SOURCE http://www.kuleuven.be/bwk/materials/Teaching/master/wg04b/l0400.htm

Lecture 1.Introduction to Fire Safety

OBJECTIVE

To introduce the global approach to fire safety. To give an overview of assessment methods for the structural fire resistance of load-bearing elements.

SUMMARY
Fire losses, fire risk and the objectives of fire safety are described as background to the fire safety concept involving structural, monitoring and extinguishing concepts. Cost-effectiveness is discussed and an overview of assessment methods of structural fire resistance is presented. The effect of active fire protection is introduced.

1. INTRODUCTION

1.1 Fire Losses

An international survey of fire losses gives the following values:

- Human fatalities: 4 to 34 fire fatalities per million head of population.
- Financial losses: 1,6 to 5,9 0/00 of the Gross National Product per year.

In order to obtain an overall perspective of the risk of fire fatalities in buildings it is interesting to compare it with other accidental causes.

Activity	Fatal accident rate per person and for an average life-time of 70 years.
Motor cycling (UK)	4,1
Scheduled flights (USA)	1,5
Average for disease (USA)	0,7
Travelling by car (USA)	0,6
Travelling by car (UK)	0,4
At home - average (excl, sickness)	0,02
At home - total able bodied persons	0,01
Fires in hotels (UK)	0,01
Fires in dwellings (UK)	0,001
Natural disasters (USA)	0,0001

Table 1 Comparison of fatality statistics from different accidental causes. (Sources [1] [2] [3])

Although the risk of life loss in fire is quite low in comparison with other causes of death, there is a tendency for an incident involving multiple fatalities, over about 5 deaths, to attract a high level of news coverage. In this sense building fires tend to be regarded in the same high profile way as air crashes or earthquakes. Nonetheless it is important that the causes of fire fatalities should be examined with a view to public safety.
A breakdown of fatal casualties by fire location shows for Europe and USA that approximately 80 to 85% of all fatalities occur in domestic buildings (dwelling, flat) and only 10% in public buildings. On another hand about 95% of all deaths in buildings are due mostly to smoke or in very few cases to heat.
A survey on non-domestic fires in the Netherlands and France shows that the financial loss of the building content outweighs the cost of building damage [4].

Losses to building content	43%	= together 4/5
Consequential losses	36%	
Losses to building	21%	= 1/5

The indication is that damage to content and consequential losses are more significant financial factors than damage to buildings.
The global cost of fire includes in Europe the following items (in 0/00)

• direct fire losses on building and content	2 - 5
• consequential losses	0,2 - 3
• human fatalities	0,3 - 2
• fire brigade costs	1 - 3
• administration costs of insurers	1 - 3
• education costs, cost of research	0,1 - 0,5
• cost of fire safety measures in buildings	2 - 5

and varies between 1,3 to 2,0% of the Gross National Product. The last item, e.g. the cost of fire safety measures in buildings represents as an average about 1 to 3% of the total building costs. In most countries a high investment in fire safety in buildings brings a reduction on direct, indirect and human losses, Still it is very important to analyze the cost-effectiveness or in other words the return of investment for each detailed fire precaution measure, see Section 1.5.

1.2 The Fire Risk

The usual way to measure the risk of fire is expressed by

$R_{fire} = P \times L_{ext}$

where
R_{fire} = Fire Risk
P = the probability of occurrence of a fire
L_{ext} = probable extent of total losses.
$R_{fire} < R_{accepted}$
$R_{accepted}$ represents the target risk which varies from country to country.

The risk R can never be zero and we have to accept a certain level of risk for every type of building and/or occupancy. This level will depend on the number of persons, their ability to escape and the value of content exposed to fire.

Table 2 gives some indications of the occurrence of fire in different types of building.

Type of building occupancy	Source	Number of fires per million m^2 floor area per year
INDUSTRIAL BUILDINGS	United Kingdom [5] Germany [6] CIB W14 [7]	2 2 2
OFFICES	United Kingdom [5] USA [8] CIB W14 [7]	1 1 0,5 + 5
DWELLINGS	United Kingdom [5] Canada [9] CIB W14 [7]	2 5 0,05 +2

Table 2 Occurrence of fire

The probable extent of losses varies for different occupancies and is a function of the degree of compartmentation, type of building, extent of automatic detection and extinguishing devices (Sprinkler/CO_2/Halon), structural fire resistance and of the involved fire brigade.

The probability of fires getting out of control is strongly related to the type of active measures available, as indicated in the table below (reference CIB W14 Workshop "Structural Fire Safety" [7]).

Type of active measures	Probability of fires getting out of control
Public fire brigade	100/1000
Sprinkler	20/1000
High standard residential fire brigade combined with alarm system	≥ 10/1000 : 1/1000
Both sprinkler and high standard residential fire brigade	≥ 1/10.000

Table 3

1.3 Objectives of Fire Safety

Fire safety in buildings is concerned with achieving two fundamental objectives:

1. to reduce the loss of life in, or in the neighbourhood of, building fires.
2. to reduce the property or financial loss in, or in the neighbourhood of, building fires.

In most countries the responsibility for achieving these objectives is divided between government or civic authorities, who have responsibility for life safety via building regulations, and insurance companies who are concerned with property loss through their fire insurance policies.
Often the two objectives are thought to be incompatible, even occasionally conflicting. For example, sprinklers and automatic detection devices tend to be regarded as property protectors rather than life protectors and insurance companies will commonly offer substantial premium discounts when they are used. They do not figure highly in most national building regulations, yet the evidence that is available suggests that they are extremely effective in preserving life.
In fact the actions required to achieve life and property preservations are very similar.Figures 1a and 1b use a systematic approach to identify the major options to reduce losses. They show that practically all options reduce the risk of human losses as well as the risk of financial direct and consequential losses. In fact we must realize that global fire safety must ultimately be answered by adequate fire safety concepts.

