

International developments in design for structural fire safety

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Abstract

This paper describes international developments in the design of structures to resist severe fires. Interest in this topic has been rapidly growing since the collapse of the World Trade Center towers on September 11, 2001.

Design for fire safety must be based on clearly established objectives. Structural design is only part of the overall provision of fire safety. Non-structural fire safety must be included as part of a comprehensive fire safety strategy. Structural design for fire conditions must consider many different fire scenarios, with realistic assessment of possible fire conditions.

To predict the structural behaviour of a building during fire, it is necessary to know the expected severity of the fire and the response of the structural materials. This paper reviews the main factors affecting fire development and the performance of steel, concrete and timber structures in severe fires, from a structural engineering perspective. Structural analysis and design of buildings in fire conditions is more complex than for normal temperature conditions.

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1. Introduction

The purpose of this paper is to give an overview of recent developments in the art and science of structural design for fire safety. The paper first summarises the legislative environment, then discusses the main steps in structural fire design. The main questions are:

1. What are the objectives for fire safety?
2. What are the loads on the structure at the time of the fire?
3. What is the fire severity?
How hot is the fire?
4. What is the fire resistance?
How hot is the material?
Can the structure carry the loads?

This paper is based on the author's book *Structural Design For Fire Safety*, published by John Wiley & Sons, UK, 2001 (Buchanan 2001). All figures are from this book unless otherwise noted.

2. Objectives for fire safety

2.1 Design objectives

Building codes are different around the world, but they all have similar objectives; these are to protect life and property from the effects of fire. Before starting any design for fire safety, it is essential to carefully establish the objectives, which will include much more than structural fire safety. A useful

starting point is the NFPA Firesafety Concepts Tree presented in Figure 1 (adapted from NFPA 1997). The bottom right hand corner of the figure is the area relating to structural design and construction, as explained by Buchanan (2001).

Fire safety systems are often described as *active* or *passive* fire safety systems. Active fire protection systems are those which have to operate on cue if there is a fire, including detection, alarm and automatic suppression systems, also systems for smoke control and escape route management. Passive fire protection systems are those which are installed in the building, permanently available to offer protection in the event of a fire. As an international trend, emphasis is shifting away from property protection and more towards life safety of occupants and fire fighters. This is resulting in more investment in active rather than passive fire protection systems. In many cases the most effective form of fire protection is an automatic sprinkler system, which would eliminate the need for structural fire protection if 100% reliability could be guaranteed. However, even the most carefully installed sprinkler system has some probability of not operating as intended because of malfunction, overloading, shutdown for maintenance or a serious event such as an earthquake or terrorist attack.

The major component of passive fire protection is *fire resistance*. Structural elements can be provided with fire resistance for either *containment* (controlling the spread of fire) or *preventing*

structural collapse, or both, depending on the functional requirements for the particular building. Non-structural elements may also be provide with fire resistance, but only for containment.

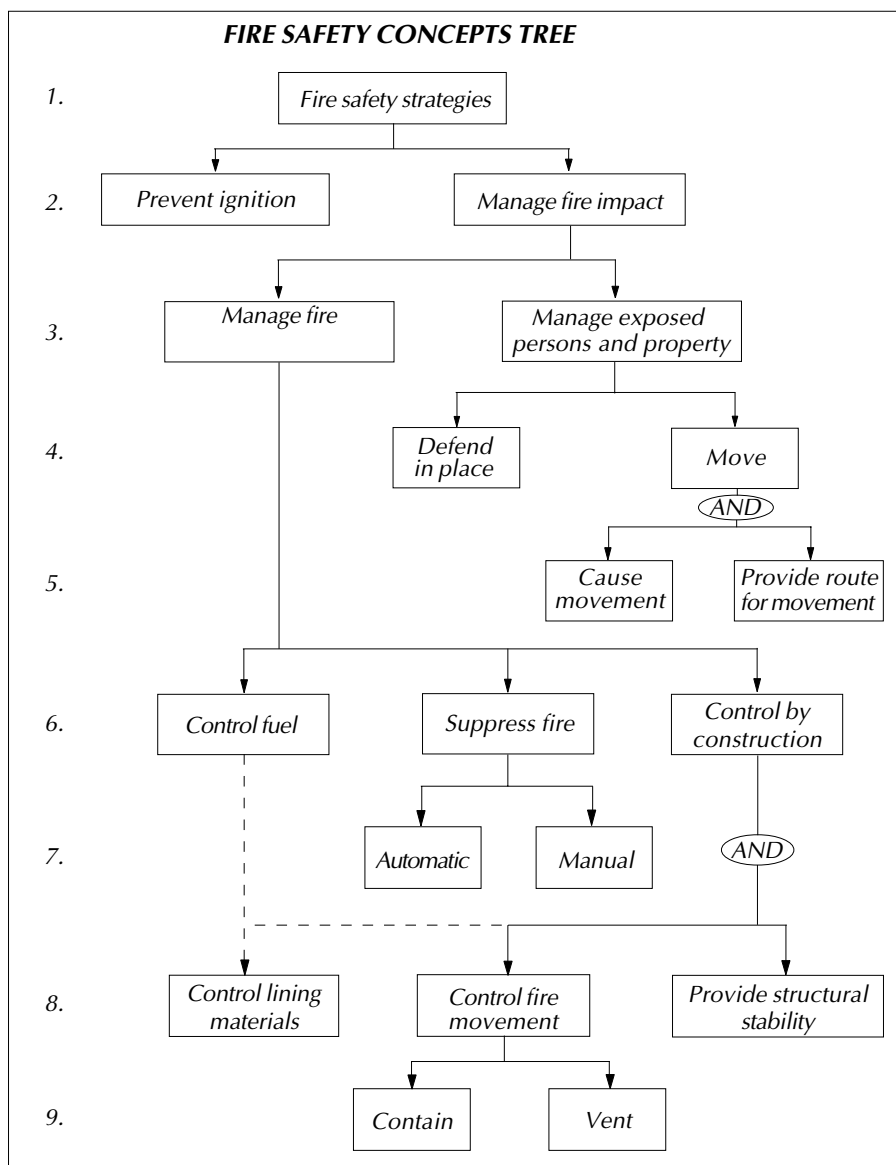


Figure 1: NFPA Firesafety concepts tree.

2.2 Legislative Environment

Current international standards range from very simple prescriptive documents to sophisticated codes which allow advanced methods of analysis under a wide range of realistic conditions, such as the Structural Eurocodes. The legislative environment is very different in different countries. Many countries are moving at various speeds to adopt performance based codes, or to move from a prescriptive code environment to a more performance based environment.

2.2.1 Performance based codes

Until recently, most structural design for fire safety has been based on *prescriptive* building codes, with little or no opportunity for designers to take a rational engineering approach to the provision of fire safety. Many countries have recently adopted *performance based*

building codes which may give designers freedom to adopt any fire safety strategy they wish, provided that adequate safety can be demonstrated. In general terms, a prescriptive code states “how a building is to be constructed” whereas a performance based code states “how a building is to perform”.

In the development of new codes, many countries have adopted the multi-level code format shown in Figure 2. At the highest levels, there is legislation specifying the overall goals, functional objectives and required performance which must be achieved in all buildings. At a lower level, there is a selection of alternative means of achieving those goals. The three most common options are either to comply with a prescriptive *Acceptable Solution*, to comply with an *Approved calculation method*, or to carry out a *Performance based alternative design* from first principles, using all the information available.

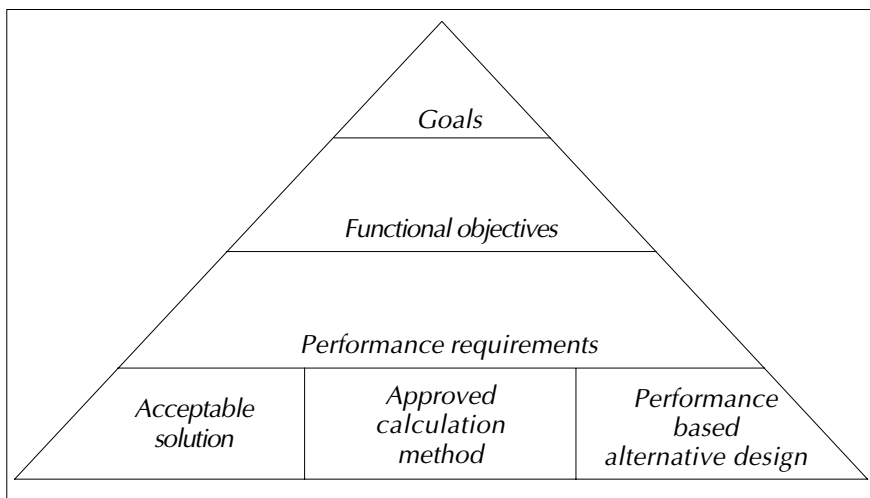


Figure 2:. Hierarchy of performance based design.

