resistance of between 30 and 120 minutes, as shown in Figure 3.4(b).



(a) Application



(b) Fire resistance for a given thickness

Figure 3.4 Sprayed non-reactive fire protection (a) Application (b) Fire resistance

Sprayed non-reactive coatings are mainly cement or gypsum based materials that contain mineral fibre, expanded vermiculite and/or other lightweight aggregates or fillers. This type of protection is applied in situ and it is particularly suitable for members that have complex profiles and are not visible in use. However, spraying the protection in situ may require extensive shielding and affect the construction programme.

Intumescent coatings

In contrast with the non-reactive boards and sprays, intumescent coatings react in fire, and change their properties from an initial decorative paint into an intumescent layer of carbonaceous char, by swelling to about 50 times their original thickness. Typical initial thicknesses of 0,25 to 2,5 mm can provide a fire resistance of 30 to 120 minutes, as shown in Figure 3.5.

Intumescent coatings are similar in appearance to conventional paints, and may be solvent based or water borne. They consist of three layers which include a compatible primer, the intumescent coat and a top coat or sealer coat (often available in a wide range of colours). Currently, most intumescent coatings are applied off site to aid the construction programme.

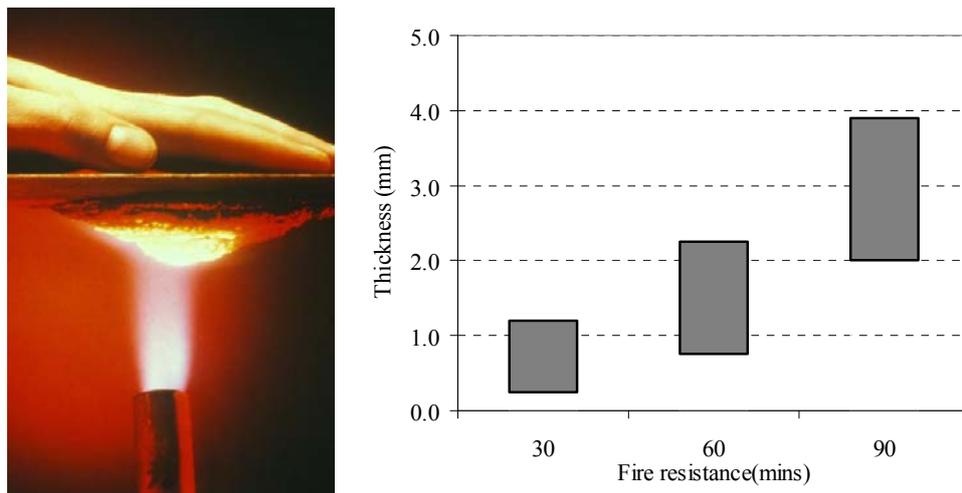


Figure 3.5 Intumescent coatings

Some intumescent coatings are also used for external applications and for heritage applications, where the appearance of the building must be maintained.

3.2.2 Thickness of fire protection materials

For a given product, the thickness of fire protection depends on the required fire resistance and on the section factor of the steel members. The section factor varies with the choice of fire protection and with the type and size of steel member. Figure 3.6 gives expressions to calculate the section factor, based on the configuration of fire protection and on the geometric properties of the steel section.

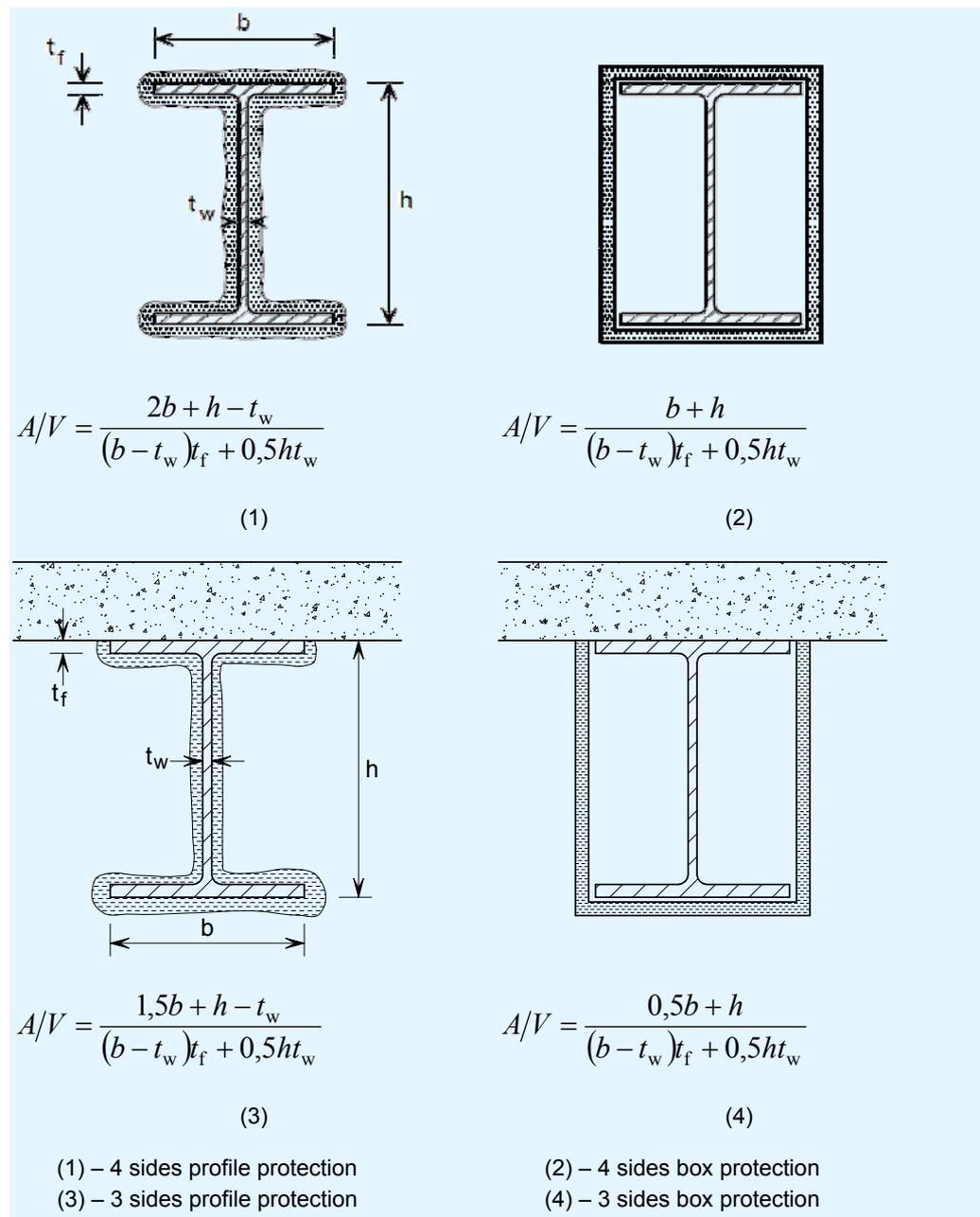


Figure 3.6 Protection configuration and section factor

The required fire resistance period is determined by the national building regulations in each country. Based on datasheets, such as Table 3.1 for board protection, Table 3.2 for sprayed non-reactive coating and Table 3.3 for intumescent coating, the thickness of protection materials can be easily defined.

Datasheets are usually published by manufacturers. In some countries manufacturers' datasheets have been consolidated into reference documents published by the relevant body and agreed for use by the designers.

Table 3.1 Example of datasheet for board fire protection

Thickness of board protection (mm)	Maximum Section factor A_m/V (m^{-1}) for beams and columns			
	R30	R60	R90	R120
20	260	260	125	70
25			198	110
30			260	168
35				232
40				256
45				260

Table 3.2 Example of datasheet for sprayed non-reactive coatings

Section factor A_m/V (m^{-1})	Required thickness (mm) of sprayed non-reactive coatings			
	R30	R60	R90	R120
40	10	10	11	15
80	10	12	16	21
120	10	14	19	24
160	10	15	21	26
200	10	16	22	28
240	10	16	23	29
280	10	17	23	30

Table 3.3 Example of datasheet for intumescent coatings for R60

Section factor A_m/V (m^{-1})	3 side- I beam	4 side- I beam	4 side- I column
40	0,25	0,26	0,26
80	0,31	0,39	0,39
120	0,39	0,53	0,53
160	0,48	0,66	0,66
200	0,69	0,83	0,83
240	0,90	1,00	1,00
280	1,08	1,74	1,74

3.3 Fire resisting construction

In recognition of the importance of structural fire resistance and the costs associated with passive fire protection materials, some alternative structural systems have been developed which utilize their inherent fire resistance and avoid the need of dedicated fire protection. These systems include composite floors, integrated beams and encased steelwork.

3.3.1 Composite floor

Precast concrete slabs have an inherent fire resistance of up to 120 minutes if adequate provision and detailing of rebar is made. However, the slabs usually rest on the top flange of a down stand beam (see Figure 3.7(a)), which is exposed on three sides and requires fire protection.

As an alternative to precast concrete floors, composite floors, shown in Figure 3.7(b), are popular in multi-storey buildings.

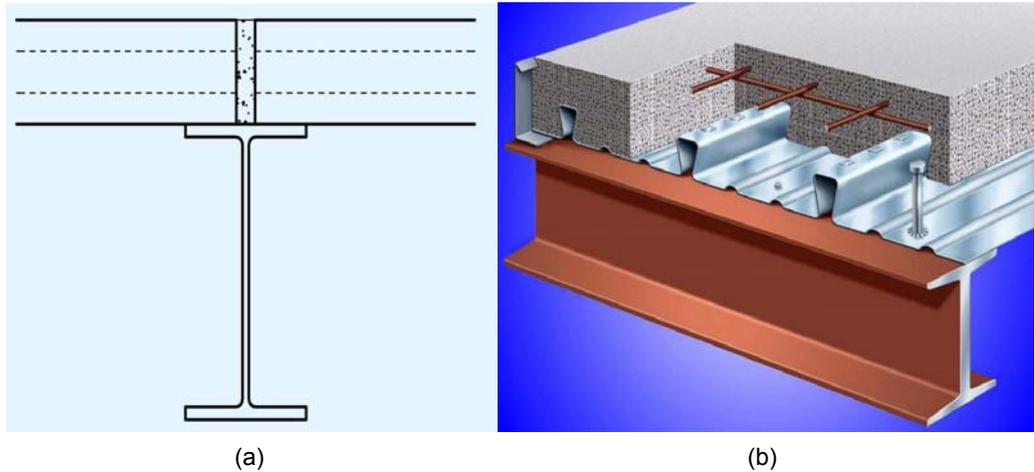


Figure 3.7 (a) Precast concrete floor on down stand beam; (b) composite floors with steel decking

Composite floors are constructed using either trapezoidal or re-entrant profiled steel decking that supports the concrete on top. In a composite floor, the concrete is reinforced using fibre or bar reinforcement to control cracks caused by the flexural tension at the floor support, and by shrinkage and settlement of the concrete. In addition to controlling cracks, the bar reinforcement also provides the floor with bending resistance at its support in the fire condition.

For a composite floor with the bottom surface exposed, the insulation criterion is usually satisfied by providing sufficient insulation depth of the concrete for the required fire resistance period, as shown in Table 3.4. The longer the required fire resistance period, the thicker the insulation concrete needs to be. The integrity criterion is generally met by using continuous steel decking.

Table 3.4 Typical minimum insulation thickness (mm), h_1 , of concrete in composite floors

Required Fire Resistance (minutes)	Trapezoidal steel decking		Re-entrant steel decking	
	Normal weight concrete	Light weight concrete	Normal weight concrete	Light weight concrete
60	70	60	90	90
90	80	70	110	105
120	90	80	125	115

Table 3.5 shows typical depths and spans of composite floors with normal weight concrete under a uniform applied load of 5,0 kN/mm².

Table 3.5 Typical depths and spans of composite floors

Required fire Resistance (minutes)	Trapezoidal steel decking				Re-entrant steel decking			
	Single span		Double span		Single span		Double span	
	Depth (mm)	Span (m)	Depth (mm)	Span (m)	Depth (mm)	Span (m)	Depth (mm)	Span (m)
60	140	3,8	140	4,2	101	3,0	101	3,4
90	150	3,1	150	3,3	105	2,9	105	3,3
120	160	3,1	160	3,4	115	2,4	115	2,9

3.3.2 Integrated beams

Integrated beams are part of a floor construction system where the steel beams are contained within the depth of an in-situ or pre-cast concrete slab, instead of supporting the slab on the top flanges of the beams. Consequently, the overall depth of the floor construction is minimized. The whole steel section, except for its bottom flange or plate, is insulated from the fire by the surrounding concrete.