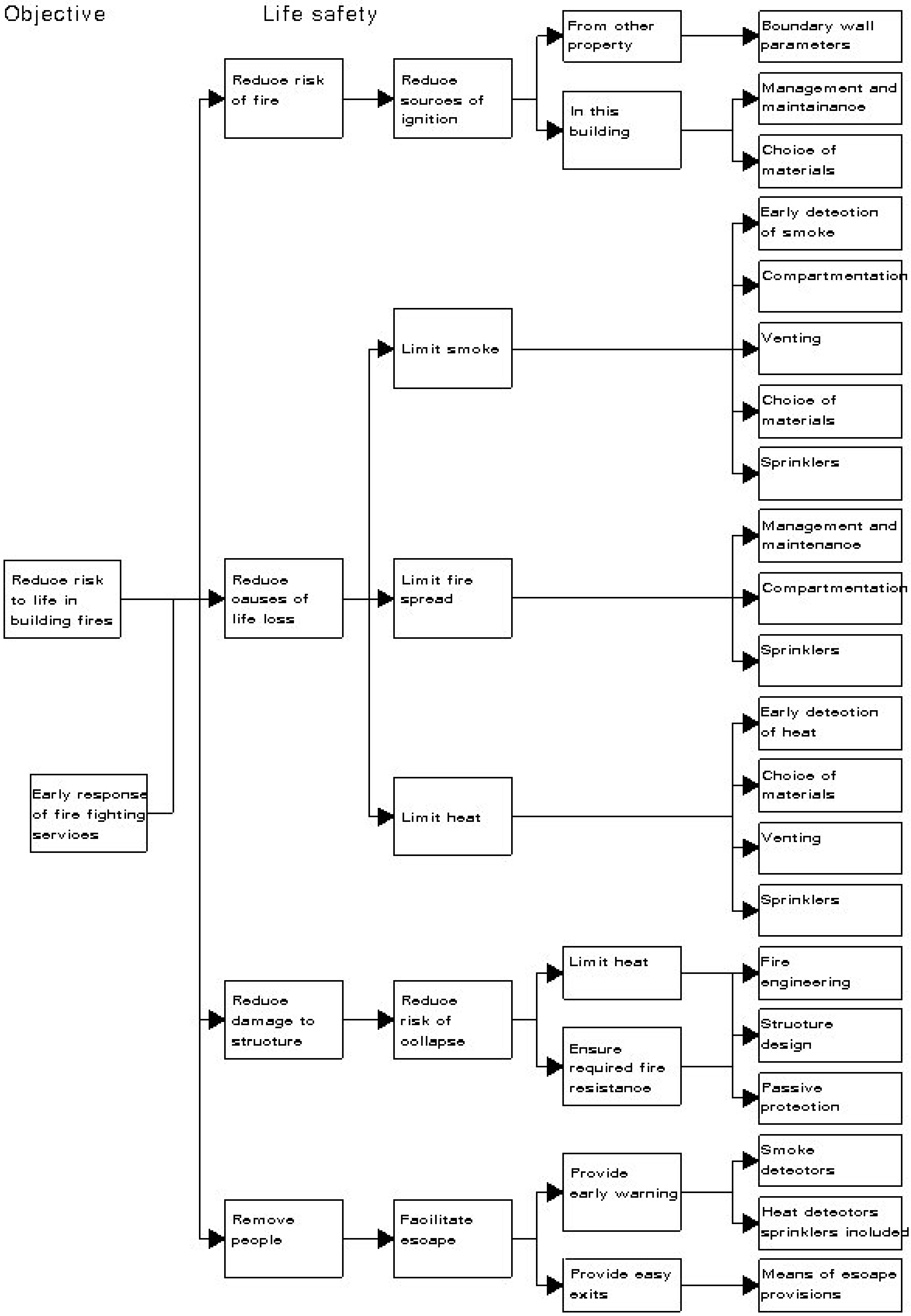
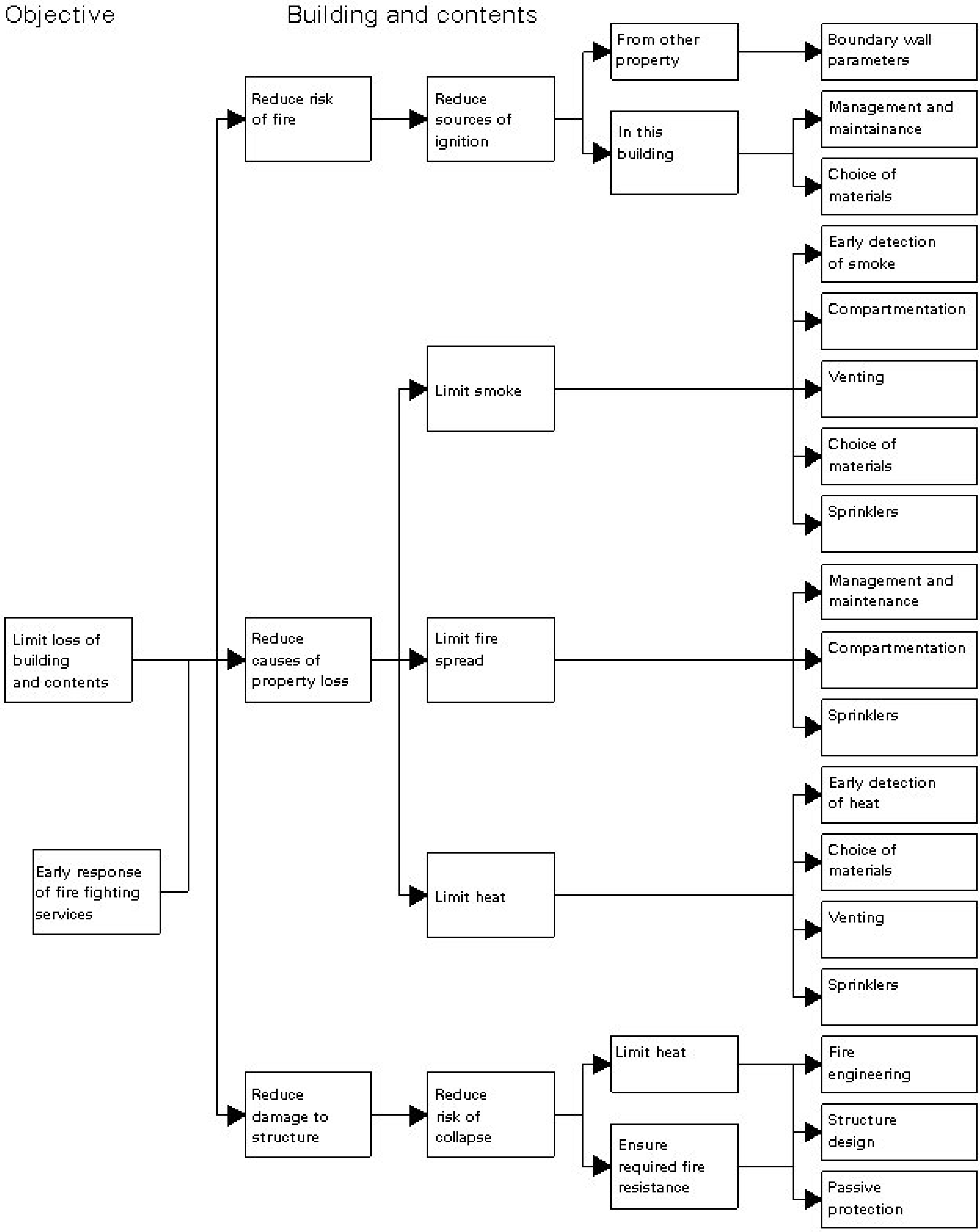


Figure 1a Reducing risk of human losses



Note : The fire safety provisions are identical in both cases shown in Figure 1a the means means of escape provisions are the only difference

Figure 1b Reducing risk of property losses

1.4 Fire Safety Concept

Fire safety concepts are defined as optimal packages of integrated structural, technical and organisational fire precaution measures which allow well defined objectives agreed by the owner, the fire authority and the designer to be fulfilled.