Approved calculation methods are the most common form of compliance for structural engineering, but for fire engineering these methods are not yet fully developed, so not often used. In design for fire safety, compliance with performance based codes is usually achieved by using an *Acceptable Solution* (often called a “deemed-to-satisfy” solution) which may be similar to a

previous prescriptive code. *Performance based alternative design* is used for special cases where variations from the Acceptable Solution will allow money to be saved or safety to be improved by innovative design. The alternative design will be based on specific fire engineering design principles, often using *scenario analysis* as shown in Figure 3 (Buchanan 2000).

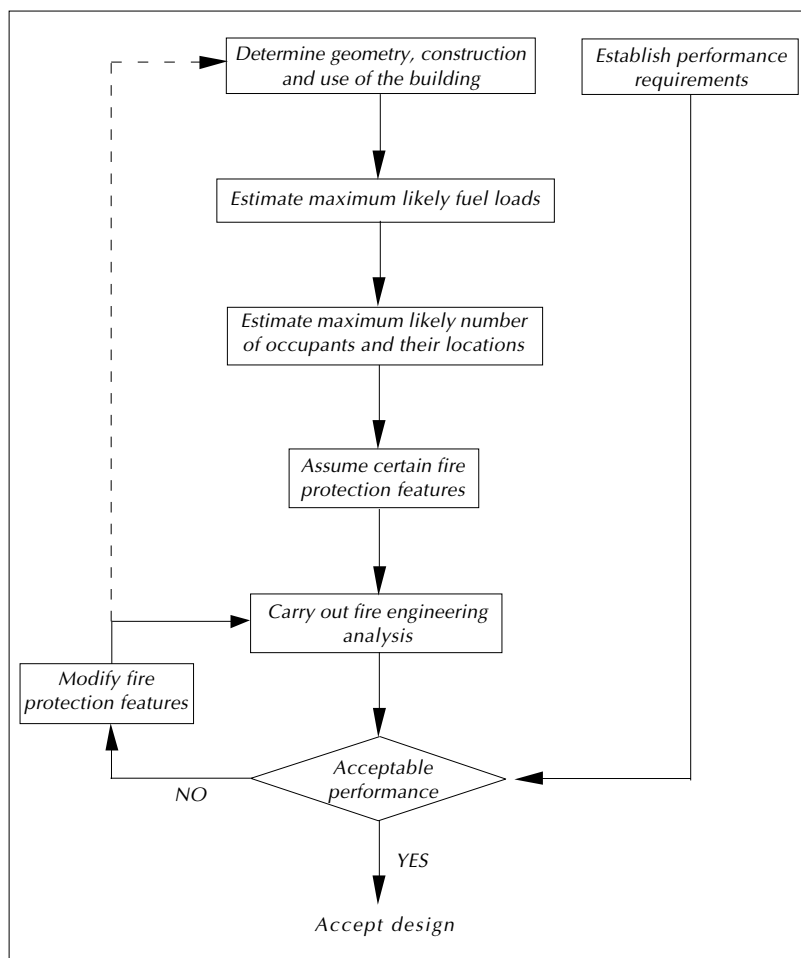


Figure 3: Scenario analysis.

2.3 International codes

The code environment in New Zealand (described by Buchanan, 1994, 2000), is similar to that in England, Australia and some Scandinavian countries. Moves towards performance based codes are being taken in the United States (IFCI 2000).

European countries have been working for more than twenty five years on a new co-ordinated set of structural design standards known as the Structural Eurocodes. These are comprehensive documents which bring together diverse European views on all aspects of structural design, for all main structural materials. The Eurocodes recognize the need for member countries to set national safety standards which may vary, so each country's national standard will comprise the full text of the Eurocode, with local modifications in a supporting document.

3. Structural design for fire conditions

3.1 Design process

Structural design for fire conditions is conceptually similar to structural design for normal temperature conditions. Before making any design it is essential to establish clear objectives and design criteria. The design can be carried out

using either working stress format or ultimate strength (LRFD) format. The main differences of fire design compared with normal temperature design are that, at the time of a fire:

- the applied loads are less;
- internal forces may be induced by thermal expansion;
- strengths of materials are reduced by elevated temperatures;
- cross section areas may be reduced by charring or spalling;
- smaller safety factors can be used, because of the low likelihood of the event;
- deflections are not important (unless they affect strength);
- different failure mechanisms need to be considered.

The above factors may be different for different materials.

3.2 Design equation

The fundamental step in designing structures under ambient conditions is to satisfy a design equation to ensure that

structural capacity $\geq$ applied loads

When designing structures for fire safety, the design equation is to verify that the fire resistance of the structure (or each part of the structure) is greater than the severity of the fire to which the structure is exposed. This verification requires that:

fire resistance $\geq$ fire severity

collapse or other failure as a result of the fire.

where

fire resistance is a measure of the ability of the structure to resist collapse, fire spread or other failure during exposure to a fire of specified severity, and

fire severity is a measure of the destructive impact of a fire, or a measure of the forces or temperatures which could cause

As shown in Table 1, there are three alternative methods of comparing fire severity with fire resistance. The verification may be in the *time* domain, the *temperature* domain or the *strength* domain, using different units, which can be confusing if not understood clearly.

Table 1: Three alternative methods of comparing fire severity with fire resistance.

Domain	Units	FIRE RESISTANCE	$\geq$ FIRE SEVERITY
<i>Time</i>	minutes or hours	Time to failure	$\geq$ Fire duration calculated, or specified by code
<i>Temperature</i>	$^{\circ}\text{C}$	Temperature to cause failure	$\geq$ Maximum temperature reached during the fire
<i>Strength</i>	kN or kN.m	Load capacity at elevated temperature	$\geq$ Applied load during the fire

3.3 Loads for structural fire design

The most likely loads at the time of a fire are much lower than the maximum design loads specified for normal temperature conditions. For this reason, different design loads and load combinations are used. Most codes refer to an “arbitrary point-in-time load” to be used for the fire design condition. Loads and load combinations are given, for example, by ASCE (1995) and the Eurocode (EC1 2002). For example, in the New Zealand code (SNZ 1992) the

load combination of dead load G and live load Q for normal temperature design is

$$U = 1.2 G + 1.6 Q \quad (1)$$

The combination for design in fire conditions is

$$U_f = G + 0.4 Q \quad (2)$$

The resulting difference between these two load combinations provides a safety margin at the time of the fire. The larger the safety margin, the larger the fire resistance.

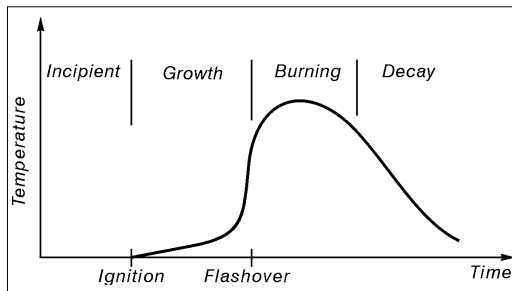


Figure 4: Typical stages of fire development.

4. Fire severity - How hot is the fire?

4.1 Fire behaviour

Typical stages of fire development are shown in Figure 4. The pre-flashover period is the time when occupants can detect and escape the fire and when automatic detection and suppression systems will operate.

If a fire in a typical room is allowed to grow without intervention, assuming sufficient fuel and ventilation, temperatures will increase as the radiant heat flux to all exposed objects increases. At a critical level of heat flux, all exposed combustible items in the room will ignite, with a rapid increase in heat release rate and temperature as shown in Figure 4. This transition is *flashover*, after which the burning period of the fire is referred to as a “post flashover fire”, “fully developed fire” or “full room involvement”. The fire severity in the post-flashover period determines the exposure for structural and containment assemblies. The rate of burning is mainly governed by the ventilation, and the duration by the amount of fuel.

4.2 Fire severity for design

Before starting the design it is essential to know the severity of the design fire, or range of design fires. The fire severity depends on the legislative environment and on the design philosophy. It is usually assumed that there is no automatic fire suppression or intervention by fire fighters. There are three alternatives:

1. in a *prescriptive* code, the design fire severity is usually strictly prescribed by the code, as a period of exposure to the standard test fire;
2. in a *performance based* code, the design fire can be a complete burnout of the fire compartment, or in some cases a shorter time of fire exposure to allow for escape, rescue, or fire-fighting operations;
3. the *equivalent time* of a complete burnout is the time of exposure to the standard test fire that would result in an similar impact on the structure.