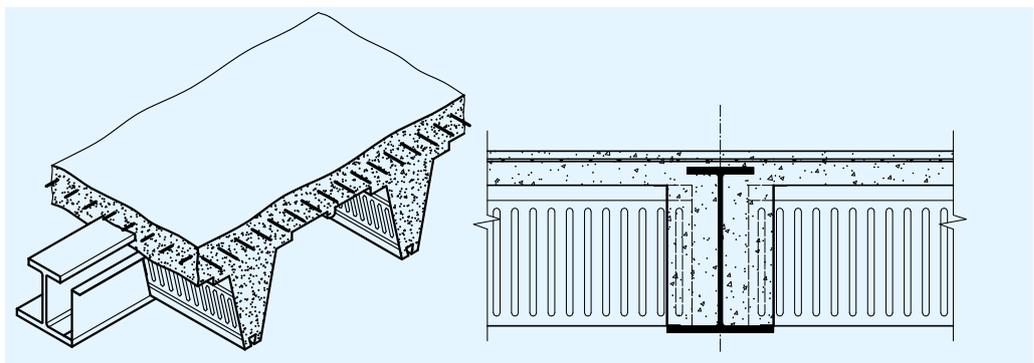


Figure 3.8 ASB - Integrated beam (rolled asymmetric steel beam)

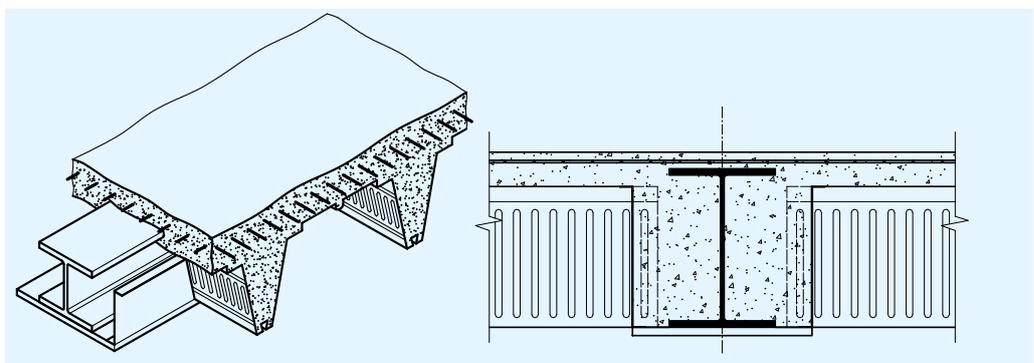


Figure 3.9 IFB - Integrated beam (I-section with a plate welded to its bottom flange)

There are two types of open section integrated beams: asymmetric steel beam (ASB) and integrated fabricated beam (IFB), as shown in Figure 3.8 and Figure 3.9 respectively.

Generally, integrated beams can achieve up to 60 minutes fire resistance without being fire-protected. With the inclusion of additional longitudinal reinforcement, 90 minutes fire resistance is possible with an unprotected bottom flange. The total structural depths and typical spans of integrated beams are summarised in Table 3.6, which may be referred to at the preliminary design of multi-storey buildings.

Table 3.6 Structural depth and typical span of integrated beams

Type of floor	Depth (mm)	Span (m)
ASB Integrated beam	280 to 400	6 to 9
IFB Integrated beam	250 to 450	6 to 9

3.3.3 Partially encased beams and columns

Partially encased beams and columns are constructed by filling the space between the flanges of I-sections using plain or reinforced concrete, as shown in Figure 3.10 and Figure 3.11.

Compared with the unprotected I-sections, which only have about 15 minutes fire resistance, partially encased sections can achieve over 60 minutes, which normally meets the fire resistance requirements of many multi-storey buildings. The increase in fire resistance period is due to the coverage of most parts of the surface area of the steelwork using concrete, which has a low thermal conductivity. Longer periods of fire resistance can also be achieved by increasing the amount of reinforcement embedded within the concrete, to compensate the loss of the strength of the steelwork in fire.

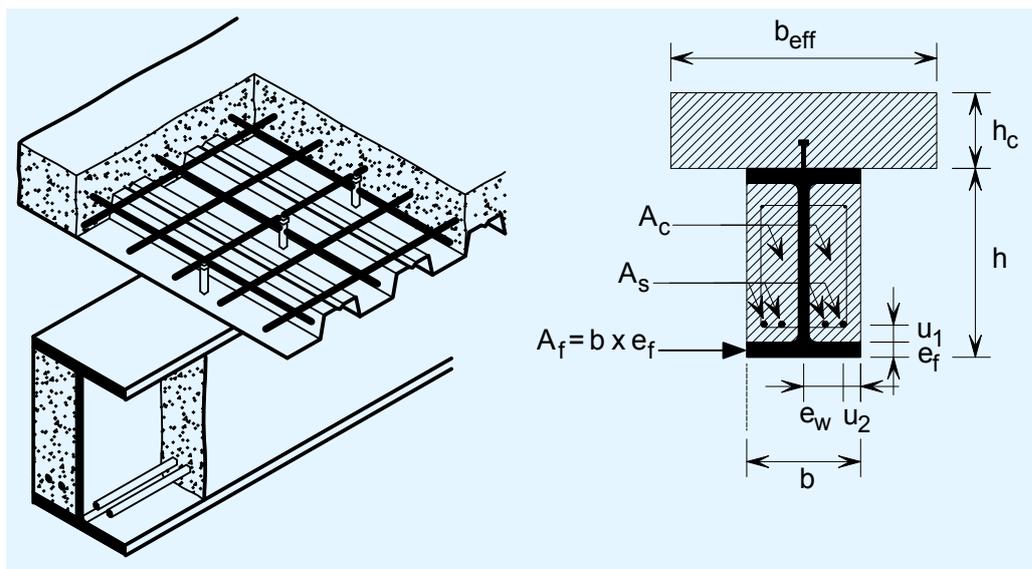


Figure 3.10 Partially encased beam

EN 1994-1-2 offers relatively simple rules and established datasheets in clauses §4.2.2 and §4.2.3 for fire design of composite beams and columns, including partially encased steel sections. These rules relate the fire resistance of

composite members to their load level (the load level is denoted as $\eta_{fi,t}$ and is described in Sections 4.4.2 and 4.4.3 of this guide), the h/b ratio, the member type and the area of reinforcement A_s .

Generally, an increase of the fire resistance or the load level requires larger cross-sections and additional reinforcement for partially encased sections. The datasheets can be used to determine the minimum cross-sectional dimensions (such as the section width b_{min}) and reinforcement ratio $A_{s,min}$ of partially encased sections, to achieve the required fire resistance period.

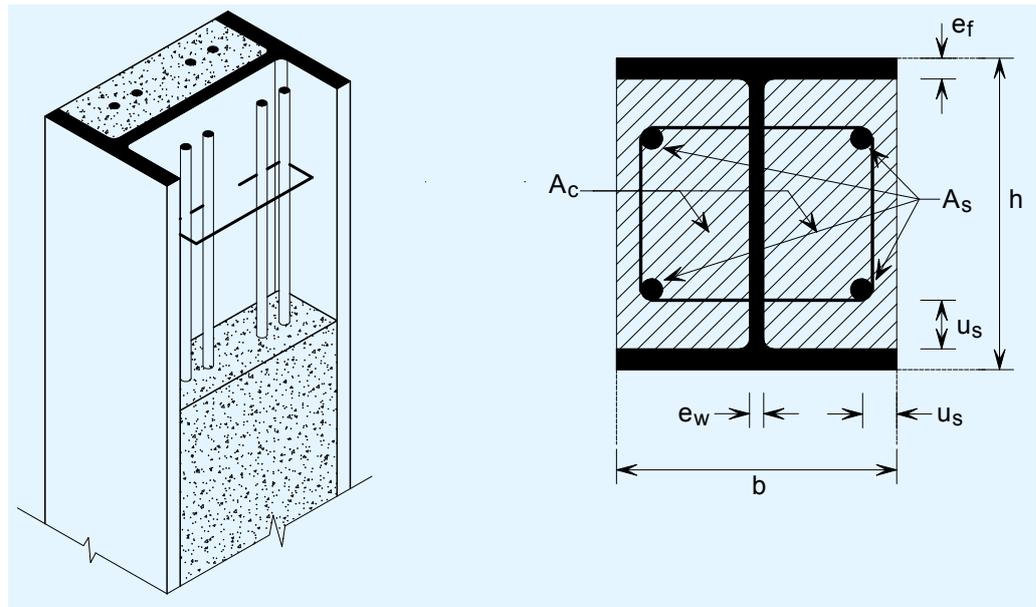


Figure 3.11 Partially encased column

Table 3.7 shows a datasheet taken from EN 1994-1-2 for the fire design of partially encased sections. When using this type of design data, the load level $\eta_{fi,t}$ may be calculated as follows:

$$R_{fi,d,t} = \eta_{fi,t} \cdot R_d$$

where:

$R_{fi,d,t}$ is the design resistance of the member in fire conditions at time, t .

R_d is the design resistance of the member for room temperature design.

When calculating the load level, EN 1994-1-2 recommends that the design resistance for room temperature design, R_d , is calculated for a buckling length that is twice the buckling length used for fire design.

Table 3.7 Typical datasheets for fire design of partially encased sections

Member	Section Ratio h/b	Load level	Width b_{\min} (mm)/Reinforcement $A_{s,\min}$ ratio (%) for the required fire resistance period (min)				
			R30	R60	R90	R120	R180
Beam	>1,5	$\eta_{fi,t} \leq 0,5$	80/0,0	150/0,0	200/0,2	240/0,3	300/0,5
		$\eta_{fi,t} \leq 0,7$	80/0,0	240/0,3	270/0,4	300/0,6	
	>3,0	$\eta_{fi,t} \leq 0,5$	60/0,0	100/0,0	170/0,2	200/0,3	250/0,3
		$\eta_{fi,t} \leq 0,7$	70/0,0	170/0,2	190/0,4	270/0,5	300/0,8
Column	Minimum h and b	$\eta_{fi,t} \leq 0,47$	160/-	300/4,0	400/4,0		
		$\eta_{fi,t} \leq 0,66$	160/1,0	400/4,0			

As an example, consider the case of a partially-encased beam with a ratio $h/b > 3$, under moderate load ($\eta_{fi,t} \leq 0,5$).

For a fire resistance period of 60 minutes (R60):

- The width should not be less than 100 mm, which leads to $h > 3b = 300$ mm. So the minimum cross-sectional area is 100×300 mm
- No reinforcement is required, $A_s = 0$.

To achieve a fire resistance period of 120 minutes (R120):

- The width should not be less than 200 mm. Therefore the height $h > 3b = 600$ mm and therefore the minimum cross-sectional area is 200×600 mm
- The reinforcement area, A_s , should not be less than 0,3% of the encased concrete area A_c , i.e., $A_s \geq 0,003A_c$.

3.3.4 External steelwork

In some circumstances, the main structural members, such as columns and beams, may be located outside the building envelope without any fire protection, as shown in Figure 3.12. The fire protection requirements of external steelwork are significantly reduced, as the temperature of the external steelwork is lower compared to members inside a compartment.

Further guidance on use of external steelwork for fire design can be found in EN 1993-1-2, §4.2.5.4.

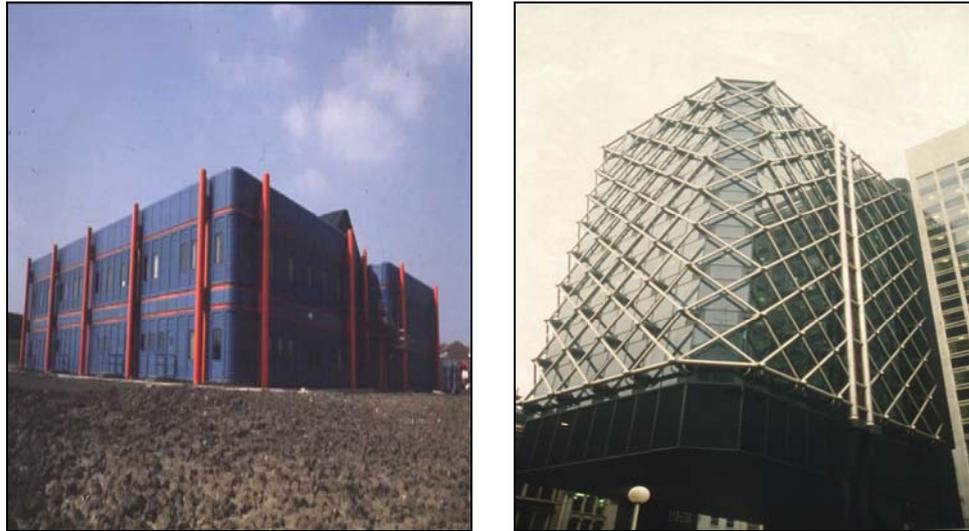


Figure 3.12 Use of external steelwork

3.3.5 Combined solutions for fire resistance

By careful selection of the structural solutions, up to 60 minutes of fire resistance can be achieved for multi-storey buildings without applying fire protection materials. Generally, this will demand the use of integrated beams and partially encased or concrete filled columns. Table 3.8 shows the fire resistance period that may be achieved for a structure using combinations of beam and column construction discussed above. This table considers the fire resistance of individual elements and less conservative results can generally be achieved by considering fire design based on assemblies of structural elements using methods such as FRACOF (see Section 5.2).