In order to develop possible fire safety concepts it is essential to show the usual development of an uncontrolled fire as shown in Figure 2.

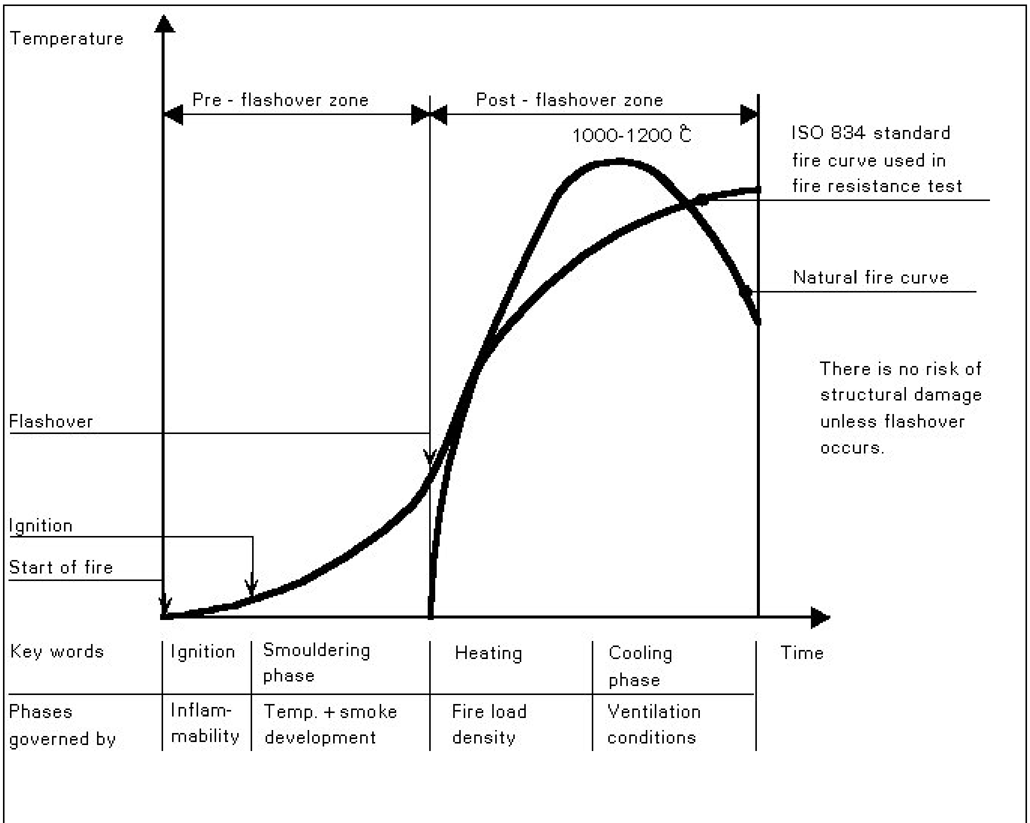


Figure 2 Development of an uncontrolled fire

Another very similar presentation given as Figure 3 allows the reasons for success or failure of well-defined fire precaution measures to be visualised.

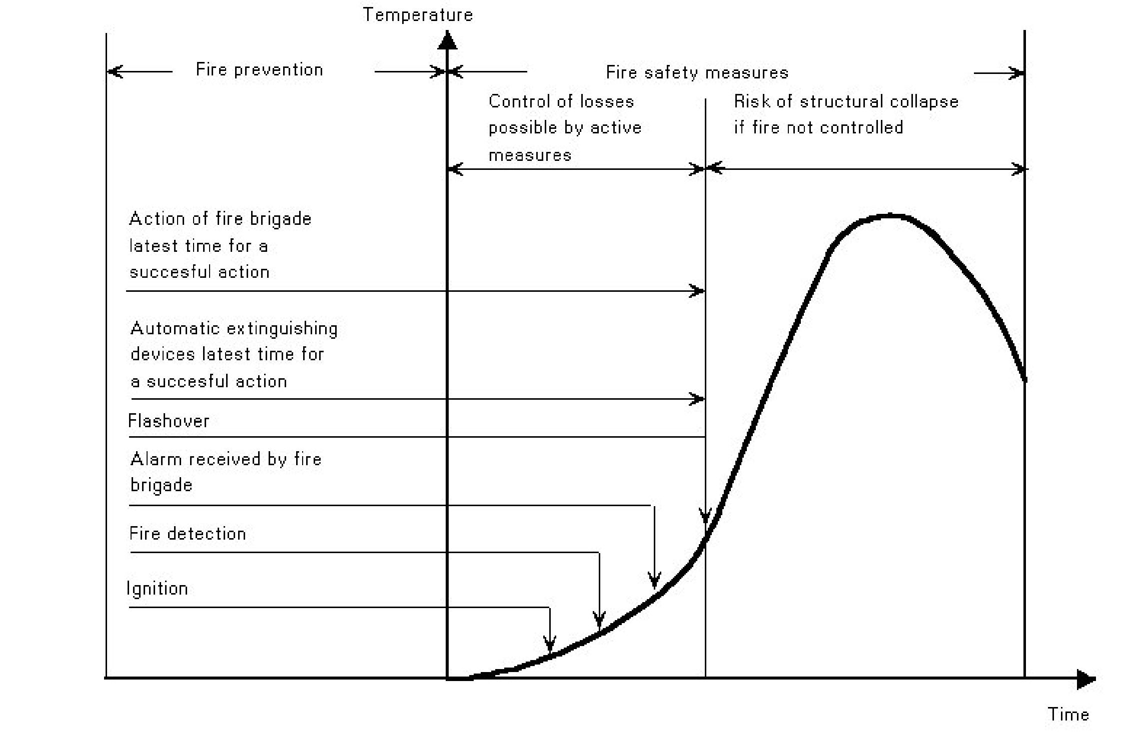


Figure 3 Fire precaution measures and conditions for effective control

Analysing this figure we realize that we will be able to overcome the fire risk through three basic concepts which are:

- a structural concept accepting the occurrence of flash-over in a limited number of fire compartments.
- a monitoring concept avoiding the occurrence of flash-over.
- an extinguishing concept avoiding the occurrence of flash-over.