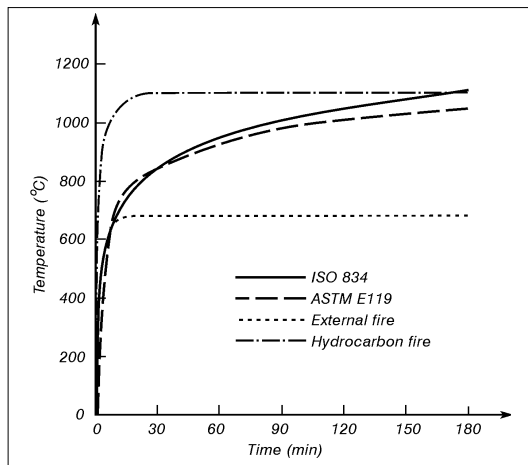


Figure 5: Standard fire curves.

4.3 Standard fire exposure

Most countries rely on full size fire resistance tests to assess the fire performance of building materials and structural elements. The time temperature curve used in fire resistance tests is called the *standard fire*. Full size tests are preferred over small scale tests because they allow the method of construction to be assessed, including the effects of thermal expansion, shrinkage, local damage and deformation under load.

The most widely used standard test specifications are ASTM E119 (ASTM 1995) and ISO 834 (ISO 1975) which have very similar time temperature curves as shown in Figure 5. All other international fire resistance test standards specify similar time temperature curves. Non of these standard fire curves represent actual real fires, but they are very useful for comparing the fire resistance of alternative assemblies.

In the ISO 834 specification the temperature T (°C) is defined by

$$T = 345 \log_{10} (8 t + 1) + T_0 \quad (3)$$

where t is the time (minutes) and T_0 is the ambient temperature (°C).

Figure 5 also shows two alternative design fires from Eurocode 1. The upper curve is the *hydrocarbon fire curve*, intended for use where a structural member is engulfed in flames from a large pool fire, given by

$$T = 1080 (1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) + T_0 \quad (4)$$

The lower curve is the *external fire curve* intended for design of structural members located outside a burning compartment. Exterior structural members will be exposed to lower temperatures than members inside a compartment. The fire temperature is given by

$$T = 660 (1 - 0.687e^{-0.32t} - 0.313e^{-3.8t}) + T_0 \quad (5)$$

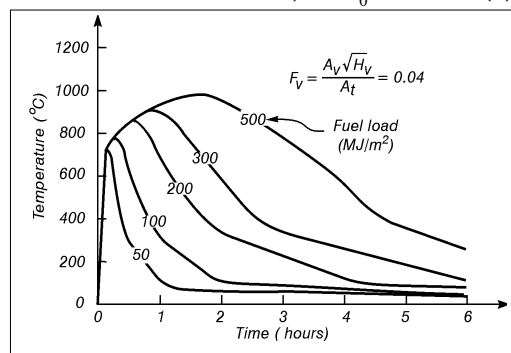


Figure 6: Time temperature curves for given ventilation and different fuel loads.

4.4 Realistic fire exposure

The most widely used time temperature curves for post-flashover fire exposure are the “Swedish” fire curves shown in Figure 6 (Magnusson and Thelandersson 1970). They are derived from heat balance calculations for the burning of ventilation controlled fires. A group of curves is provided for different ventilation factors, with fuel load as marked. Note that the units of fuel load MJ per m² of total internal surface area (not MJ per m² of floor area which is more often used in design calculations). Computer programs are available for calculating temperatures in post-flashover room fires.

The Eurocode “parametric” fires (EC1, 2002) allow a time- temperature relationship to be produced for any combination of fuel load, ventilation openings and wall lining materials, to give an approximation to the burning period of the Swedish curves shown above. The Eurocode equation for temperature T (°C) is

$$T = 1325 (1 - 0.324e^{-0.2 t^*} - 0.204e^{-1.7 t^*} - 0.472e^{-19 t^*}) \quad (6)$$

where t^* is a fictitious time (hours)
given by $t^* = \Gamma t$
 t is the time (hours) and

$$\Gamma = \frac{(F_v / 0.04)^2}{(b / 1160)^2} \quad (7)$$

where b is $\sqrt{\text{thermal inertia}} = \sqrt{(k\rho c_p)} \quad (Ws^{0.5}/m^2K)$
 F_v is the ventilation factor
 $F_v = A_v \sqrt{H_v} / A_t \quad (\sqrt{m})$
 A_v is the area of the window opening (m)
 A_t is the total internal surface area of the room (m²)
 H_v is the height of the window opening (m)

Equation (6) is a good approximation to the ISO 834 standard fire curve for temperatures up to about 1300°C, so the Eurocode parametric fire curve is close to the ISO 834 curve for the special case where $\Gamma = 1$.

The duration of the burning period t_d (hours) in the Eurocode has been

increased by 50% in the latest (2002) version, simplified as:

$$t_d = 0.0002 e_t / F_v = 0.0002 E / (A_v \ddot{O}H_v) \quad (8)$$

where e_t is the fuel load (MJ / m² total surface area) or E is the total energy content of the fuel (MJ).

The Eurocode uses a basic decay rate of 625°C per hour for fires with a burning period less than half an hour, decreasing to 250°C per hour for fires with a burning period greater than two hours, all modified by the G factor. Recent research using the COMPF2 program and many test fire results has shown that the temperatures in the Eurocode formula are often too low and the rate of decay is often inappropriate, leading to proposals for empirical modifications (Feasey and Buchanan 2002).

4.5 Time equivalence

The concept of *equivalent fire severity* is used to relate the severity of an expected real fire to the standard test fire. This is important when designers have information about fuel load and ventilation, but want to specify materials with published fire resistance ratings from standard fire tests. There are several methods of comparing real fires to the standard test fire, the most common being the time equivalence formula given in Eurocode 1 (EC1 2002). The equivalent

time is an estimate of the time of exposure to the standard fire that would result in the temperature of a protected steel beam reaching the maximum temperature obtained in the real fire, as shown in Figure 7.

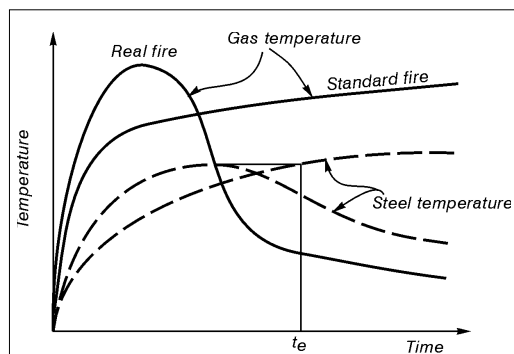


Figure 7: Time equivalence concept for a steel beam.

The Eurocode formula for equivalent time t_e takes the form:

$$t_e = k w e \quad (9)$$

where k is a factor representing the thermal properties of the construction, w is a ventilation factor and e is the fuel load in MJ per square metre of floor area.

Although developed for steel beams as described above, the time equivalence formula is widely used for structural and non-structural elements of various materials, as a rough design tool.

5. Fire resistance - How hot is the material? Can the structure carry the loads?

Fire resistance is a measure of the ability of a building element to resist a fire. It is the time for which the element can meet certain criteria during exposure to a standard fire resistance test. Fire resistance is a property assigned to building *elements* which are constructed from a single material or a mixture of materials. A building *material* on its own does not possess fire resistance. A *fire resistance rating* is the fire resistance assigned to a building element on the basis of a test or some other approval system. Some countries use the terms *fire endurance rating* or *fire resistance level*.

5.1 Failure criteria

The three failure criteria for fire resistance testing are *stability*, *integrity* and *insulation*. These apply in different ways to various assemblies as shown in Table 2. The integrity and insulation criteria are for containment, to prevent fire spreading from the room of origin.

1. To meet the *stability* criterion, a structural element must perform its load bearing function and carry the applied loads for the duration of the test, without structural collapse.

2. For *integrity*, the test specimen must not develop any cracks or fissures which allow smoke or hot gases to pass through the assembly.
3. For *insulation*, the temperature rise on the unexposed side of the test specimen must not exceed an average of 140°C or a maximum of 180°C at a single point.

Table 2: Failure criteria for construction elements.

	Stability	Integrity	Insulation
Partition		X	X
Door		X	X
Load-bearing wall	X	X	X
Floor-ceiling	X	X	X
Beam	X		
Column	X		
Fire resistant glass		X	

Some building codes specify the required fire resistance separately for stability, integrity and insulation. For example a typical load bearing wall may have a specified fire resistance rating of 60/60/60, which means that a one hour rating is required for stability, integrity and insulation. If the wall was non-load bearing, the specified fire resistance rating would be - /60/60. A fire door with a glazed panel may have a specified rating of - /30/ - , which means that this assembly requires an integrity rating of 30 minutes, with no requirement for stability or insulation.