Table 3.8 Combined solution for steel frame with or without fire protection

	Unprotected beam	External beam	Integrated beams	Encased beam	Protected beam
Fire resistance period (min)					
Unprotected steel column	15	15	15	15	15
External column	15	>30	>30	>30	>30
Encased column	15	>30	>60	>60	>60
Protected column	15	>30	>60	>60	>60

Note: Fire resistance period stated is the lower value for the beam or column construction.

4 SIMPLE CALCULATION MODELS

In previous Sections, the prescriptive approach to fire design through use of datasheets has been shown to provide a safe solution. However, it does not necessarily offer the most efficient design. By using the simple calculation models described in the Eurocodes, the designer may be able to demonstrate that less or no protection is required in some or all of the structural elements, thus leading to a more economic solution to fire design.

There are two approaches to these simple calculations: the critical temperature approach and the load bearing capacity approach. Based on these methods the designer may reasonably decide whether or not fire-protection is required.

These methods, however, deal with individual members under standard fire exposure, instead of an entire structure in natural fire. Hence, unlike the performance-based analysis (see Section 6), they do not consider the actual behaviour of the structural member in a real fire.

In order facilitate the understanding of the reasoning behind the simple methods given in the Eurocode and described in Sections 4.4.2 and 4.4.3 of this guide, an introduction to the thermal effects of fire is given in Section 4.1.

4.1 Fire behaviour and thermal actions

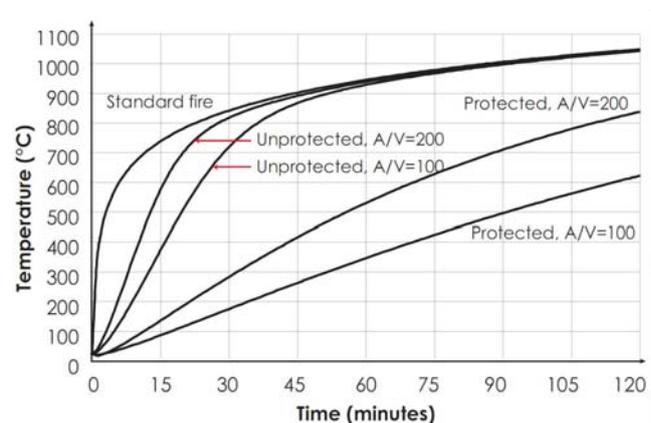
4.1.1 Fire action and standard fire

Fire is a very complex phenomenon that involves different kinds of chemical reactions. Fire releases heat energy in the form of flames and smoke within the building compartment, as shown in Figure 4.1 (a).

When a fire occurs, the temperature of the gas within the building compartment rises rapidly. For the purpose of fire design, the fire action is represented by a standard temperature-time curve, as defined in EN 1991-1-2. This curve is denoted ‘standard fire’ in Figure 4.1(b).



(a) Fire (Cardington test)



(b) Standard fire curve and temperature rise of members

Figure 4.1 Fire action

The temperature rise of a member in fire is predominantly governed by radiation and convection mechanisms, via a complex diffusion process. It depends on the thermal properties of the materials and the thickness of the protection layer, if it is fire-protected.

Due to the rapid increase of the gas temperature, the heat energy from the fire (i.e. thermal action) flows into the member through the exposed surfaces, and heats up the member. As a result, the temperature of the member rises, typically following the curves shown in Figure 4.1 (b) for different scenarios of protected and unprotected members.

4.1.2 Modelling fire behaviour

As a thermal action on building structures, a fire can be classified into localised and fully-developed fires.

Localised fire

A localised fire is in the pre-flashover phase and occurs only in some part of a compartment. A localised fire is unlikely to spread to the whole compartment and to cause a flashover, due to its slow propagation and the low temperature developed. A localised fire is generally modelled using plume, zones and computational fluid dynamics (CFD) models.

Plume model

Annex C of EN 1991-1-2 gives a so-called plume model to determine the thermal action of a localised fire. If the flame remains below the ceiling, as shown in Figure 4.2 (a), the model is used to calculate the corresponding temperature along its vertical axis. However, if the flame impacts the ceiling, as shown in Figure 4.2 (b) then the model determines the heat flux at the level of the ceiling together with the flame length.

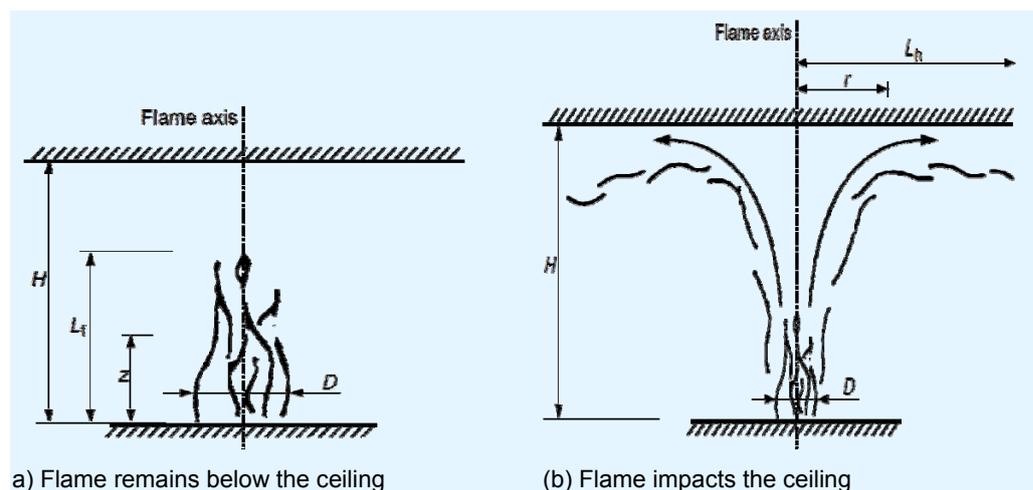


Figure 4.2 Plume model for a localised fire in EN 1991-1-2

4.1.3 Fully developed fire

A fire is fully developed when all the available fuels within a compartment are burning simultaneously and the maximum heat is released. A fully developed fire is commonly modelled using either standard or parametric fires, as shown

in Figure 4.3. The severity of a fully developed fire is governed by the ventilation condition and quantity and nature of the fuel within the compartment.

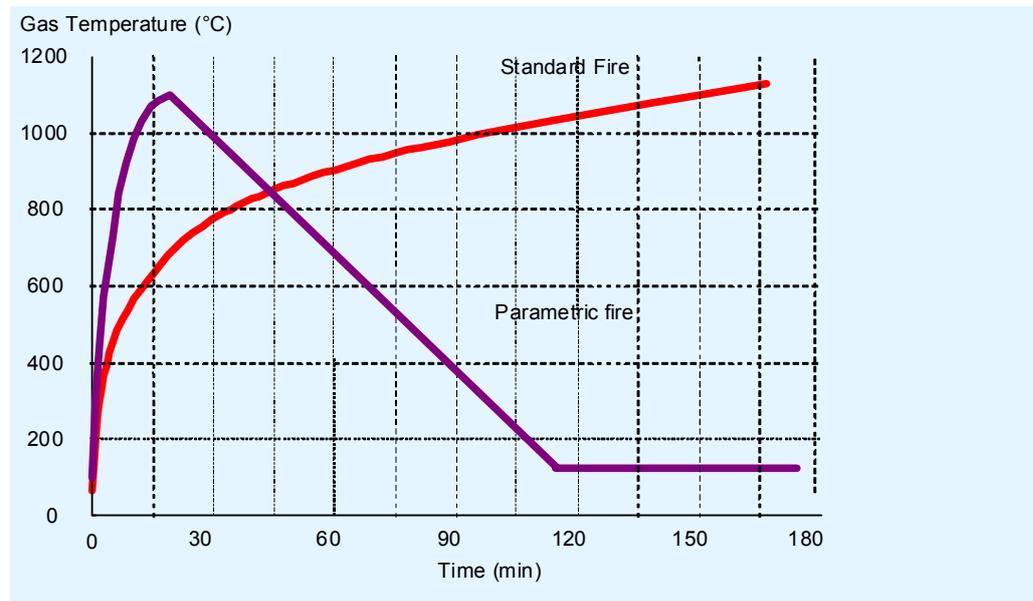


Figure 4.3 Model of fully developed fire

4.1.4 Nominal temperature-time curve

Standard fire

EN 1991-1-2, §3.2 uses the standard temperature-time curve to represent the thermal action of a fully developed fire. This ‘standard fire’ is used to classify the fire performance of the structural materials and members in standard fire furnace tests. It forms the basis on which the fire resistance time and load bearing resistance of structural members are evaluated using the simple calculation models from EN 1993-1-2 and EN 1994-1-2. It is also used in the performance-based analysis for fire design of a whole structure.

4.1.5 Natural fire model

Parametric fire

EN 1991-1-2, Annex A defines a parametric temperature-time curve for fire compartments up to 500 m² floor area. This ‘parametric fire’ consists of a heating, cooling and residual phases. The heating phase is normally represented using an exponential curve up to a maximum temperature. The cooling phase is described by a sloping straight line until it reaches the residual phase. For a parametric fire, the heating phase depends on ventilation conditions and the thermal properties of the compartment boundary. The heating duration and maximum temperature are governed by the density of the fire loads and the ventilation condition. The cooling phase is controlled by the heating duration and the maximum temperature that has been reached.

To some extent, the parametric fire represents the characteristics of a natural fire. However, its accuracy for estimating the thermal response of fire depends on the accuracy of the input data, such as fire load, ventilation condition, compartment size and thermal properties of the boundary condition.

4.2 Heat transfer

This Section explains the fire actions and the evolution of the temperature in structural members under a standard fire exposure, with an emphasis on the concepts of standard fire and section factor.

Section factor A_m/V

An important parameter in the rate at which the temperature of a member increases is the ratio A_m/V of the member, commonly known as the section factor. The section factor is defined as the ratio of the exposed surface area of a member to its volume per unit length.

The influence of the section factor is shown in Figure 4.1(b) for protected and unprotected members. A larger section factor leads to a faster heating of the member. For example, after 15 minutes of fire exposure, the temperature of an unprotected member with a section factor of $A_m/V = 200$ increases to about 580°C , while that of the unprotected member with $A_m/V = 100$ only reaches 380°C .

This difference is due to the fact that a large value of the section factor represents a large exposed surface area compared with its volume, and therefore, the member receives more heat than that with a low section factor, which represents a small exposed surface area. This is illustrated in Figure 4.4.

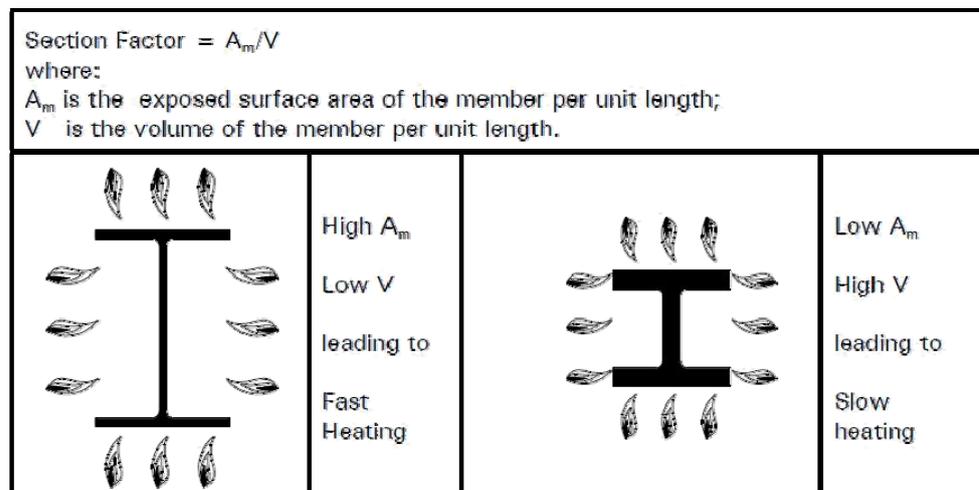


Figure 4.4 Definition of section factor A_m/V of a member in fire

Member temperature at time t

The temperature $\theta_{a,t}$ of the member at time t can be calculated using the simple models given in §4.2.5 of EN 1993-1-2 or §4.3.4.2 of EN 1994-1-2.

If the critical temperature exceeds the design temperature, i.e. $\theta_{cr} > \theta_{a,t}$ then the fire resistance of the unprotected member is adequate for that duration.

If the unprotected member is not adequate, measures have to be taken to improve its fire resistance by:

- Choosing a larger cross-section for the steel member
- Selecting a higher grade of steel
- Providing the necessary fire protection.

4.3 Structural Analysis

Figure 4.5 illustrates the general behaviour of a structural member under the effect of the standard fire and applied actions. As the gas temperature θ_g rises, the member temperature $\theta_{a,t}$ increases and its load bearing resistance $R_{fi,d,t}$ decreases.