1.4.1 Structural fire safety concept

A structural concept comprises compartmentation combined with an adequate fire resistant structure; it may be the best choice as long as the normal (cold-design) use of the building allows compartmentation by fire resistant floors and walls.

It is admitted that the fire may reach flashover conditions before fire fighting action begins.

The necessary time of fire resistance should be determined by the condition that the fire should not spread outside the fire compartment. Hence the separating and (possibly) load-bearing function of the relevant building components should be maintained during the anticipated duration of the fire.

Whenever possible fire spread should be limited by fireproof partition walls and floors. Combustible building components should be designed or treated to prevent fire spread by smouldering, eg. in two layer built-up roofs the combustible layer should be covered by a non-combustible one. The design of the facade should prevent flames climbing into an upper storey.

It is important to underline that all partition elements like walls, decks, ceilings and roofs (in some cases) must fulfil three criteria to be classified in a fire class (30/60/90...).

- a load bearing criteria proving the stability of the element.
- an insulation criteria proving the insulation capacity of the element.
- an integrity criteria proving that no flames and no smoke goes through the element.

The load bearing structural elements with no partitioning function only have to fulfil the first criteria.

Fire resistance of the building components is usually prescribed in the building codes where it is normally expressed in units of time.

The required time for fire resistance is usually expressed in terms of multiples of 30 minutes: for example 30, 60, 90 minutes, related to ISO Standard fire. This means that a component is able to fulfil its function during the required time under a temperature exposure according to ISO.

Actual office buildings realized in London are excellent examples of this type of concept.

The time-temperature relationship in the standard fire may significantly differ from that in a real fire but modern fire design procedures allow fire resistance to be determined for natural fires as will be shown in paragraph 1.5. The time criterion should not be interpreted as an escape time for occupants or an intervention time for the fire brigade.

For structures and their occupancies it is often more effective to use alternative concepts based on the avoidance of FLASHOVER by means of non-structural active fire precaution measures. Active measures are based on a monitoring or an extinction concept.

1.4.2 Monitoring concept

The monitoring concept is based on automatic detection devices and automatic alarm transmission to an adequate fire brigade (around the clock), preferably to an on-site fire brigade.

A monitoring concept (shown in Figure 4) which involves limited or no structural fire resistance may represent the best choice when the normal (cold-design) use of a building calls for a minimum of compartmentation.

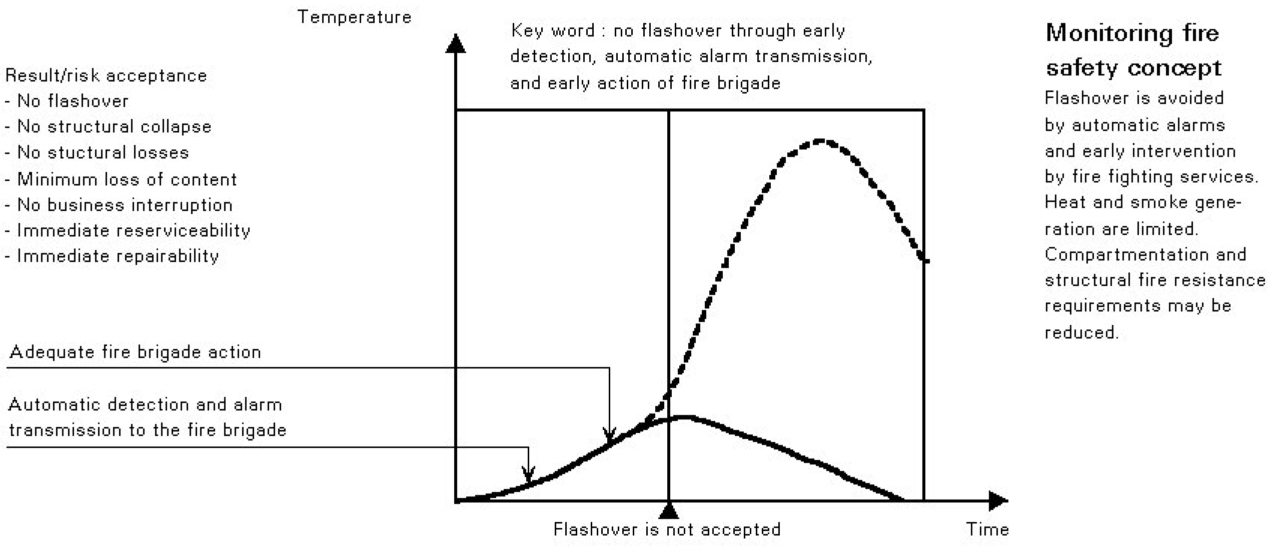


Figure 4 Monitoring fire safety concept.

It is most applicable for occupancies with reduced fire load densities, for low to medium-rise buildings in which fires may be expected to develop slowly and where an effective and quick-responding fire brigade is available.

- Fire detection

Automatic alarm systems are activated by smoke, heat or flames. They work mechanically or by electric or electronic systems. Preference is given to smoke detection, since this is, in general, by far the most effective way. When detectors begin to operate, an alarm is automatically set off. For maximum effectiveness, the alarm should be transmitted day and night to a nearby fire brigade station. Alarm systems with sound generating sirens are almost the only means against deliberate fires.

Sprinklers act as extinguishing devices and as a "slow" alarm system (heat detectors).

- Fire fighting

The effectiveness of fire fighting mainly depends on the time of arrival of the fire brigade and the access to fire.

The easiest means is the use of hand fire extinguishers, if there are people who detect the fire and who are skilled enough to use an extinguisher.

Fire fighting services may be either public fire brigades or works (on-site) fire brigades. Work fire brigades have the advantage of being acquainted with the locality and having shorter distances to reach the fire, but for all fire brigades it is essential to have access routes for their vehicles. For sprinklers as well as for fire brigades a sufficient water supply is necessary, and special precautions may be necessary in winter time. In a compartment the effective radius of action for firemen is up to 20 metres.