5.2 Approvals

Most countries require that fire resistance tests be certified by a recognised

testing laboratory. In North America, independent testing organisations maintain registers of fire resistance ratings (SWRI 1996, UL 1996, ULC 1989). Most of these ratings are based on standard tests. Ratings based on these approvals are listed in some national building codes (eg NBCC 1995, UBC 1997). Small countries may need to use approvals from other countries, so that in New Zealand for example, a register of approved listings is maintained by the national standards organisation (SNZ 1991). Some trade organizations (eg. ASFPCM 1988; Gypsum Association 1994) maintain industry listings of approvals for products manufactured or used by their members. Listings generally fall into three categories; *generic ratings*, *proprietary ratings*, or *calculation methods*.

1. *Generic* fire resistance ratings, or “tabular ratings” are lists which assign fire resistance to typical materials such as concrete or steel. Generic ratings are derived from full-scale tests carried out over many years, and are widely used because they can be applied to common materials in any country. However, generic ratings often make no allowance for the size and shape of the fire exposed member or the level of applied load at the time of the fire.
2. *Proprietary* fire resistance ratings apply to proprietary products made by specific manufacturers,

so they may be more accurate than generic ratings, but cannot be applied to similar products from other manufacturers. Load levels are not usually considered.

3. *Calculated* fire resistance ratings are becoming more common as the science of fire engineering develops, especially for structural members and assemblies. Many national design codes include approved calculation methods for assessing fire resistance. Calculations should be verified by full scale fire resistance test results of similar assemblies.
4. *Expert opinions* are increasingly used by approving agencies to establish resistance ratings. The opinion will state whether the assembly would be considered likely to pass a test, based on observations of similar successful tests and the considered experience of the testing and approving personnel.

6. Structural design

The overall process of calculating structural fire performance is shown in Figure 8. The *fire model* has been described above. The *heat transfer model* may consist of tabulated or graphical data, or preferably a finite element heat transfer model which includes the variations of thermal properties with increasing temperatures (eg Franssen et

al. 2002). The *structural model* need to consider factors such as transient temperature gradients, variation of thermal properties with temperature, axial and flexural restraint, thermally induced forces, and thermally induced deformations, throughout the duration of the expected fire. The effects of creep are not usually included explicitly in the calculation, but the stress-strain relationships can be modified to include creep implicitly (EC3 2002).

Figure 9 shows a typical deformed shape calculated for a structural frame (Bresler and Iding 1982), where it can be seen that a localized fire has a significant effect on structural behaviour elsewhere in the frame.

Figure 10 shows the bending moment diagram for a simply supported beam designed to support both dead load and snow load. Under factored design loads of self weight and snow load, the bending moment diagram is shown by the solid curve, where the mid-span bending moment is $M_{cold}^* = w_c L^2/8$ (where w_c is the uniformly distributed factored load and L is the span).

The short term flexural capacity under cold conditions is shown as R_{cold} (horizontal line). The designer selects a beam size such that R_{cold} is greater than M_{cold}^* to allow a safety margin under normal temperature conditions. If a fire occurs when there is no snow on the roof, the bending moment at mid-span will less, as shown by the dotted curve. The mid-span bending moment is given by $M_{fire}^* = w_f L^2/8$ (where w_f is the load

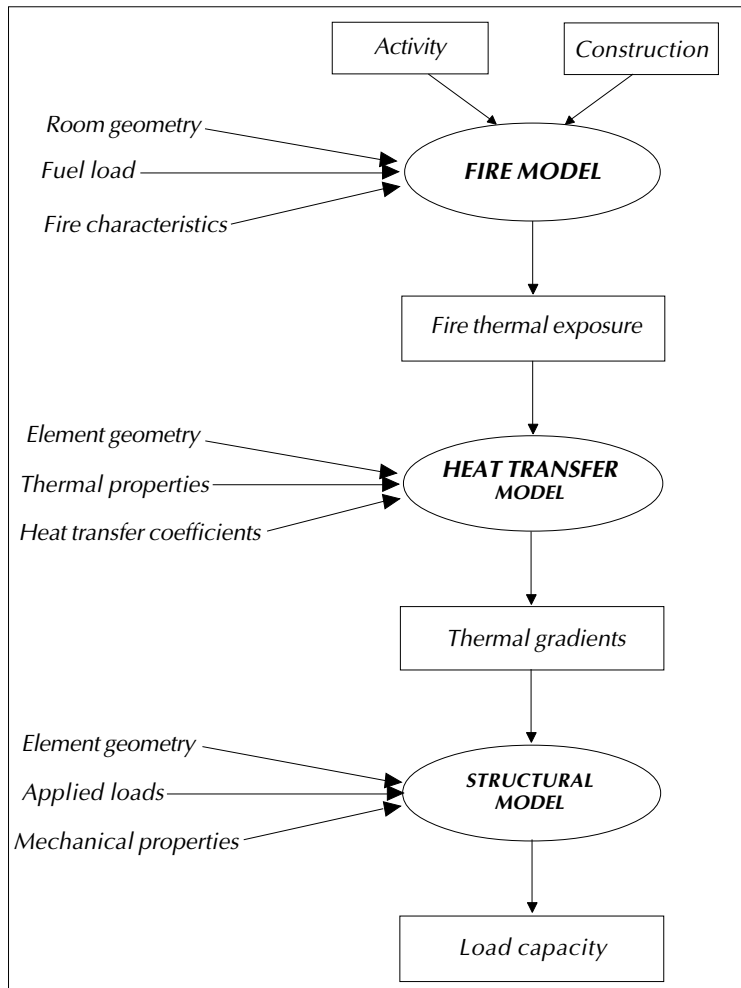


Figure 8: Flow chart for structural design in fire.

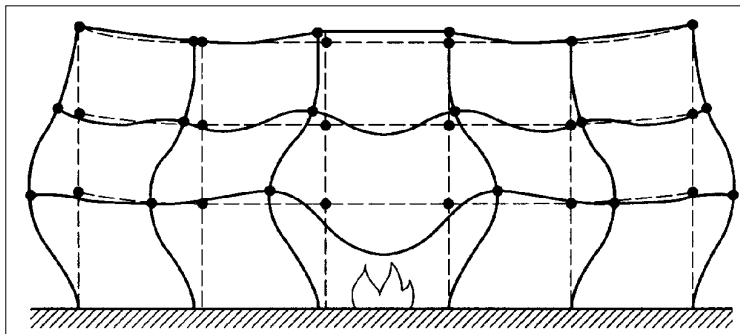


Figure 9: Deformed shape for a fire in the lower central bay of a structural frame.

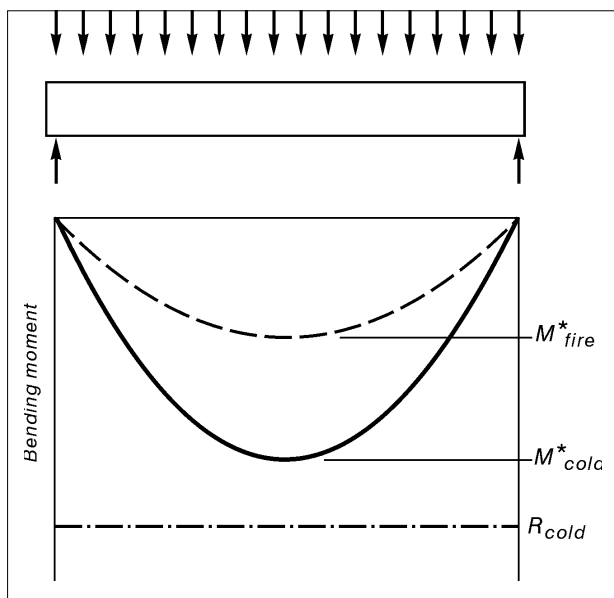


Figure 10: Bending moment diagram for simply supported beam.

for fire conditions). It can be seen that for failure to occur as a result of the fire, the flexural capacity would have to drop from R_{cold} to M_{fire}^* . The design equation becomes

$$M_{fire}^* \leq M_f \quad (10)$$

where $M_f = f_{b,f} Z_f$
 $f_{b,f}$ is the characteristic flexural strength of the material at elevated temperature
 Z_f is the section modulus of the cross section (possibly reduced by fire exposure)

The ratio M_{fire}^* / R_{cold} is the “load ratio”. This example demonstrates the important principle that if the load ratio is low, the necessary drop in strength for failure to occur is large, hence the greater the fire resistance.