The critical temperature θ_{cr} is defined as the temperature at which a member can no longer support the design effects of the applied actions. The effects of actions (also called action effects) on a member are the internal forces or moments induced by the actions. For example the wind blowing on a structure is an action (the wind action) and the internal forces and bending moments induced in a column are the effects.

Given a structure subject to various actions (e.g. wind, gravity), a member is subject to the design value of an effect, E_d . Under the same actions but also subject to fire, the effects on the member are modified, and denoted as $E_{fi,d}$.

The critical temperature θ_{cr} is therefore defined as the temperature at the time of failure, when the resistance of a member is equal to the effect on it:

$$R_{fi,d,t} = E_{fi,d}$$

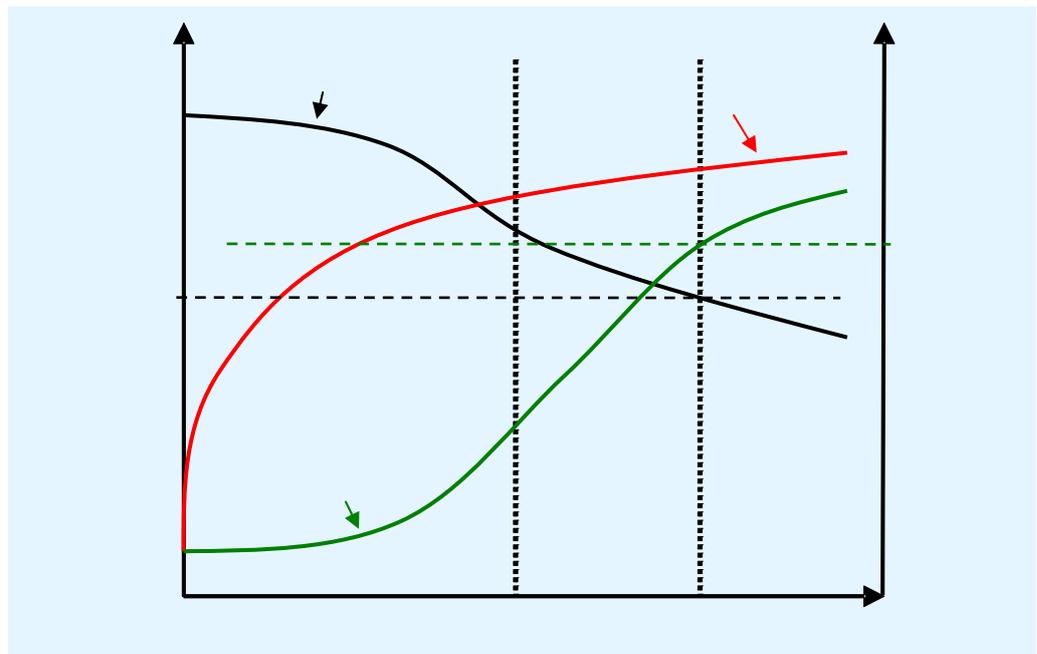


Figure 4.5 Behaviour of a structural member in fire

To meet the requirements of fire safety engineering, the designer must ensure that one of the following conditions is met:

- At $t_{fi,requ}$ the temperature of the member is lower than the critical temperature: $\theta_{cr} \geq \theta_{a,t}$ or
- At $t_{fi,requ}$ the resistance of the members is greater than the design effects: $R_{fi,d,t} \geq E_{fi,d}$.

4.4 Simple structural fire design methods

4.4.1 Introduction

The simple design methods follow the principle of ultimate limit design as for normal temperature. Fire safety requirements are satisfied by providing a design which will meet the above conditions.

Simple calculation models take account of the reduction of the action effects for fire design and the variation of material properties at elevated temperature. The simple calculation models have broader application than the prescriptive approach, and can be applied to:

- Unprotected steel members, including tension members, steel beams and steel columns.
- Unprotected composite members, including composite slabs and beams, concrete encased beams and columns, concrete filled hollow columns.
- Protected steel and composite members.

Design effect of actions, $E_{fi,d}$

The fire action is designated as an accidental action in the fire parts of Eurocodes. Due to the low probability of both a major fire and full external actions occurring simultaneously, the effect of actions for fire design can be determined by reducing the design effects of external actions for normal design using a reduction factor η_{fi} :

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

where:

E_d is the design effect of actions for normal design using EN 1990

η_{fi} is the reduction factor of the effect E_d for fire design, calculated as

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \text{ for combination 6.10 from EN 1990}$$

G_k is the characteristic value of the permanent action

$Q_{k,1}$ is the characteristic value of the leading variable action

ψ_{fi} is the combination factor given by $\psi_{1,1}$ or $\psi_{2,1}$, see EN 1991-1-2

γ_G is the partial factor for permanent actions

$\gamma_{Q,1}$ is the partial factor for the leading variable action.

For the calculation of the reduction factor for use with combination 6.10a and 6.10b, the designer should refer to EN 1993-1-2 §2.4.2.

4.4.2 Critical temperature approach

The basis for this approach is to calculate the temperature of the member after the required period of fire resistance and to compare it to the critical temperature, at which the member would collapse. The steps that must be followed to apply the critical temperature approach are shown in Figure 4.6.

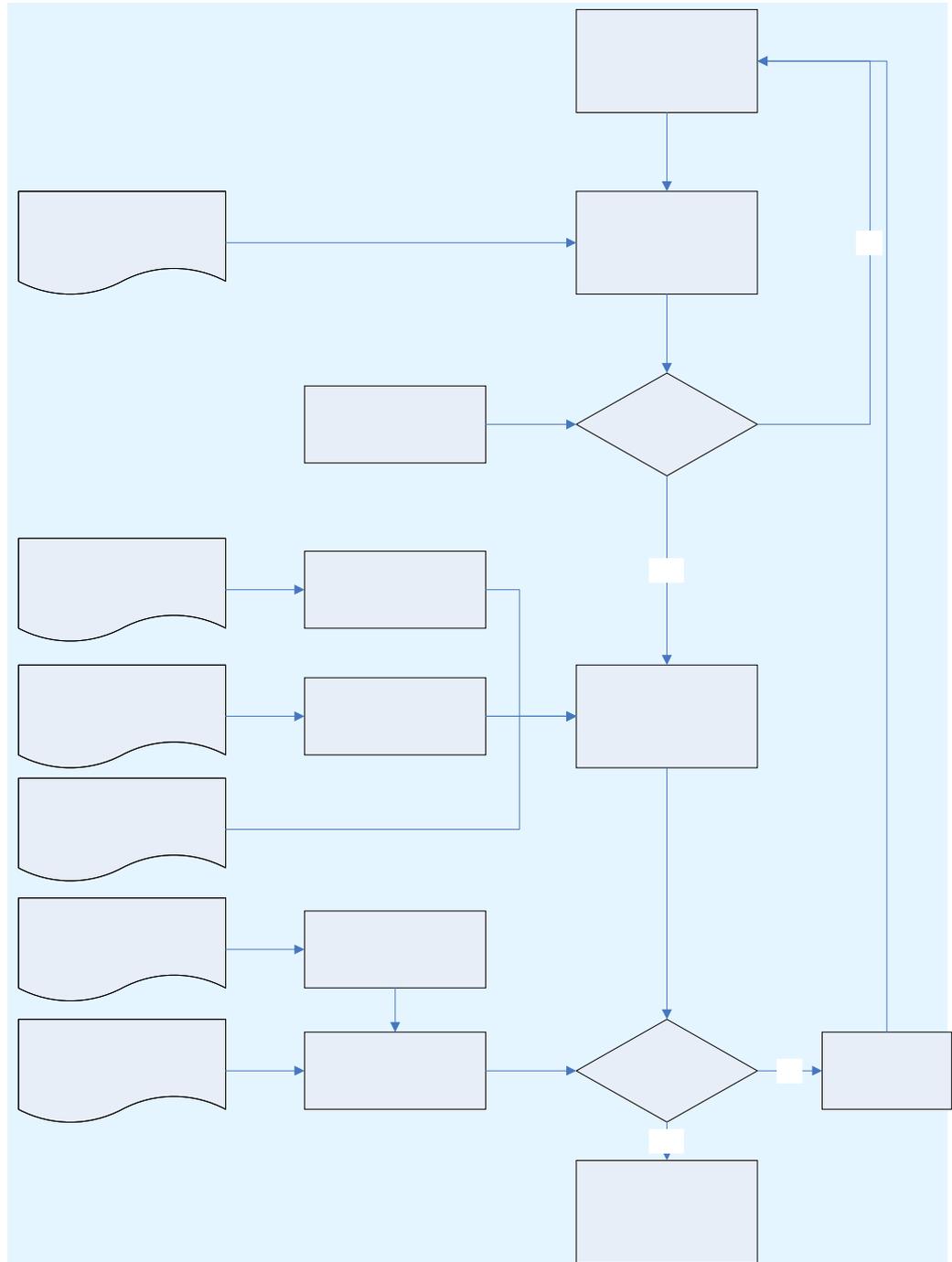


Figure 4.6 Fire design of members using the critical temperature approach

Critical temperature

The simple model with the critical temperature approach can only be used for individual members, when their deformation criteria or stability considerations do not have to be taken into account. This approach is only permitted for

tension members and for restrained beams, but not for unrestrained beams and columns when buckling is a potential failure mode.

The critical temperature of a non-composite steel member can be calculated using the simple model given in §4.2.4 of EN 1993-1-2. The critical temperature decreases with the degree of utilisation of the member, μ_0 , which is defined as the ratio of the design effect of actions at elevated temperature, $E_{fi,d}$, to the member resistance at normal temperature but using partial factors for fire design $R_{fi,d,0}$ such that:

$$\mu_0 = E_{fi,d}/R_{fi,d,0}$$

For Class 4 sections, a critical temperature of 350°C is recommended.

The critical temperature of composite members is given by §4.3.4.2.3 of EN 1994-1-2.

4.4.3 Load bearing capacity approach

The basis for this approach is to calculate the resistance of the member after the required period of fire resistance and to compare it to the design effect of the actions at elevated temperature, $E_{d,fi}$. The steps that must be followed to apply the load bearing capacity approach are shown in Figure 4.7.

Section classification

As for normal temperature design, cross-sections are classified according to Table 5.2 from EN 1993-1-1.

The ε coefficient is modified by applying a 0,85 factor, in order to account for the reduction in yield strength and elastic modulus of the steel at elevated temperatures, as specified in §4.2.2 of EN 1993-1-2. This modification lowers the c/t limits for the various section classes, so some sections may have a more onerous classification than for normal design.

Tension, shear and bending resistance of steel members in fire

The simple models to calculate the design tension, shear and bending resistances of steel members under standard fire are given in §4.2.3.1, §4.2.3.3 and §4.2.3.4 of EN 1993-1-2.

These models are based on the assumption that the members have a uniform temperature throughout, and make use of a reduced yield strength and the relevant partial factors for fire design.

However, in reality the temperature across and along a member is hardly ever uniform, which affects its mechanical behaviour. For example, if a steel beam supports a concrete slab on its upper flange, the temperature in the top flange is lower than in the bottom flange and therefore its ultimate moment resistance is higher than for a uniform temperature equal to that of the bottom flange.