1.4.3 Extinguishing concept

The extinguishing concept is based on automatic extinguishing devices such as sprinklers, CO_2 or Halon-Systems with automatic alarm transmission to an adequate fire brigade and the owner. It is illustrated in Figure 5.

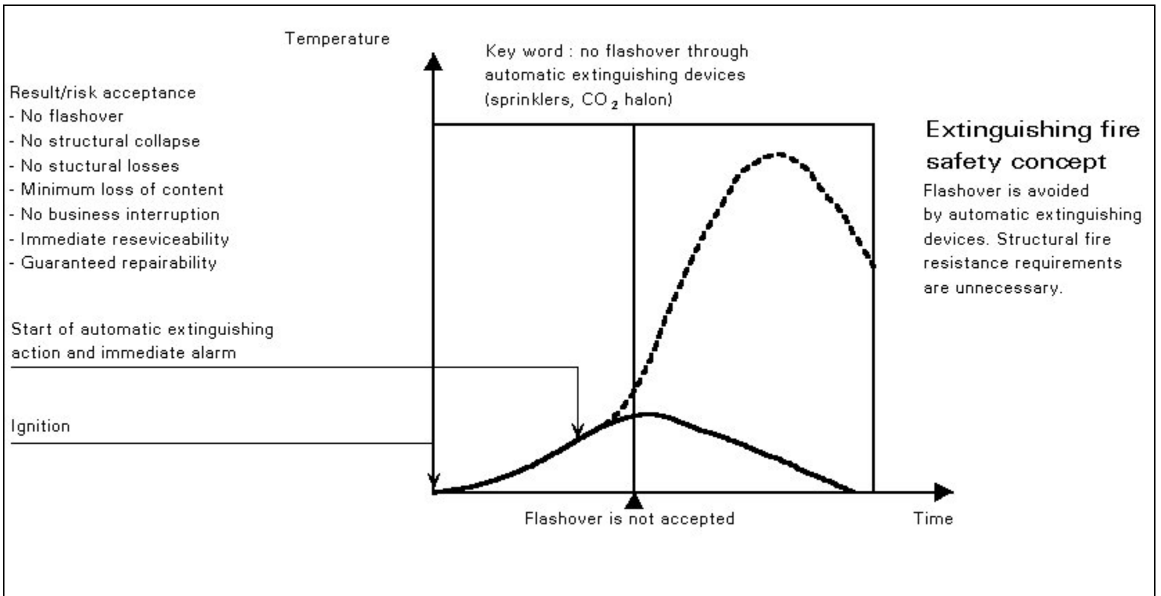


Figure 5 Extinguishing fire safety concept

The extinguishing concept with limited or no structural fire resistance may represent the best choice when the normal (cold design) use of a building calls for a minimum of compartmentation. It is most applicable for occupancies with medium or high fire load densities and fast developing fires.

Building owners often are afraid of the damage which these systems may cause by the water poured on the stored material or the manufacturing machines. But sprinklers open their valves only at the spot where temperature reaches a critical limit of 70° to 140°C. It has to be noted that 75% of all fires in premises with sprinklers devices are controlled by 1 to maximum 4 sprinkler heads. This represents approximately 50 m^2 watered by opened sprinkler heads. By means of an automatic alarm transmission system, they inform owner and fire brigade at once. It is important to know that automatic detection and extinguishing systems have to be maintained once or twice a year by specialists.

The alternative concepts of monitoring and/or extinguishing are gaining more and more acceptance in many countries. The brochure "Steel and Fire Safety: a global approach" edited by the Steel Promotion Committee of EUROFER, gives a survey how far these alternative concepts with no or reduced fire resistance requirements are internationally accepted.

1.5 Cost-Effectiveness

The type of occupancy and the choice of the structural "cold-design" are the main variable governing the amount of fire protection measures necessary and thus the cost of the total FIRE SAFETY CONCEPT. The cold-design concept and the fire safety concept should be integrated from the beginning in order to obtain an optimum safety level with a minimum of investment. This aim can only be reached through a dialogue between the designers of a building and the fire authority at a very early stage of the planning.

An outline cost-benefit analysis indicates that the return on investment in fire precautions is variable.

Figure 6 shows that, as the expenditure level and therefore also the level of safety precautions is chosen higher, the loss expectation due to fire will decrease. This relation is indicated schematically by the broken line. The loss-expenditure curve has a hyperbolic shape which means that, beyond a certain point, there is little benefit in increasing the level of protection.

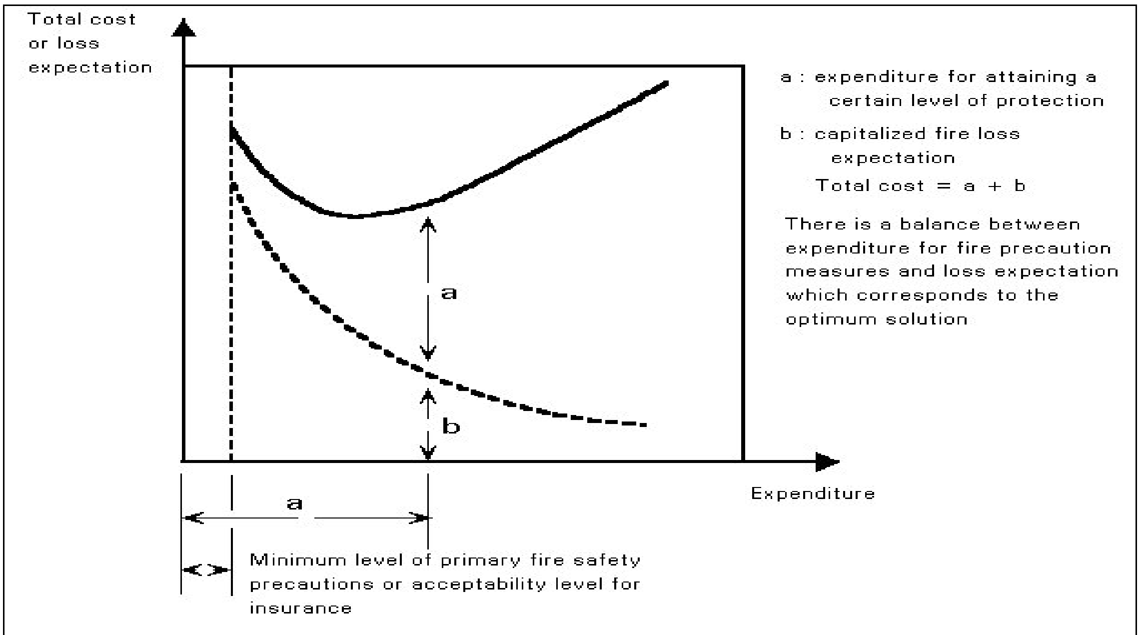


Figure 6 Relation between expenditure on preventive measures and loss expectation (schematic)

From the relation between expenditure and loss expectation it is possible to deduce the relation between expenditure and overall cost due to fire (= loss expectation + expenditure).