Figure 11 shows a similar example for one span of a beam continuous over several supports. Flexural continuity can improve the fire resistance of a flexural member which would need to fail at more than one cross section to lose its load carrying capability. A simply supported beam has no continuity, so failure at mid-span will cause collapse. Beams which are continuous over several supports or built into rigid frames have better fire performance than simply supported beams because more than one plastic hinge is needed to cause failure. An additional factor is that moment redistribution can occur, to further increase the fire resistance, as shown by the dotted curve in Figure 11.

Figure 12 shows the bending moment diagram for a simply supported beam built into strong supports which can prevent axial elongation during a fire. The resulting axial restraint can be beneficial to the fire behaviour, provided that the line of action of the restraint force is low in the cross section.

The fire resistance of simple members such as these can be assessed by

calculation. However, large ductile and redundant structures may have much better fire performance than predicted by these methods, because there are so many alternative mechanisms and load paths for the structure to resist the applied loads during a fire. A large series of full-scale fire tests in realistic buildings at Cardington in the UK has demonstrated this behaviour.

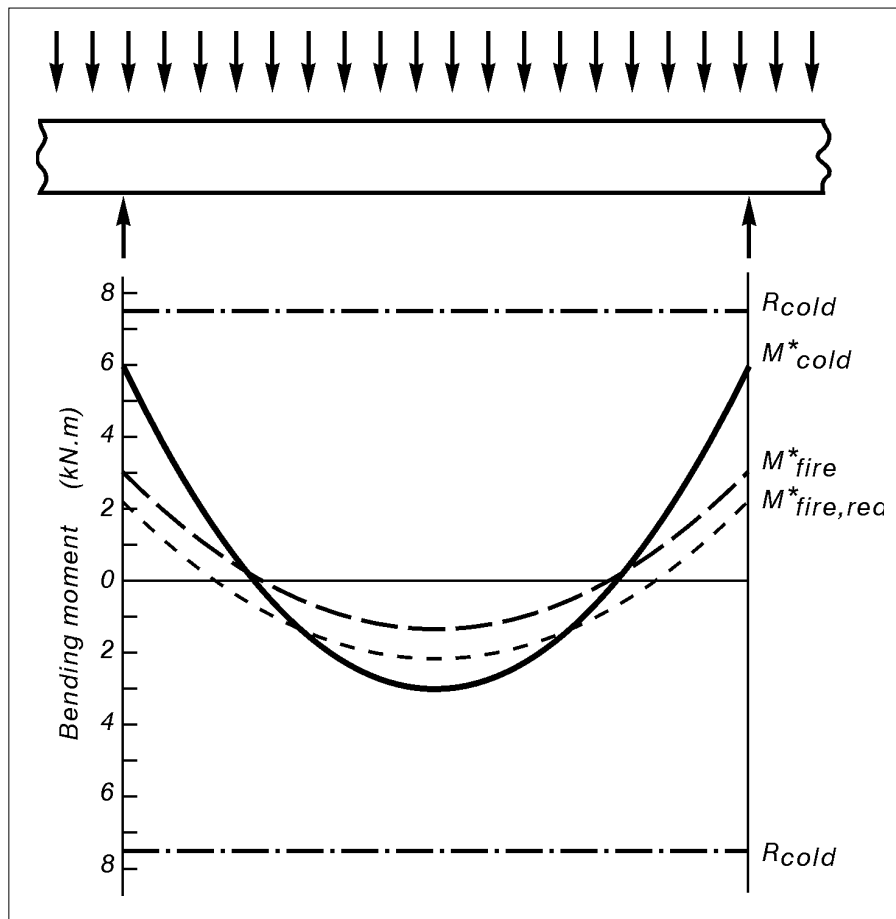


Figure 11: Bending moment diagram for continuous beam with moment redistribution.

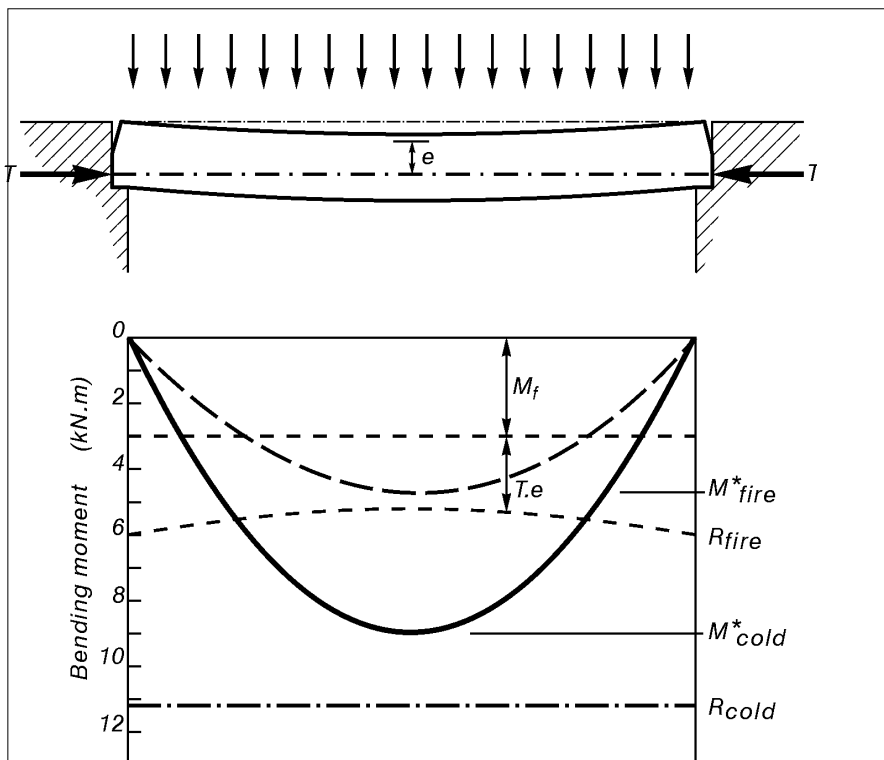


Figure 12: Bending moment diagram for simply supported beam with axial restraint.

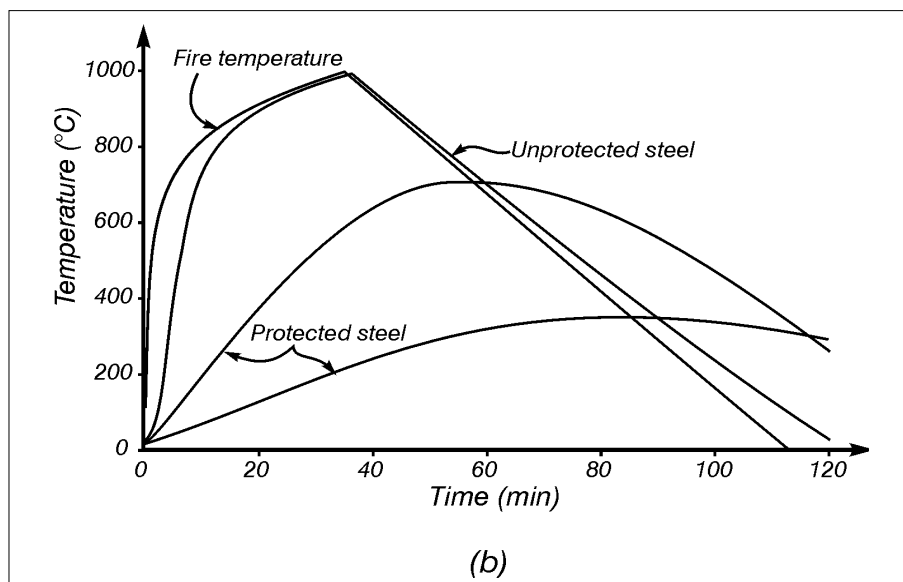


Figure 13: Temperature of protected and unprotected steel beams exposed to fire.