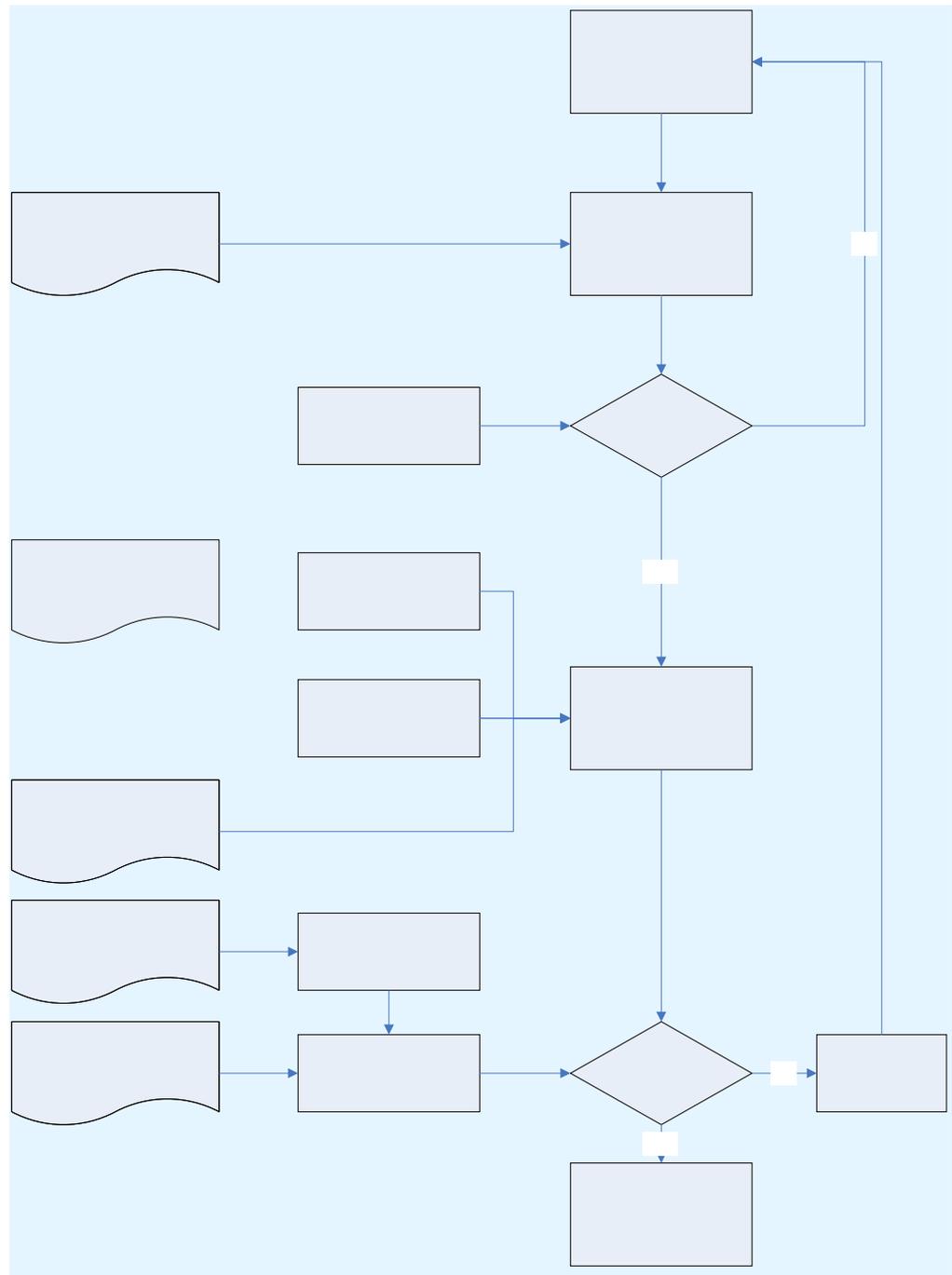


Figure 4.7 Fire design of members using the load bearing capacity approach

The benefits from such thermal gradient can be taken into account by dividing the cross-section into a number of elements and allocating a reduced yield strength to each element according to their temperature. The total resistance of the cross-section can be calculated by adding up the resistances of each element. Alternatively, the beneficial influence of thermal gradient on the resistance of these members can be calculated conservatively using two empirical adaptation factors, κ_1 and κ_2 , as given in §4.2.3.3 of EN 1993-1-2.

Resistances of unrestrained beams and columns

The simple models for calculating the buckling resistances of unrestrained beams and columns with a uniform temperature are given in §4.2.3.3, §4.2.3.4 and §4.2.3.2 of EN 1993-1-2. These rules were developed by imposing the following modifications on the relevant models given in EN 1993-1-1 for normal design to account for the effect of elevated temperature:

- Reduced yield strength for elevated temperature
- Use of partial factors for fire design
- Increased non-dimensional slenderness
- Buckling length of columns in fire is taken as 0,5 and 0,7 of the system length for the top storey and for the other storeys, respectively
- Specific buckling curves for the fire condition.

Resistance of composite members in fire

The simple models to calculate the resistances of composite members in fire are given in §4.3 and Annexes D, E, F, G and H of EN 1994-1-2 for composite slabs, composite beams, encased beams, encased columns and filled columns.

Since the temperature generally is not uniformly distributed over the cross-section, the design resistance of a composite member in fire generally has to be calculated by dividing the section into a number of elements. The temperature and the corresponding reduction factors for steel and concrete strengths in each element are determined and the design resistance of all the elements are added to calculate the resistance of the entire cross-section.

The simple models for composite members are more complex than the calculation for bare steel members. For this reason, most calculations for temperature rise and load bearing resistance of composite members are carried out using computer software. In addition, the resistances of many fire design solutions for composite members have been tabulated, as given in §4.2 of EN 1994-1-2.

Resistance of composite slabs

The simple models for fire design of composite slabs are specified in §4.3.1, §4.3.2 and §4.3.3 of EN 1994-1-2. As partition members, composite slabs are required to meet the criteria of insulation '*I*', integrity '*E*' and the load bearing resistance '*R*'. These must be demonstrated through fire tests, design calculations or both.

The calculation of sagging and hogging moment resistances of slabs are based on different assumptions and temperature distributions as given in D.2 and D.3 of EN 1994-1-2. The contribution of the steel decking is generally included in the calculation of the sagging moment resistance, but conservatively ignored in the calculation of the hogging moment resistance.

Resistance of composite beams

Composite beams comprise steel sections structurally attached through shear connectors to a concrete or composite slab, with or without concrete encasement. Generally, the voids created by the deck shape over the beam

flanges are filled, to ensure that the top flange of the steel beam remains relatively cool in fire.

Composite beams may be simply supported or continuous. Their design sagging and hogging moment resistances as well as their vertical and longitudinal shear resistances can be determined by using the simple models given in §4.3.1, §4.3.4 and Annex E of EN 1994-1-2.

Resistance of composite columns

The simple model for buckling resistance of composite columns is based on the assumption that the building is braced and the column has no horizontal displacement. The calculation rules are given in §4.3.5 and Annexes G and H of EN 1994-1-2.

5 TENSILE MEMBRANE ACTION

Full scale fire tests and real fire investigations in multi storey buildings have shown that when a floor develops tensile membrane action, it can achieve a higher fire resistance than calculations and tests on isolated members indicate. In order to take account of this beneficial behaviour of multi-storey buildings in fire, a new design model has been developed, which allows for a more economic solution to fire design. The model has been validated by tests.

5.1 Cardington fire test

The prescriptive approach and the simple calculation models described in this guide are well-established methods to carry out the fire design of a multi-storey building.

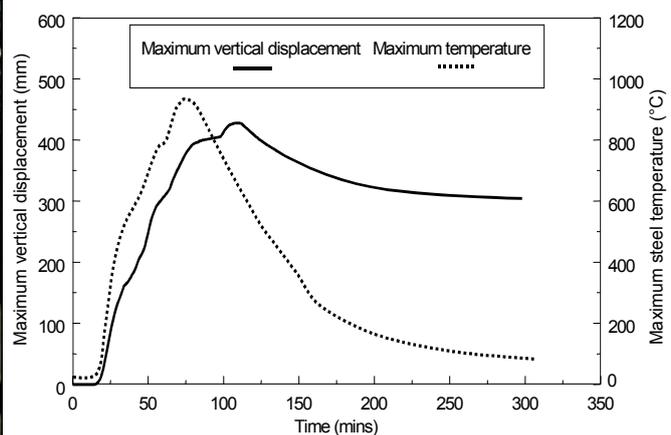
The application of such methods to an unprotected multi-storey steel building determines the fire resistance period as no more than 30 minutes. However, large-scale fire tests and real fire investigations over the years have demonstrated that some savings can be achieved in the fire design of the frame, and a new simple design method (BRE-BM) has been developed.

5.1.1 Full-scale fire tests

Following the investigations on real fire-damaged buildings in the UK at Broadgate and Basingstoke, fire tests were conducted in Cardington (UK) on a full-scale eight-storey steel-composite framed building with unprotected steel beams and trapezoidal composite slab, as shown in Figure 5.1(a).



(a) Full-scale building tested in Cardington



(b) Temperature and displacement of secondary beam

Figure 5.1 Cardington fire tests

Observations from the tests and the real fire investigations have consistently shown that the performance of a whole steel-framed building in fire is very different from that of its individual members. Under applied actions and a real fire, significant interactions between different types of structural members and important changes of their load-carrying mechanisms take place in real structures. The inherent fire performance of unprotected steel elements in

framed buildings, even in severe fires, is much better than standard fire tests would demonstrate.

As shown in Figure 5.1(b), instead of full collapse, as expected from the standard fire tests, a typical secondary beam in the Cardington fire test maintained its stability even when its temperature reached 954°C. Its vertical displacement reached 428 mm at its peak temperature of 954°C and recovered to a permanent displacement of 296 mm after the cooling phase. This indicates that there are large reserves of fire resistance in steel-framed buildings. The amount of fire protection being applied to steel elements may, in some cases, be excessive and unnecessary.

The main reason for the above large reserves of fire resistance of multi-storey framed buildings comes from the tensile membrane action of their steel-composite floors, as shown in Figure 5.2 and explained below.

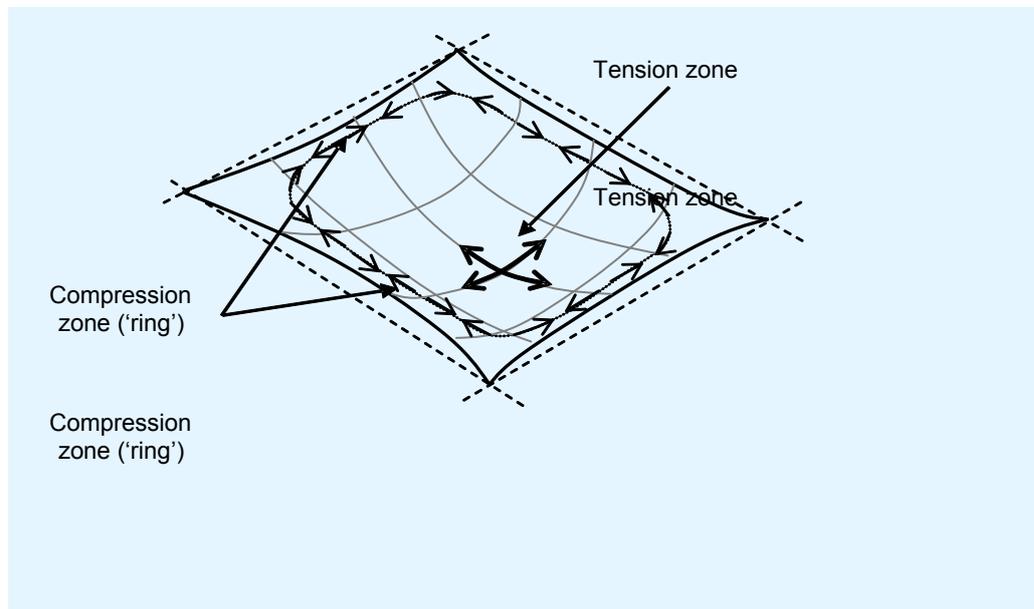


Figure 5.2 Membrane action in a horizontally restrained concrete slab

As described in the Section 4, the simple calculation models for fire design deal with each individual member. These simple models assume the floor slab to be a one-way spanning beam, resisting actions through bending and shear. Observations of the Cardington tests, however, show that as the steel beams lose their load bearing capacity, the composite slab utilises its full bending capacity in spanning between the adjacent, cooler members. As its displacement increases, the slab acts as a tensile membrane, carrying loads in tension, as shown in Figure 5.3.

If the slab is well supported against its vertical deflection along lines which divide it into reasonably square areas, for example, by the primary and secondary beams on the column grid-lines, then tensile membrane action can be generated as a load bearing mechanism. The slab is then forced into double-curvature and hangs as a tensile membrane in its middle regions. A peripheral compressive 'ring-beam' is generated either around its supported periphery or in its edge-beams, as shown in Figure 5.2.

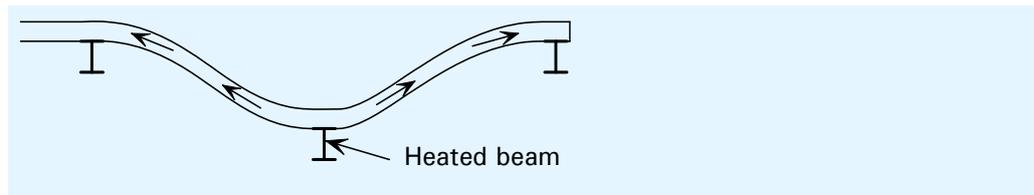


Figure 5.3 Slab bridging by tensile membrane action

This forms a self-equilibrating mechanism and supports the slab loading. If the temperature continues to increase, the structure may collapse due to either the failure of the edge support of the slab, or the fracture of the slab at its edges, or within its middle region.

5.1.2 Real fires compared with standard fire

In addition to the difference between the behaviour of isolated members and the members in a whole structure, real fires are also different from the standard fire that is used in the simple calculation models. A real fire in a building comprises three phases – initial growth, full development and post-peak decay. These phases are shown in Figure 5.4. The temperature increases most rapidly when all organic materials in the compartment combust spontaneously. This point in time is known as flashover point. The growth and decay phases of a real fire depend on the quantity and type of fuel available, as well as on the ventilation conditions of the compartment.