See the solid curve, the minimum of which corresponds to the optimum solution.

In this context it should be pointed out that in general the expenditure must not fall below a certain minimum, having regard to the requirements of life safety and/or the minimum level of acceptability for purposes of insurance. These aspects are also indicated in the figure.

Finally, attention must be drawn to the criteria by which the behaviour of the structure under fire conditions will have to be judged. In applying measures with a view to improving the fire safety of a building it will certainly be necessary to consider what the ultimate effect of such measures will be. It is known from experience that major building fires may damage the structure to such an extent that demolition of the building becomes necessary even though it has not collapsed. The money spent on protecting it from collapse will then have to be regarded as lost. In such a case it would be better either to limit the precautions merely to a level where escape of the occupants in the event of a fire is ensured, or to choose an alternative fire safety concept.

For a detailed cost-benefit-analysis a differentiated approach is necessary by calculating the annual costs of fire safety and trying to optimize them by comparison of different fire safety concepts. The basic formula is the following one:

Annual costs of fire safety =

[Sum of all investments for fire safety].[the mortgage rate in %]

+ [The repetitive maintenance costs per year]

+ [The annual premiums for the chosen fire safety concept (fire, acts of God, liability, business interruption)]

In most cases alternative concepts will show more cost-effective than structural concept.

For architects and engineers the crucial question consists in the definition of the level of fire resistance requirements they will have to fulfil, taking into consideration the global fire safety approach and the optimization of the cost-effectiveness of adequate modern fire safety concepts.

2. OVERVIEW ON ASSESSMENT METHODS OF STRUCTURAL FIRE RESISTANCE OF LOAD- BEARING ELEMENTS

Fire Resistance is governed by two basic models:

- a HEAT MODEL
- a STRUCTURAL MODEL

which normally have three to four levels of sophistication.

Traditional methods of assessment are based on the standard fire curve as far as HEAT MODELS are concerned, but more quantitive methods are available based on natural fires.

Table 4 visualises the existing three assessment methods, where:

$F_{element}$ is the fire resistance in minutes of the chosen element.

$F_{required}$ is the required fire resistance.

ASSESSMENT METHODS 1 and 2 are GRADING SYSTEMS

F-required and F-element are usually graded in catalogues or by calculation in FIRE-RESISTANCE CLASSES starting with 15 and 30 minutes and continuing by steps of 30/60/90...minutes

ASSESSMENT METHOD 3 (a + b) are ENGINEERING METHODS using models of real fire, the proof of the stability of the structure has to be shown.

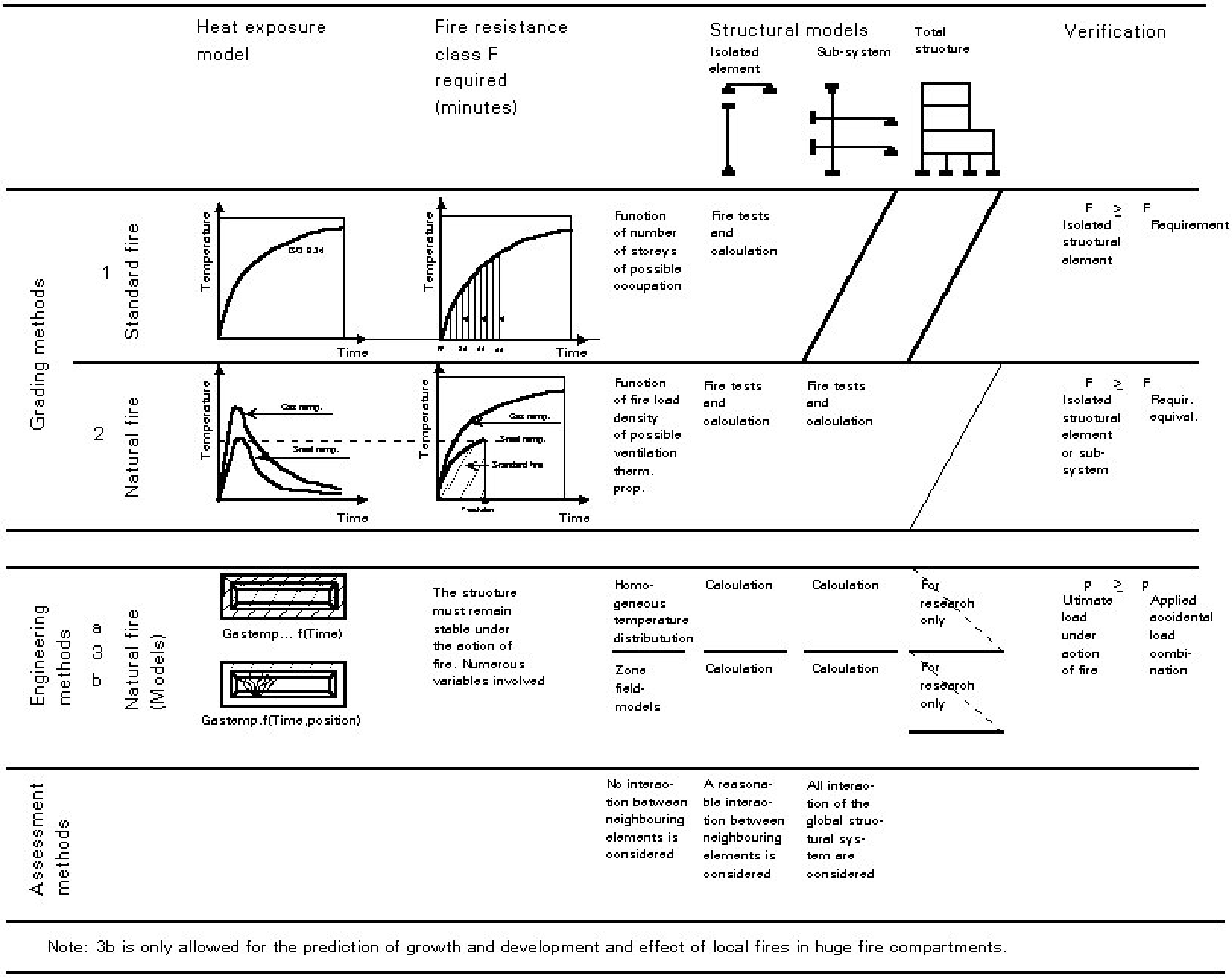


Table 4 Overview on assesment methods

Each method is discussed below together with the improvements that a closer approach to reality will bring.