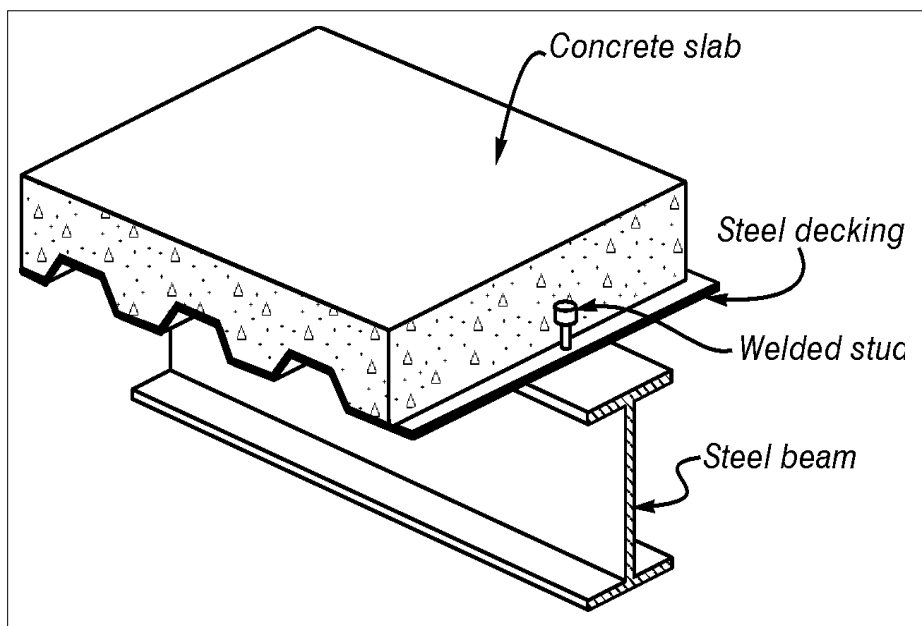


Figure 14: Composite steel-concrete construction.

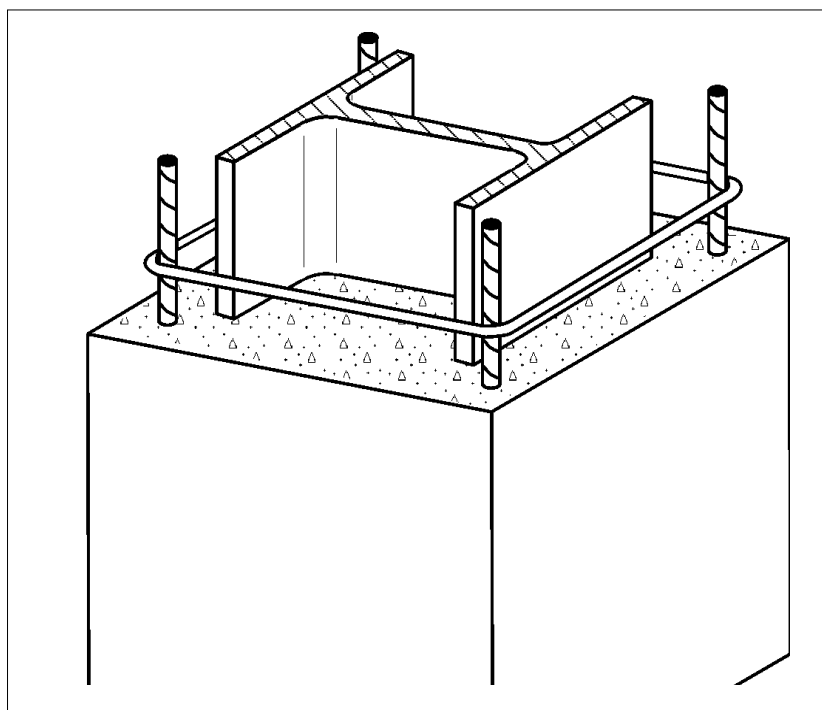


Figure 15: Structural steel column protected by reinforced concrete.

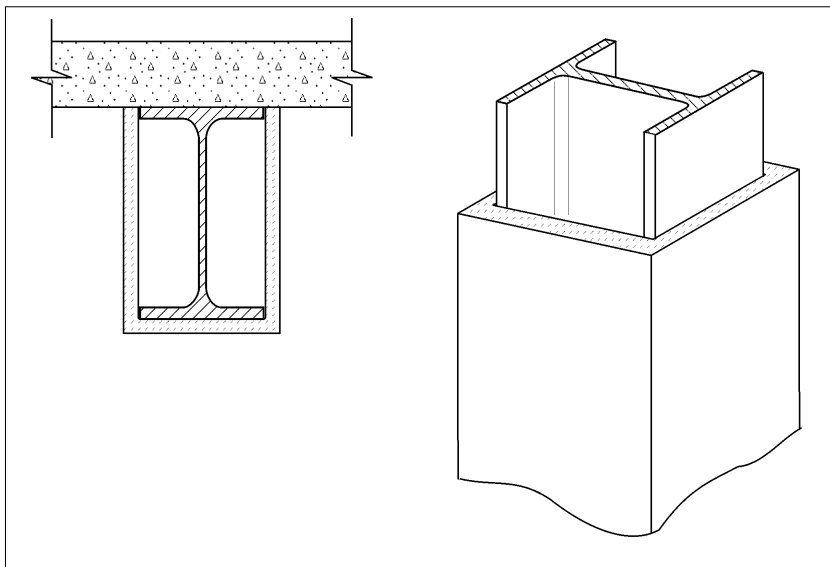


Figure 16: Structural steel beam and column protected by fire-resisting board material.

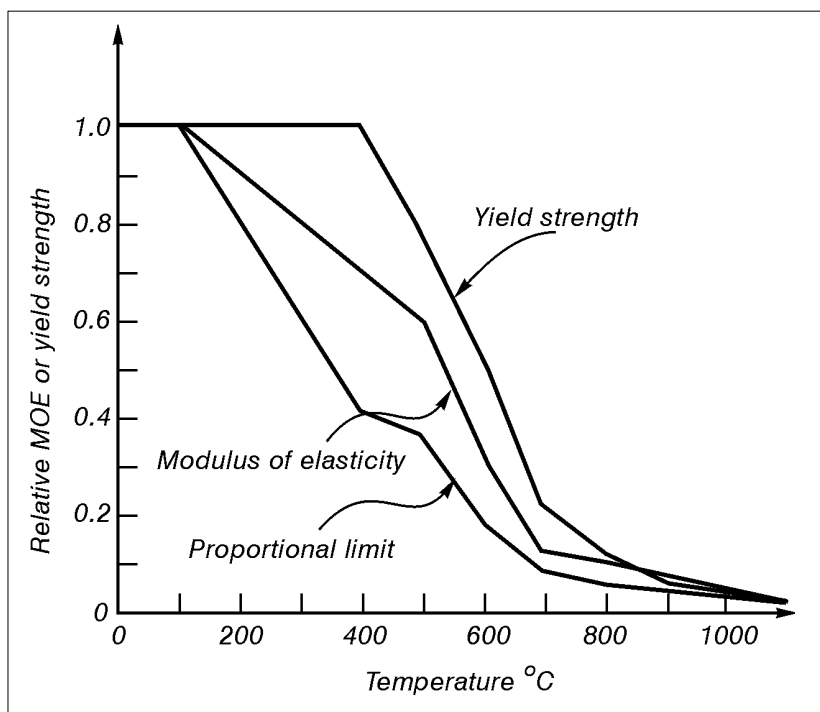


Figure 17: Mechanical properties of steel at elevated temperatures.

7. Steel structures

For steel structures exposed to fires, it is essential to know the temperature of the structural members. Figure 13 shows how the temperature of an unprotected steel member follows the temperature of the fire with only a small time lag, which depends on the size and shape of the cross section. Depending on the type and thickness of protection, a protected steel member can have much lower temperatures than the fire, as shown by the lower two curves for different levels of protection.

Figure 14 shows typical concrete-steel composite construction, where the bottom flange and web of the steel beam will have temperatures close to the fire temperature, but the top flange will be cooler due to the thermal mass of the concrete slab which protects the top of the flange. Design for composite construction is described in Eurocode 4 (EC4 2003).

Figures 15 and 16 show typical methods of providing fire protection to reduce the rate of temperature rise in structural steel members exposed to severe fires. Encasement in reinforced concrete as shown in Figure 15 is traditional, but rarely used because it is very expensive. Lightweight spray-on materials or board materials (Figure 16) are more widely used because they are much more economical.

Once the steel temperatures are known, it is necessary to know how the mechanical properties of the steel are reduced by the elevated temperatures. Figure 17 shows the reduction in mechanical properties from Eurocode 3

(EC3 2002), with separate lines for proportional limit, yield strength and modulus of elasticity. These properties can be used to make calculations such as shown in Figure 10. All other aspects of structural behaviour such as lateral buckling of beams or instability of columns must also be considered (Buchanan 2001; Vila Real 2003).

8. Concrete structures

The behaviour of reinforced concrete structures in fire is summarised in Figure 18, which shows a simply supported concrete slab exposed to fire from below. The bending moment diagram is shown to the right. As explained above, it is essential that the flexural capacity during the fire exceeds the bending moment M_{fire} resulting from fire reduced loads on the structure. Calculation of flexural capacity is shown in the lower part of the figure, where the only effect of the fire is to reduce the yield strength of the reinforcing steel. The temperature of the rebars can be determined by calculation or by simple charts such as shown in Figure 19 (adapted from EC2 2002).