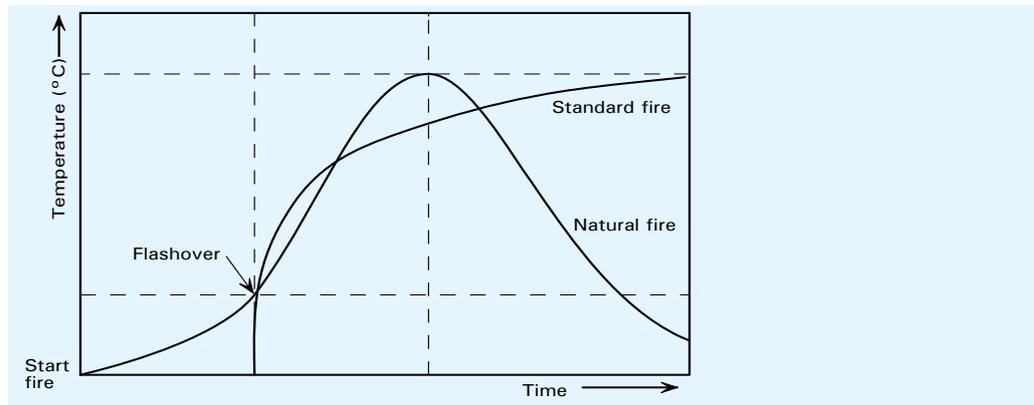


Figure 5.4 Real natural fire with the standard fire curve

The fire resistance time for a member, which is based on standard fire tests and simple calculation models, does not reflect the actual fire performance of the member as part of the whole building in a real fire. It does not indicate the actual time for which the member will survive in a building in fire.

5.2 FRACOF fire tests

To demonstrate the actual behaviour and load bearing resistance of composite floors under standard fire exposure, full scale fire tests were carried out in Maizienes-les-Metz (France) in 2008 as part of the FRACOF project, which was funded by ArcelorMittal and the Steel Alliance.

Figure 5.5 shows the composite floor, with an occupying area of more than 60 m² and the bottom surface exposed. All structural members of the composite

floor were designed in accordance with the relevant Eurocodes for normal design. The floor was uniformly loaded using sand bags. The four steel columns and four boundary beams were fire-protected, but the two intermediate secondary beams were left unprotected.

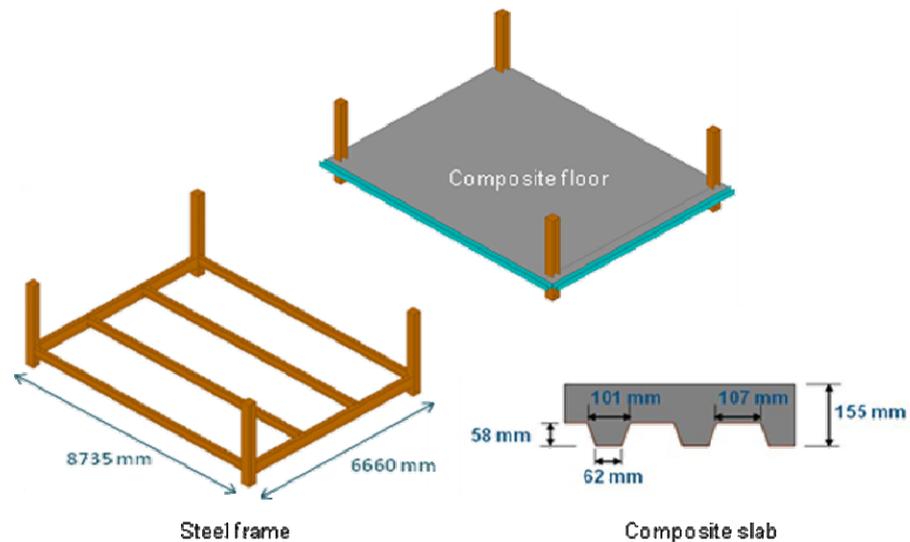
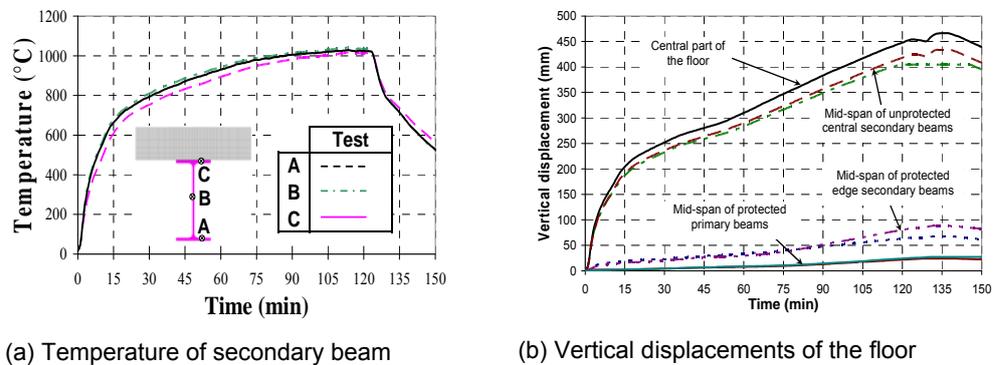


Figure 5.5 Details of full-scale fire tests on composite floor used in FRACOF



(a) Temperature of secondary beam (b) Vertical displacements of the floor

Figure 5.6 Results of full-scale fire tests on composite floors

As shown in Figure 5.6, under the standard fire and applied loads, even when the temperature of the unprotected beam rose to 1040°C and the central deflection increased to 448 mm, the composite floor construction maintained its structural stability for more than 120 minutes. This demonstrates that the fire resistance of composite floors with unprotected secondary beams is much better than the 15 minutes predicted by standard fire tests.

5.2.1 Design concepts

Based on the full-scale fire tests results and real fire observations, a new design concept has been developed to assess the fire performance of multi-storey steel framed buildings with composite floors.

5.2.2 Design models

This model uses a holistic, rather than an elemental approach. It calculates the residual strength of the composite floors using yield line and membrane action, as shown in Figure 5.7.

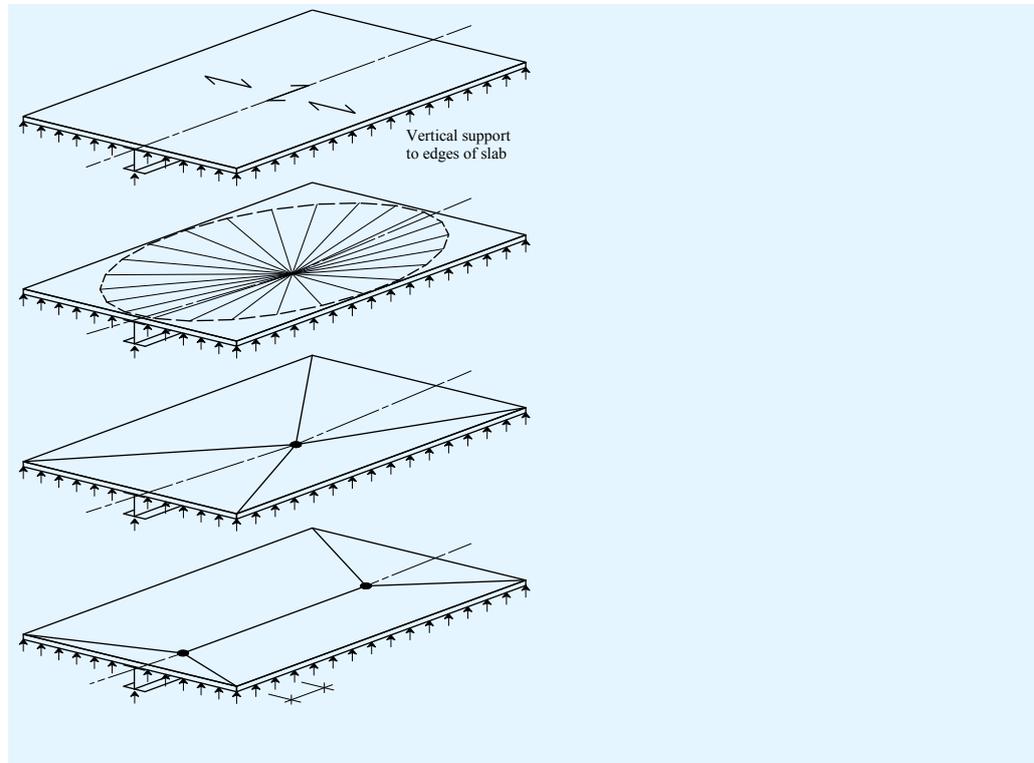


Figure 5.7 Yield line and membrane action of a composite floor in fire

This design model takes account of the interaction between the components of the composite floor, and concludes that some beams may remain unprotected whilst maintaining the required level of fire safety. Thus, this design model gives a more economic and site-specific fire safety solution, with a significant number of steel beams of the composite floor left unprotected.

This design model was calibrated against the fire tests, and is complemented by design tables and computer software. However, it demands expertise knowledge for the fire design of multi-storey buildings. Further information can be found in SCI publication P388^[6].

5.2.3 Applicability

The FRACOF design model may only be used to demonstrate the fire resistance of a partially protected structure where national building regulations permit the use of performance-based approach for fire design of buildings. In addition, this design model is only applicable to composite steel-framed buildings with the following limitations.

- The frame is braced and not sensitive to buckling in a sway mode
- Composite floor slabs comprise steel decking and reinforced concrete
- Floor beams are designed to act compositely with the floor slab

This model may not be used with precast concrete floor systems.

6 USE OF NATURAL FIRE EXPOSURE AND ADVANCED STRUCTURAL MODELLING

This Section introduces advanced calculation models that are used to demonstrate the actual behaviour of multi-storey buildings in fire and to explore more economic solutions for their fire design.

6.1 General

To date, both the datasheet approach and the simple calculation models have been widely used in the fire design of multi-storey buildings. They have been shown to be adequate for the minimum life-safety requirements. However, they do not account for the features of a real fire and the interaction between structural members in a fire. In addition, increasing innovation in design, construction and usage of modern buildings have made it difficult in some situations to satisfy the legislative fire regulations economically using the simple calculation models. Therefore, advanced calculation models have been developed for fire design of multi-storey buildings.

The advanced calculation models are based on the fundamental physical description of fire development, heat transfer and the structural response, which are simulated using numerical models. Therefore, they are able to give a more realistic and economic solution for the fire design of a structure. The advanced calculation models may be used to represent the behaviour of individual members, the whole structure or its sub-assemblies. They allow an appreciation of how buildings actually behave in a fire and give the option to design a more robust building.

Performance-based design involves establishing performance criteria, defining fire actions, modelling heat transfer and simulating the structural response to the subsequently derived elevated temperatures, as briefly described below.

6.2 Modelling fire severity

6.2.1 Two-zone model

Annex D of EN 1991-1-2 gives a two-zone model to determine the temperature caused by a localised fire. It assumes that combustion products within a compartment are accumulated in a layer beneath the ceiling. It divides the whole compartment with different fire conditions into a number of separated zones. Within each zone, the fire condition is assumed to be uniform, and the gas temperature is defined as a function of time by considering the conservation of mass and energy in the fire compartment. In each zone, the relevant differential equations are solved for the value of the temperature, using computer software.

6.2.2 Computational fluid dynamics (CFD)

Annex D of EN 1991-1-2 proposes a computational fluid dynamic model (CFD) for a localised fire. CFD involves fluid flow, heat transfer and

associated phenomena. It uses a series of partial differential equations to determine the temperature within the compartment due to a localised fire. The CFD model is very demanding and requires expertise to define the input parameters and to assess the appropriateness of the results.

6.2.3 Fully developed fire

The definition of the fully developed fire is given in Section 4.1.3, where the description of the standard fire and the parametric fire are also given. These fire curves are applicable to advanced models as well as to simple methods.

6.3 Modelling heat transfer

The temperature rise of a structural member in fire is due to the heat transfer from the gas in the compartment to the member. The temperature rise of a structural member in fire is predominantly controlled by radiation and convection.

Advanced models aim to provide a more realistic solution of temperature rise over the section of a structural member. Advanced models consider:

- Thermal actions given in EN 1991-1-2
- Non-uniform thermal exposure
- Non-uniform heat transfer
- Time-dependant boundary conditions
- Time and temperature-dependant material properties.

Advanced heat transfer models are developed based on 2D or 3D finite difference or finite element techniques. Therefore, they can only be performed by using commercial computer software packages.

6.4 Advanced structural models

Advanced models for structural responses to both the thermal actions and the applied actions are based on the fundamentals of structural engineering and carried out by means of finite element techniques.

The structure under consideration is generally divided into numerous small 2D or 3D elements with different initial and boundary conditions. Although finite element simulation clearly presents significant information to the designer, such an approach should not be taken without due care. There are potential problems associated with:

- The choice of software
- The options set within the software
- The way the structure is modelled
- The acceptance criteria used.

6.5 Validation/verification of advanced models

Since a number of advanced calculation models are available to model fire action as well as the thermal and structural response of buildings, both validation and verification must be performed to ensure that a sensible solution is obtained.

Validation is the demonstration of the suitability of the design model or approach for the intended purpose, which includes those for predictions of fire severity, heat transfer and structural response.