2.1 Current Fire Resistance Requirements = Assessment Method 1

The term "Current Fire Resistance Requirements" is taken generally to mean the values fixed by NATIONAL CODES. They always use Fire Resistance classes (15/30/60/90...minutes) which represent the time an isolated element will resist the action of a STANDARD FIRE as defined by the heat exposure given by ISO-834. The level of requirements is a function of the number of storeys, and depending on the country, can be a function of the occupancy of the building and of the fire load.

2.2 Fire Resistance Requirements Based on T-Equivalent = Assessment Method 2

The concept of equivalent or effective fire duration provides a first but important step towards a more differentiated approach. The equivalent fire duration (T_e) is a quantity which relates a non-standard or natural fire exposure to the standard fire, in a way as is shown in Figure 7 and can be calculated if the fire load density and the ventilation conditions of the fire compartment are known.

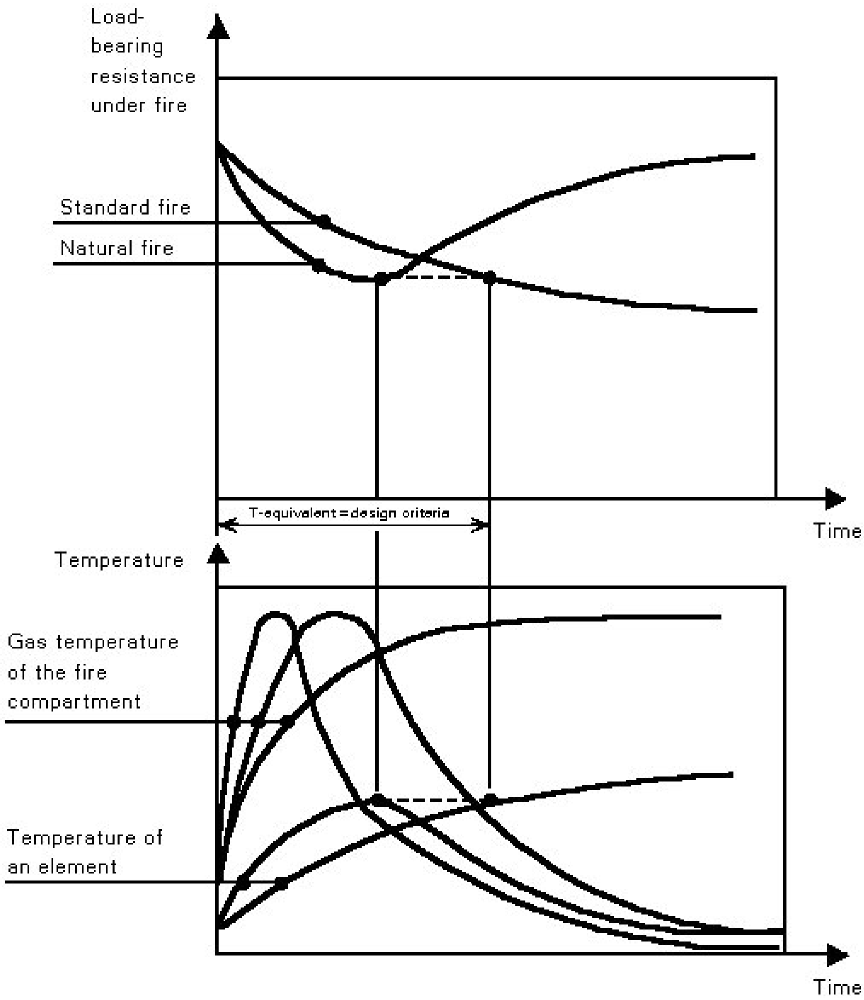


Figure 7 T-equivalent concept

In more advanced concepts of the equivalent fire duration, the effects of the thermal properties of the building components surrounding the fire compartment are accounted for.

For current occupancies and their types of fire compartments, the T-equivalent concept gives a reasonable approach to the reality of fire.

Basically the value of the required fire resistance ($F_{required}$) should be:

$F_{required} = \gamma \, . \, T_e$

where γ is a partial safety factor for model uncertainties. For normal cases γ is often taken equal to one, due to the fact that a part of the fire load burns outside the fire compartment and that the combustion is never a total one.

Normal cases $F_{required} = T_e$ for T_e calculated on the assumption of a 100% combustion of all fire loads burning inside the fire compartment.

Currently $F_{required}$ is normally fixed at the next 15 or 30 minute step (15/30/60/90...)

This method is aimed to ensure that elements designed for γ . T-equivalent will resist the action of a natural fire without collapsing even if no fire brigade action occurs.

This is the main feature of the T-equivalent concept.

An important advantage of this concept is that the tremendous amount of knowledge and data given by past fire testing can be used to verify the results of any calculation.

Many countries have already officially adopted this T-equivalent method in a more or less sophisticated way.

The design guide for Structural Fire Safety prepared by the CIB Workshop W14 [7] give the following formula for T-equivalent:

$T_e = c \cdot w \cdot q_f$

where c	is a conversion factor accounting for the effect of the thermal inertia (heat absorption) of the fire compartment.
where w	is a ventilation factor accounting for all openings (windows, doors, gaps, skylights, fanlights) which allow heat to leave and fresh air to enter the fire compartment.
where q_f	is the fire load density related to the floor area of the fire compartment. Appendix 1 of the design guide gives a detailed overview on evaluation, individual assessment and statistical values of fire load densities.

The following table gives the range of the average values for the variable fire load density in MJ/m^2 of some well-defined occupancies (combustion factor 1,0)

dwellings	330 : 780 MJ/m^2	
hospitals	100 : 330 MJ/m^2	
hotels	310 : 330 MJ/m^2	
offices	80 : 550 MJ/m^2	(excluding files, storage, library and special rooms)
schools	215 : 340 MJ/m^2	(excluding corridors, collection rooms, material, storage rooms)
shopping centres	400 : 900 MJ/m^2	(excluding USA - figures)
parking facility	200 : 300 MJ/m^2	

The fixed fire load density (fixed to the building) for these occupancies varies between approximately 50 MJ/m^2 (car park) and 450 MJ/m^2 (rooms of teachers in a school).