In this particular case of positive bending moment throughout the span, the effect of temperature on concrete strength is not important because the concrete in the compression region is protected from the fire.

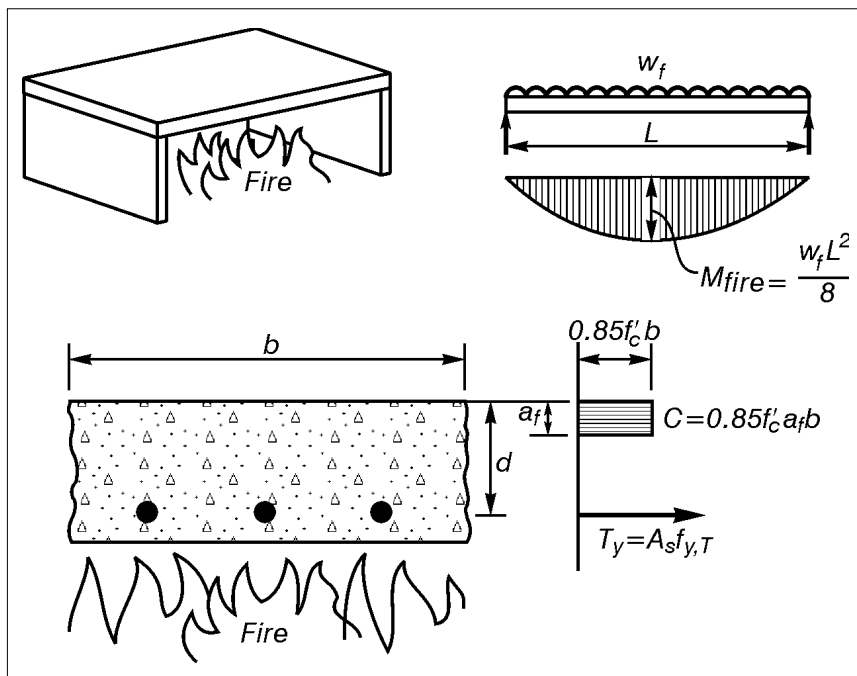


Figure 18: Simply supported concrete slab exposed to fire.

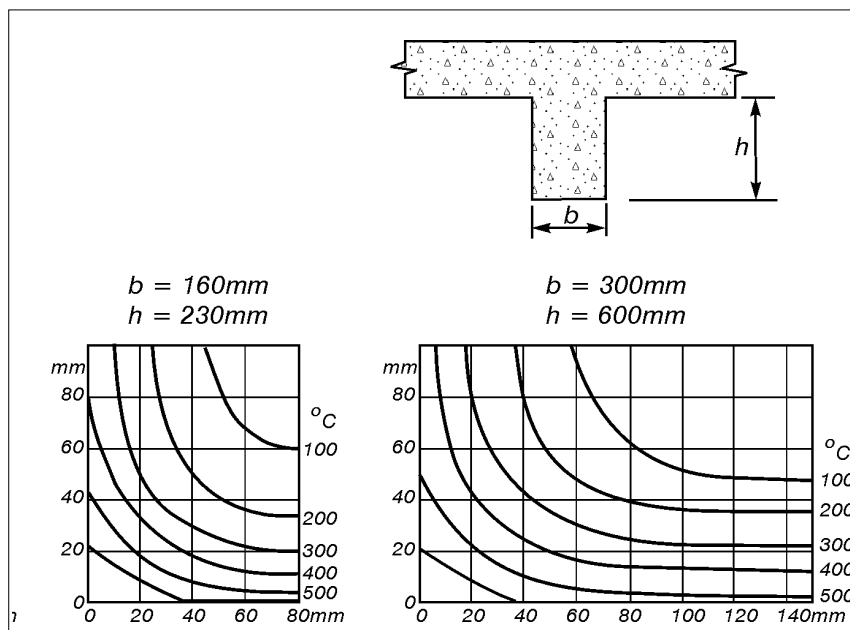


Figure 19: Thermal gradients in reinforced concrete beam exposed to standard fire.

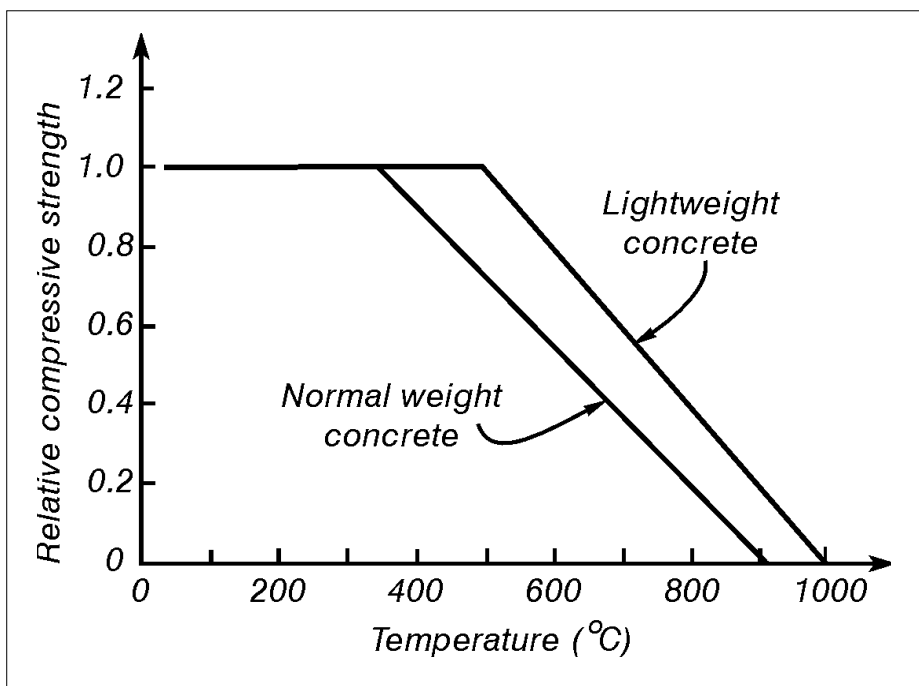


Figure 20: Reduction of concrete strength with elevated temperature.

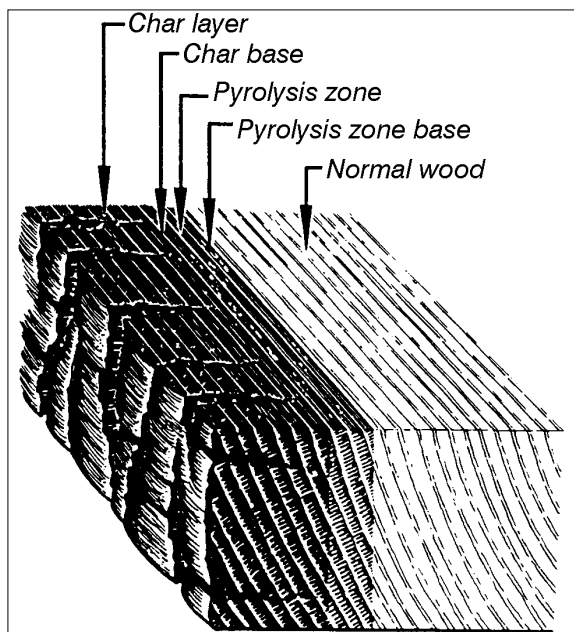


Figure 21: Surface charring of heavy timber member in fire.

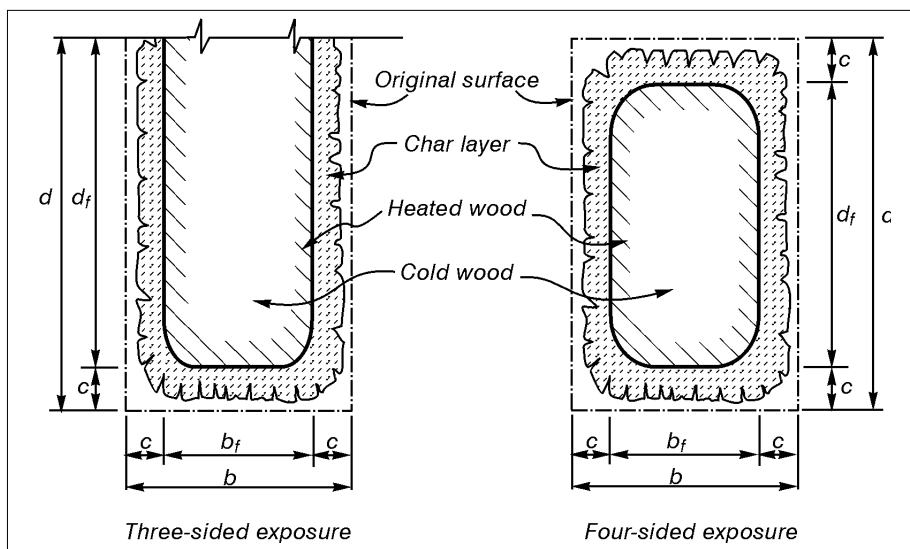


Figure 22: Reduced cross section of charred member, for structural design.