Verification is the assessment of whether or not the design model has produced valid results. It includes a careful check on the input data, on the consistence between the results obtained from the model and the results anticipated on qualitative analysis, and on the degrees of risk associated with possible errors. Advanced models should be verified against relevant test results and other calculation methods. They should be checked for compliance with normal engineering principles by the use of sensitivity studies.

In the context of validation and verification of models and results, the standard ISO 16730^[7] provides a framework for assessment, verification and validation of all types of calculation methods used as tools for fire safety engineering. This international standard does not address specific fire models, but is intended to be applicable to both simple methods and advanced methods.

6.6 Regulatory approval

The complexity of obtaining regulatory approval will vary from country to country. However, regulatory bodies may require the designer to present a fire design in a form that may be easily checked by a third party, with each design step clearly documented, including any assumptions and approximations that have been made. A checklist should be produced that covers the overall design approach, the fire model, the heat transfer and the structural response.

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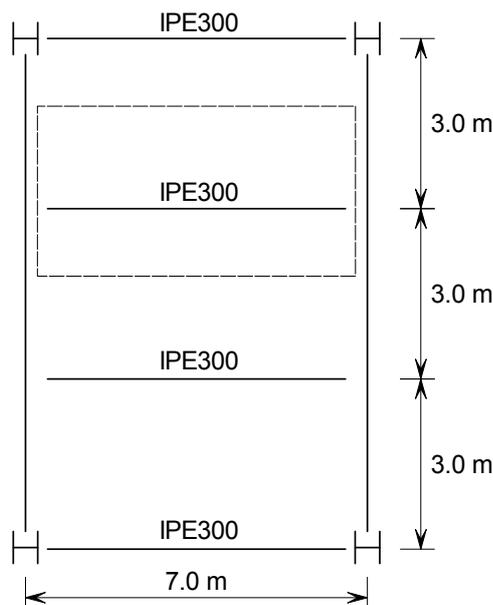
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Worked Example: Fire safety strategies and design approach of steel floor beam

This worked example illustrates the fire safety design of a simply-supported steel beam using fire strategies and design approaches described in this publication. Five different strategies are discussed to give the designer various alternatives of fire safety solutions. These include passive protection, alternative structural solutions, simple calculation models, tensile membrane action and FRACOF software. The prescriptive approach and simple calculation models were used to verify the fire resistance of the beam.

The figure below shows the floor system of a typical multi-storey office building. The beams are simply supported and they are required to have a 60 minute fire resistance under the standard fire exposure, i.e. R60. This example will focus on the design of one of the secondary beams. This beam has to resist and transmit the load from the dashed area and is constructed using an IPE300 in S275 steel.



Floor construction

1. Design Data

1.1. Steelwork properties

Yield strength: $f_y = 275 \text{ N/mm}^2$

Elastic modulus: $E = 210 \text{ kN/mm}^2$

Unit mass: $\rho_a = 7850 \text{ N/mm}^2$

Title	Worked Example – Fire safety strategies and design approach of steel floor beam	2 of 10
<p>1.2. Applied actions</p> <p>Permanent actions: $g_k = 3,5 \text{ kN/m}^2$ (self-weight of beam and slab) Variable actions: $q_k = 3,0 \text{ kN/m}^2$ (imposed loads on the floor)</p> <p>1.3. Partial safety factors</p> <p>Permanent actions: $\gamma_G = 1,35$ Variable actions: $\gamma_Q = 1,5$ Cross-section design at normal temperature: $\gamma_{M0} = 1,0$ Fire design: $\gamma_{M,fi} = 1,0$</p> <p>1.4. Floor system</p> <p>Beam spacing: $B = 3,0 \text{ m}$ Beam span: $L = 7,0 \text{ m}$</p> <p>2. Verifications for normal temperature</p> <p>2.1. Actions at normal temperature</p> <p>Characteristic value of actions: $w_k = 3,5 + 3,0 = 6,50 \text{ kN/m}^2$ Design value of actions: $w_d = 1,35 \times 3,5 + 1,5 \times 3,0 = 9,23 \text{ kN/m}^2$ Bending moment: $M_{Ed} = w_d B L^2 / 8 = 9,23 \times 3 \times 7^2 / 8 = 169,5 \text{ kNm}$ Shear force: $V_{Ed} = w_d B L / 2 = 9,23 \times 3 \times 7 / 2 = 96,9 \text{ kN}$</p> <p>2.2. Resistance at normal temperature</p> <p>2.2.1. Section classification</p> <p>Based on the geometric properties of an IPE300 and according to the rules given in section 5.5 of EN 1993-1-1, IPE300 is a Class 1 section.</p> <p>2.2.2. Resistance calculation</p> <p>The concrete floor is assumed to provide full lateral support to the compression flange. Therefore, the lateral–torsional buckling of the beam does not need to be considered. The cross-sectional checks are as follows:</p> <p>Bending moment resistance:</p> $M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{628 \times 10^3 \times 275}{1,0} \times 10^{-3} = 172,7 \text{ kNm} > 169,5 \text{ kNm} \quad \therefore \text{OK}$		
		EN 1993-1-1, §5.5
		EN 1993-1-1, §6

Title	Worked Example – Fire safety strategies and design approach of steel floor beam	3 of 10
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Shear resistance:

$$V_{c,Rd} = V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3} \gamma_{M0}} = \frac{2567 \times 275}{\sqrt{3} \times 1,0} = 407,7 \text{ kN} > V_{Ed} = 96,6 \text{ kN}$$

∴ OK

2.2.3. Deflection

$$\delta = \frac{5 w_k B L^4}{384 EI} = \frac{5}{384} \frac{6,5 \times 3000 \times 7000^4}{210000 \times 8356 \times 10^4} \times 10^{-3} = 34,7 \text{ mm} < \frac{L}{200} = 35 \text{ mm} \quad \therefore \text{OK}$$

Hence, the selected section IPE300 satisfies the requirements at normal temperature.

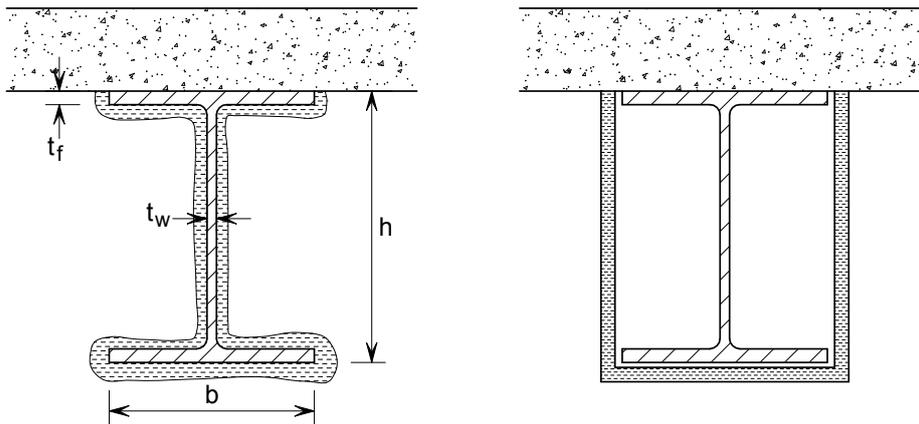
3. Fire safety strategies and design methods

The objective of fire safety design is to demonstrate that the beam satisfies the regulatory requirements of 60 minutes fire resistance, R60.

3.1. Passive fire protection

Passive protection includes board, sprayed and intumescent coatings

3.1.1. Section factor of unprotected / protected beam



$$A/V = \frac{1,5b + h - t_w}{(b - t_w)t_f + 0,5ht_w}$$

$$A/V = \frac{0,5b + h}{(b - t_w)t_f + 0,5ht_w}$$

3 side protection possibilities and A/V ratios.

(a) 3 sides profile protection; (b) 3 sides box protection

For either the unprotected section or 3 sides profile protection using sprayed or intumescent coatings, as shown in the above figure (a), the section factor is:

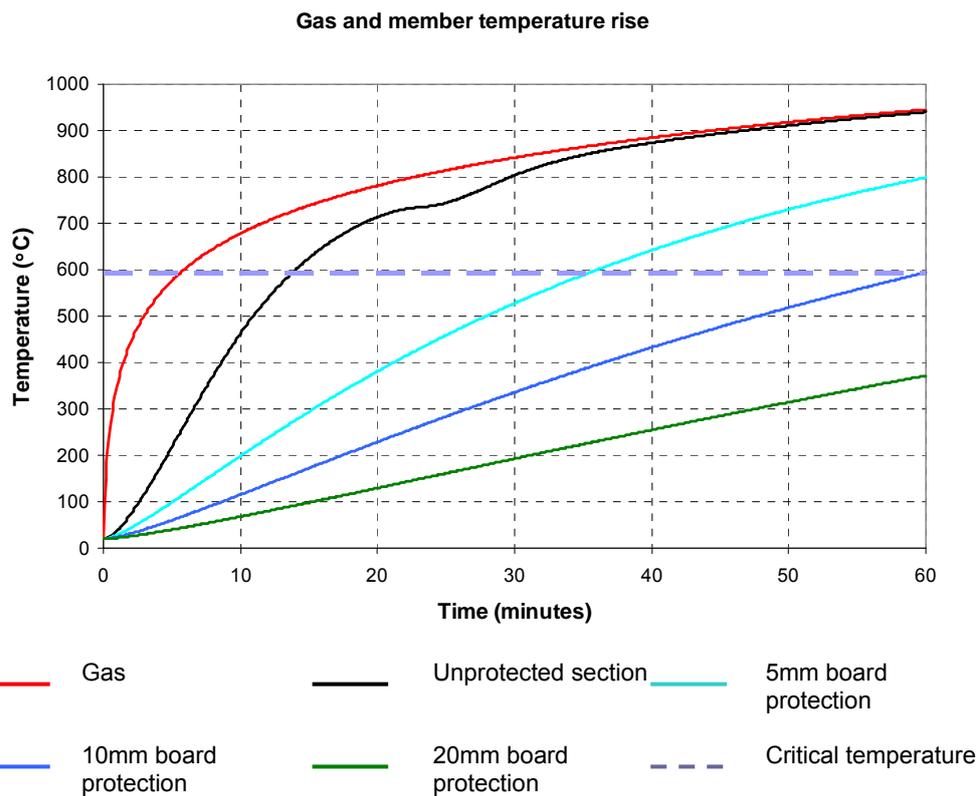
$$A_m/V = \frac{(1,5b + h - t_w)}{[(b - t_w)t_f + 0,5ht_w]} = \frac{(1,5 \times 150 + 300 - 7,1)}{[(150 - 7,1)10,7 + 0,5 \times 300 \times 7,1]} = 200 \text{ m}^{-1}$$

For 3 sides box protection using fire board, as shown in (b) above, the section factor is:

$$[A_m/V]_b = \frac{(0,5b + h)}{[(b - t_w)t_f + 0,5ht_w]} = \frac{(0,5 \times 150 + 300)}{[(150 - 7,1)10,7 + 0,5 \times 300 \times 7,1]} = 145 \text{ m}^{-1}$$

Title	Worked Example – Fire safety strategies and design approach of steel floor beam	4 of 10
	<p>3.1.2. Fire protection boards</p> <p>The section factor of the beam section</p> $A_m/V = 200 \text{ m}^{-1} < [A_m/V]_{\max} = 260 \text{ m}^{-1}$ <p>∴ 20mm thick fire board is adequate for R60.</p> <p>3.1.3. Sprayed coatings</p> <p>$A_m/V = 200 \text{ m}^{-1}$, 16mm thick sprayed coating is adequate for R60.</p> <p>3.1.4. Intumescent coatings</p> <p>$A_m/V = 200 \text{ m}^{-1}$, 0,69 mm thick intumescent coating is adequate for R60.</p> <p>3.2. Alternative structural solutions</p> <p>To avoid the costs of protection materials and reduce the cost of construction, it is preferred to use unprotected sections, as long as the R60 requirement is satisfied, by using alternative structural solutions. These include encasing the beam in concrete or using a composite slab. These options are discussed in the sections that follow.</p> <p>3.2.1. Partially encased beam</p> <p>Load level: $\eta_{fi} = 0,477 < 0,5$</p> <p>Section ratio: $h/b = 300/150 = 2,0 > 1,5$.</p> <p>Hence, for R60 fire resistance, a partially encased beam using concrete without reinforcement can satisfy the R60 requirement.</p> <p>Note that the extra weight of the concrete should be added to the permanent action.</p> <p>3.2.2. Integrated beams</p> <p>Integrated construction makes use of asymmetric steel sections containing the steel beam within the depth of the precast concrete slab. This solution may satisfy the R60 requirement without the need of fire protection.</p> <p>3.3. Simple calculation models</p> <p>To verify the fire resistance of the beam at its fire ultimate limit state, the simple model given in EN 1993-1-1 is used, following the flowchart given in Section 4.4 of this publication. This model is applied to unprotected sections and to sections protected with fire boards.</p> <p>The next section presents the calculation using the load bearing resistance approach. In order to obtain an optimised design, the worked example uses different protection thicknesses until the optimal is found.</p> <p>The critical calculation approach can only be used when the temperature across the cross-section is uniform and it is not shown in this example.</p>	<p>Table 3.1</p> <p>Table 3.2</p> <p>Table 3.3</p> <p>Table 3.7</p> <p>Section 3.3.2</p>