The reduction of concrete compression strength at elevated temperature, shown in Figure 20, becomes very important for columns, or for continuous or restrained concrete beams and slabs where the compression region is exposed to the fire.

9. Timber structures

The fire performance for timber structures must be considered separately for heavy and light timber structures.

9.1 Heavy timber construction

Heavy timber structures are those where the principal structural elements are beams, columns, decks, or truss members made from glue laminated timber or large dimension sawn timber,

usually with the smallest dimension of about 100mm. Many historic commercial and industrial buildings consist of external load-bearing masonry walls, with internal timber columns and beams supporting thick timber floor decking. Heavy timber construction has become recognised as having very good fire resistance. There are many well documented examples of structures surviving severe fire exposure without collapse, and many of these have been repaired for re-use.

When large timber members are exposed to a severe fire, the surface of the wood initially ignites and burns rapidly. The burned wood becomes a layer of char which insulates the solid wood below. The initial burning rate decreases to a slower steady rate which

continues throughout the fire exposure. The charring rate will increase if the residual cross section becomes very small. The temperature of the outer surface of the char layer is close to the fire temperature, with a steep thermal gradient through the char. The boundary between the char layer and the remaining wood is quite distinct, corresponding to a temperature of about 300°C.

Below the char layer there is a layer of heated wood about 35 mm thick. The part of this layer above 200°C is known

as the pyrolysis zone as shown in Figure 21 (Schaffer 1967). The wood in the pyrolysis zone is undergoing thermal decomposition into gaseous pyrolysis products, accompanied by loss of weight and discolouration. When wood further below the char layer is heated above 100°C, the moisture in the wood evaporates, some of this moisture travels out to the burning face, but some travels further into the wood, resulting in an increase in moisture content. The heated layer of wood below the char has reduced mechanical properties because of the increased temperature and the changing moisture content.

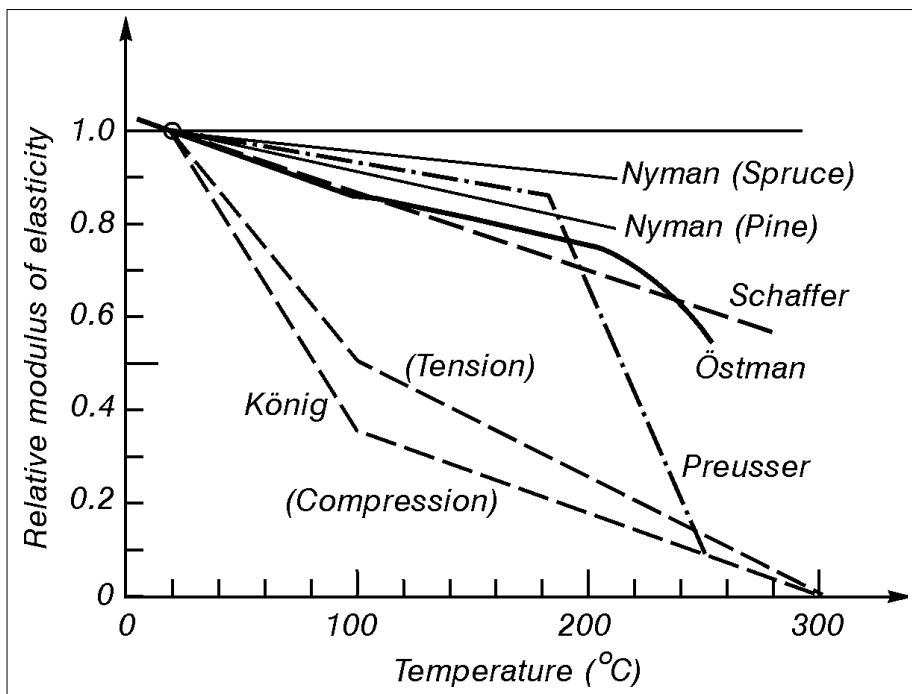


Figure 23: Reduction of mechanical properties of wood with elevated temperature.

The reduction of modulus of elasticity of wood at elevated temperature is shown in Figure 23. It can be seen that there is a lot of variability depending on species, the moisture content of the wood and the type of loading.

Structural design of heavy timber members is based on the rate of charring of the wood surface, so it is not necessary for designers to calculate temperatures within the fire-exposed wood. The rate of charring of most species of softwood is in the range 0.6 to 0.7mm/minute. Figure 22 shows the reduced cross section which can be used for design. Design methods for timber construction are described in Eurocode 5 (EC5 1994).

9.2 Light timber construction

Light timber frame construction uses smaller sizes of wood framing, as studs in walls, and as joists in floors. Walls and floors are covered with panels of lining materials to provide resistance to impact, sound transmission, and fire spread. Unprotected small sizes of wood members will burn away rapidly in a severe fire because of the large ratio of exposed surface area to volume. For this reason it is essential to protect such timber members with non-combustible linings, most often gypsum plasterboard. A large number of research projects have investigated the fire performance of light timber construction, but there are no simple calculation methods for

design. Most designers use proprietary fire resistance ratings published by the manufacturers of fire resistant lining materials.

10. Sources of information

By far the most comprehensive international codes for structural design of buildings and structures in fire conditions are the Structural Eurocodes, listed below:

- Eurocode 1 Basis of design and actions on structures;
- Eurocode 2 Design of concrete structures;
- Eurocode 3 Design of steel structures;
- Eurocode 4 Design of composite steel and concrete structures;
- Eurocode 5 Design of timber structures;
- Eurocode 6 Design of masonry structures;

All of these have substantial fire sections. Most are nearing completion in final draft form, and will go to a formal vote this year after final editing and translation into French and German.

All the structural Eurocodes include sections on fire exposure, verification methods and structural analysis. Mechanical properties and thermal properties are given for the main materials. The fire exposure allows for standard or realistic fire design curves to be used. Design procedures are given as a hierarchy:

1. *tabulated data* for selection of simple generic elements, based on standard fire exposure without any calculation of fire severity or structural behaviour;
2. *simple calculation methods* for predicting the structural behaviour of single members based on simple assumptions about the fire and structural behaviour;
3. *advanced calculation methods* provide the principles for computer analyses based on fundamental physical behaviour, for both thermal analysis and mechanical behaviour.

In the United Kingdom, a comprehensive recent publication is *Structural Response and Fire Spread Beyond the Enclosure of Origin* (BSI 2003) which is a "Published Document" in support of BS 7974 *Application of Fire Safety Engineering Principles to the Design of Buildings* (BSI 2001), which is the most comprehensive code for specific fire engineering design in any country.

Structural design for fire safety in the United States has not moved as quickly as in Europe. Existing building codes include prescriptive requirements for fire resistance which have not changed greatly in recent years. The current movement from regional to national building codes (IFCI 2000, NFPA codes) has not been accompanied by significant changes in design for fire

resistance. However the need for change has been recognized and several recent documents (ASCE/SFPE 1999; SFPE 2003) will contribute to code changes. The collapse of the World Trade Center towers in 2001 has given new impetus for change (FEMA 2002). Industry groups for particular materials (steel, concrete and timber industries) are also developing standards and guidance documents for structural fire resistance.

11. Conclusions

- Design for fire safety must be based on clearly established objectives.
- Structural design is only part of the overall provision of fire safety. Non-structural fire safety must be included as part of a comprehensive fire safety strategy.
- Structural analysis and design of buildings in fire conditions is more complex than for normal temperature conditions.
- Structural design for fire exposure must consider many factors including the fire severity, the loads induced into the structure, and the material properties at elevated temperatures.
- It is possible to use simple methods for specifying the required fire resistance, and to use tabulated data for compliance. The more simple the method, the more conservative the underlying assumptions need to be, and the less the accuracy.

- Design methods for steel, concrete and timber structures are available.
- For large or prestigious buildings where the consequences of failure are most serious, the need for advanced structural design for fire safety becomes very important.
- Structural design for fire conditions must consider many different fire scenarios, with realistic assessment of possible fire conditions.

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