Title	Worked Example – Fire safety strategies and design approach of steel floor beam	5 of 10									
<p>3.3.1. Temperature of beam after 60 minutes fire exposure</p> <p>The temperature rise of the unprotected and protected sections are determined using the incremental calculation procedure and the following models given in EN 1993-1-2; §4.5.2.1.</p> <p>For unprotected section:</p> $\Delta\theta_{a,t} = k_{sh} \frac{1}{c_a \rho_a} \frac{A_m}{V} \dot{h}_{net} \Delta t$ <p>k_{sh} is the correction factor for the shadow effect. For an IPE section it is given as:</p> $k_{sh} = 0,9 \frac{A_m / V}{[A_m / V]_b} = 0,9 \frac{145}{200} = 0,652$ <p>For protected sections (assuming only protection boards are used):</p> $\Delta\theta_{a,t} = \frac{\lambda_p}{d_p c_a \rho_a} \frac{A_p}{V} \left(\frac{1}{1 + \phi/3} \right) (\theta_{g,t} - \theta_{a,t}) \Delta t - (e^{\phi/10} - 1) \Delta\theta_{g,t}$ $\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p \frac{A_p}{V}$ <p>The net heat flux is: $\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r}$</p> <p>Convection:</p> $\dot{h}_{net,c} = \alpha_c (\theta_g - \theta_m) = 25 \times (\theta_g - \theta_{at})$ <p>Radiation:</p> $\begin{aligned} \dot{h}_{net,r} &= \sigma \varepsilon_m \varepsilon_f \Phi [(\theta_t + 273)^4 - (\theta_m + 273)^4] \\ &= 5,67 \times 10^{-8} \times 0,7 \times 1,0 \times 1,0 \times [(\theta_g + 273)^4 - (\theta_{at} + 273)^4] \end{aligned}$ <p>The curve of the nominal standard fire is given by the following expression:</p> $\theta_g = 20 + 345 \log_{10} (8t + 1)$ <p>The equations above are evaluated based on the following thermal properties of steel and board protection materials:</p> <table data-bbox="183 1691 1117 1848"> <tr> <td>$c_a = 600 \text{ J/kg}^\circ\text{C}$</td> <td>$c_p = 1200 \text{ J/kg}^\circ\text{C}$</td> <td>$d_{p,5} = 0,005 \text{ m}$</td> </tr> <tr> <td>$\rho_a = 7850 \text{ kg/m}^3$</td> <td>$\rho_p = 300 \text{ kg/m}^3$</td> <td>$d_{p,10} = 0,01 \text{ m}$</td> </tr> <tr> <td>$\Delta t = 5 \text{ sec}$</td> <td>$\lambda_p = 0,1 \text{ W/m}^\circ\text{C}$</td> <td>$d_{p,20} = 0,02 \text{ m}$</td> </tr> </table> <p>With these data and equations, the graph in the Figure below can be drawn. This shows the temperature change in the air, the unprotected section and the protected section with 3 protection thicknesses: 5 mm, 10 mm and 20 mm.</p>			$c_a = 600 \text{ J/kg}^\circ\text{C}$	$c_p = 1200 \text{ J/kg}^\circ\text{C}$	$d_{p,5} = 0,005 \text{ m}$	$\rho_a = 7850 \text{ kg/m}^3$	$\rho_p = 300 \text{ kg/m}^3$	$d_{p,10} = 0,01 \text{ m}$	$\Delta t = 5 \text{ sec}$	$\lambda_p = 0,1 \text{ W/m}^\circ\text{C}$	$d_{p,20} = 0,02 \text{ m}$
$c_a = 600 \text{ J/kg}^\circ\text{C}$	$c_p = 1200 \text{ J/kg}^\circ\text{C}$	$d_{p,5} = 0,005 \text{ m}$									
$\rho_a = 7850 \text{ kg/m}^3$	$\rho_p = 300 \text{ kg/m}^3$	$d_{p,10} = 0,01 \text{ m}$									
$\Delta t = 5 \text{ sec}$	$\lambda_p = 0,1 \text{ W/m}^\circ\text{C}$	$d_{p,20} = 0,02 \text{ m}$									



Temperature rise of gas and beam due to exposure to natural fire

3.3.2. Load resistance

Design effects for fire design

In accordance with the simplified rule in EN 1991-1-2, the design effect of actions for fire situation, can be determined by reducing those for normal design by a factor η_{fi} .

For office buildings, the combination factor for accidental action of fire with the variable action $\psi_{fi} = \psi_{2,1} = 0,3$, and the reduction factor:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_k}{\gamma_G G_k + \gamma_Q Q_k} = \frac{3,5 + 0,3 \times 3,0}{1,35 \times 3,5 + 1,50 \times 3,0} = 0,477$$

Therefore, the design effect of actions for fire:

$$\text{Bending moment: } M_{fi,Ed} = \eta_{fi} M_{Ed} = 0,477 \times 169,5 = 80,9 \text{ kNm}$$

$$\text{Shear force: } V_{fi,Ed} = \eta_{fi} V_{Ed} = 0,477 \times 96,9 = 46,2 \text{ kN}$$

Section classification

For fire design, the value of ε is equal to 85% of that for normal design

$$\varepsilon = 0,85 \sqrt{\frac{235}{f_y}} = 0,85 \times \sqrt{\frac{235}{275}} = 0,72$$

EN 1993-1-2;
§4.2.2

Title	Worked Example – Fire safety strategies and design approach of steel floor beam	7 of 10
<p>The ratios for local buckling of the section elements are:</p> <p>Flange: $c_f/t_f = 5,28 < [c_f/t_f]_{lim} = 9\varepsilon = 7,07$</p> <p>Web: $c_w/t_w = 35,01 < [c_w/t_w]_{lim} = 72\varepsilon = 56,57$</p> <p>The section is class 1.</p> <p>Resistance of beam after 60 minutes fire exposure</p> <p>The moment resistance of a beam with non-uniform temperature along the depth of its cross-section can be calculated as:</p> $M_{fi,Rd} = M_{fi,t,Rd} = k_{y,\theta} W_{pl,y} f_y / (\kappa_1 \kappa_2)$ <p>Where</p> <p>$k_{y,\theta}$ is the reduction factor for effective yield strength</p> <p>κ_1, κ_2 are adaptation factors for non-uniform temperature across the cross-section and along the beam</p> <p>Unprotected beam</p> <p>The temperature of the unprotected beam after 60 minutes fire exposure as obtained from the figure on Sheet 6, is:</p> $\theta_{at} = 935^\circ\text{C}$ <p>The reduction factor for effective yield strength can be obtained for:</p> $\theta_a = 900^\circ\text{C} \quad k_{y,\theta} = 0,060$ $\theta_a = 1000^\circ\text{C} \quad k_{y,\theta} = 0,040$ <p>By interpolating for $\theta_a = 935^\circ\text{C}$, we obtain:</p> $k_{y,\theta} = 0,052$ <p>For an unprotected beam exposed on three sides, with a composite or concrete slab on side four:</p> $\kappa_1 = 0,70$ <p>for any case where the supports of the beam are not statically indeterminate:</p> $\kappa_2 = 0,85$ <p>Therefore the moment resistance of the beam is:</p> $M_{fi,Rd} = 0,052 \times 628 \times 10^3 \times 275 / (0,7 \times 1,0) \times 10^{-6} = 12,83 \text{ kNm} < 80,9 \text{ kNm}$ <p>Therefore the unprotected section is not safe.</p> <p>10 mm board protection</p> <p>Hence some protection is required for the beam. Following from the findings of the previous simplified calculation based on the temperature, the first option to be explored is the 10mm fire board protection. This solution has already been found to be safe.</p> <p>From the above Figure the temperature of this solution after 60 minutes of fire exposure is:</p> $\theta_{at} = 594^\circ\text{C}$		<p>EN 1993-1-2; Table 3.1</p> <p>EN 1993-1-2; §4.2.3.3</p> <p>∴ FAIL</p>

Title	Worked Example – Fire safety strategies and design approach of steel floor beam	8 of 10
<p>The reduction factor for effective yield strength can be obtained for:</p> $\theta_a = 500^\circ\text{C} \quad k_{y,\theta} = 0,780$ $\theta_a = 600^\circ\text{C} \quad k_{y,\theta} = 0,470$ <p>By interpolating for $\theta_a = 594^\circ\text{C}$,</p> $k_{y,\theta} = 0,46$ <p>The κ_1 and κ_2 factors are as previously obtained.</p> <p>The moment resistance of the fully restrained beam with non-uniform temperature along the depth of its cross-section:</p> $M_{fi,Rd} = k_{y,\theta} \frac{W_{pl,y} f_y}{\kappa_1 \kappa_2} = 0,46 \frac{628 \times 10^3 \times 275}{0,7 \times 1,0} \times 10^{-6} = 113 \text{ kNm} > 80,9 \text{ kNm}$ <p>The shear resistance can be obtained as follows:</p> $V_{fi,Rd} = k_{y,\theta} \frac{A_v f_y}{\sqrt{3} \gamma_{M0}} = 0,46 \frac{2567 \times 275}{\sqrt{3} \times 1,0} = 187,5 \text{ kN} > 96,6 \text{ kN}$ <p>The 3 side box protected section with 10 mm fire boards, is safe.</p> <p>3.4. Make use of tensile membrane action</p> <p>Full size fire tests and real fire investigations have resulted in the conclusion that the use of composite floors with steel decking leads to a much higher fire resistance than predicted by the simple calculations adopted by the Eurocode and shown in this example.</p> <p>As long as the slab is well supported against its vertical deflection along lines which divide it into reasonably square areas, it has been shown that tensile membrane action can be developed as a load bearing mechanism, consequently providing a reserve load carrying resistance that simplified models do not account for.</p> <p>As a result, it could be assumed that the beam in this example could be left unprotected, while maintaining the level of safety required by EC3. This construction system has been shown to provide a higher fire resistance for secondary beams but not for primary beams, and therefore care must be taken when adopting this solution.</p> <p>3.5. FRACOF</p> <p>The FRACOF software can be used to determine whether or not unprotected secondary steel beams are adequate when composite construction is used.</p>		<p>∴ OK</p> <p>∴ OK</p> <p>∴ OK</p> <p>Section 8.</p>

For illustration purposes, the IPE300 S275 beam used in this example is used as a composite beam, with the following assumptions for use in Fracof:

- IPE550 S275 primary beams
- Cofrastra 40 deck
- 130 mm deep slab, using C25/30 concrete
- ST 40 C mesh, 500 N/mm²
- All beams assumed to have 100% shear connection.

The design value of the actions for normal temperature is 9,23 kN/m². The reduction factor for fire has been calculated as 0,477, which gives a fire action of 4,4 kN/m².

For these input details, Fracof determines that it is adequate to use unprotected IPE300 secondary beams for R60 fire resistance. The details are summarised in the table below and graphically shown in the figure below.

Time	Beam	Mesh	Slab top	Slab bottom	Beam capacity	Maximum allowable deflection	Slab yield	Enhancement	Slab capacity	Total capacity	Unity factor
mins	°C	°C	°C	°C	kN/m ²	mm	kN/m ²		kN/m ²	kN/m ²	
0	20	20	20	20	20.25	190	2.66	1.97	5.25	25.50	0.32
5	247	21	20	173	20.25	233	2.66	2.20	5.85	26.11	0.32
10	534	24	20	387	13.96	292	2.66	2.52	6.70	20.66	0.40
15	692	31	21	534	5.31	333	2.66	2.74	7.28	12.59	0.66
20	761	39	24	631	3.36	359	2.66	2.88	7.66	11.01	0.75
25	802	48	28	697	2.35	377	2.66	2.97	7.90	10.25	0.80
30	832	60	33	746	2.02	389	2.66	3.03	8.08	10.10	0.82
35	857	70	39	784	1.76	398	2.66	3.08	8.20	9.96	0.83
40	879	86	47	815	1.53	404	2.66	3.12	8.30	9.82	0.84
45	897	91	55	841	1.33	409	2.66	3.14	8.37	9.70	0.85
50	914	112	63	864	1.24	413	2.66	3.17	8.43	9.67	0.85
55	928	127	71	883	1.18	416	2.66	3.18	8.47	9.65	0.86
60	942	138	78	901	1.12	419	2.66	3.20	8.51	9.63	0.86

Maximum unity factor: 0.86 **Floor slab adequate**

According to the data shown in the table, after 20 minutes fire exposure, the beam capacity is 3,36 kN/m² < 4,4 kN/m² fire action.

Considering the slab capacity, the total resistance remains over 9 kN/m² after 60 minutes fire exposure, which means that the beam doesn't need to be protected.