Globally the total fire load density may be ranged in the following classes:

LOW	less than 250 MJ/m^2 (no flashover possible):	car park
MEDIUM	500 to 1000 MJ/m^2:	dwellings, hospitals, homes, schools, offices, etc.
HIGH	1000 to 2000 MJ/m^2:	manufacturing and storage of combustible goods < 150 kg/m^2
VERY HIGH	more than 2000 MJ/m^2:	storage, warehouses

2.3 Engineering Design Methods Based on Natural Fires = Assessment Method 3

2.3.1 Introduction

These methods will only be introduced as a last and most sophisticated method of defining the correct level of Structural Fire Resistance.

Modern computer-assisted calculation methods are available which allow any Heat Exposure Model to be introduced.

In any case the reality of fire must be introduced through more or less simplified Heat Exposure Models. Two types of models are currently used:

- The compartment fire model with a uniform temperature distribution in the fire compartment after the occurrence of flashover.
- Models with non-uniform temperature distribution in the fire compartment (Zone and Field Models)

All these engineering methods are based on an improved Heat Model in connection with an improved Structural Model. The verification consists in proving that the structure remains stable under the action of a real fire for the loads present at the time of the fire.

It is emphasized that with slight modifications Assessment Method 3 can also be used for buildings where only a limited time period, long enough to provide time for a safe escape and rescue, is required.

The interest in these engineering fire design methods will certainly be awakened by the new generation of Eurocodes [10, 11] introducing fire as an accidental situation.

2.3.2 Compartment fires = assessment method 3a

This method applies for fire compartments of a size usually found in hotels, offices, schools, dwellings, etc. with an equal distribution of the fire load. The assumption of a uniform distribution of temperature in the fire compartment is then correct. This method introduces the following main variables:

- The amount of equally distributed combustible materials in the fire compartment = mean fire load density (fixed and mobile);
- The combustion rate of variable combustible materials;
- The geometry of the fire compartment;
- The ventilation of the fire compartment;
- The thermal response of walls and floors enclosing the fire compartment.

Some variables may be approximated or even ignored. Two variable will always have a strong influence.

- The fire load density.
- The ventilation of the fire compartment.

The influence of fire load density and ventilation of compartment gas temperature is illustrated in Figures 8a and 8b. They correspond to a simplified compartment fire theory as a basic heat exposure model for engineering fire design. Current compartment fire theories neglect the pre-flashover period, the structural response being mainly governed by post flashover temperature evolution.

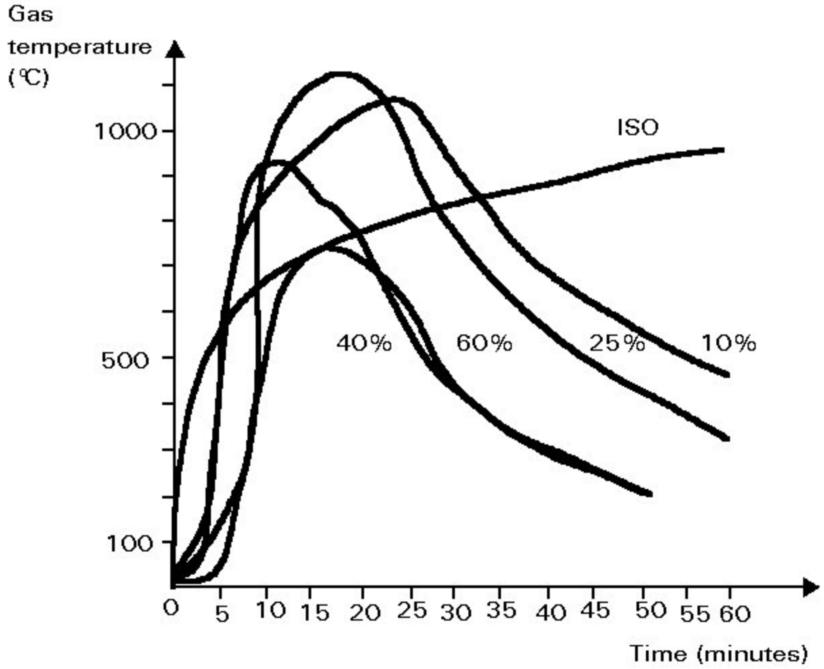


Figure 8a Evolution of gas temperature as a function of ventilation (percentage area of openings to total area of facade)

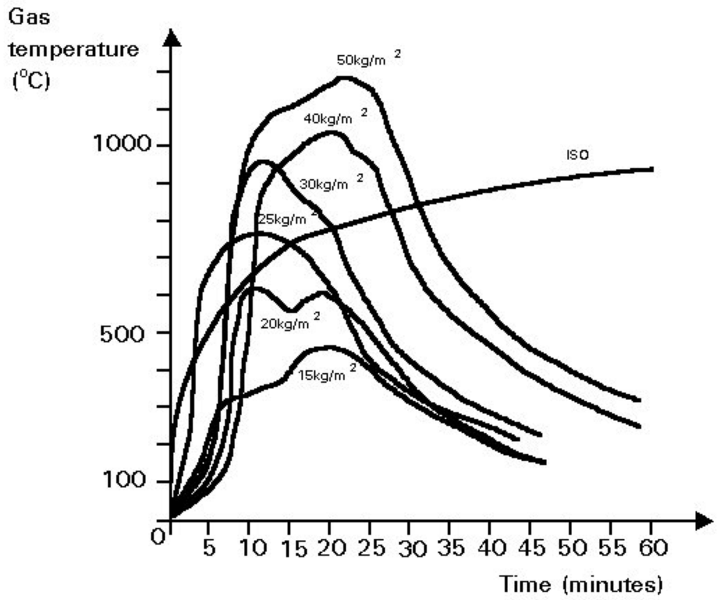


Figure 8b Influence of fire density on gas temperature (Ventilation factor $f_v = 0{,}091 m^{-1/2}$)

2.3.3 Fire modelling - assessment method 3b

These methods try to evaluate the evolution of fire as a non-uniform problem where for a given compartment and a known localized fire load temperature will be governed by

- The location of a local fire
- The growth of such a local fire
- The size, geometry, ventilation and thermal inertia of the fire compartment.

Therefore the temperature evolution will be a function of:

- Time, and
- Location of a give structural element in this compartment.

These methods must be calibrated. International tests have been carried out either in large fabrication halls (CTICM France...) or in test facilities with large compartments (Finland/Espoo) which allowed the temperature evolution of natural fires to be measured at different points. These methods are useful for all cases of localized fire in large compartments or large spaces.

2.4 Some Thoughts on Fire Resistance Requirements Considering the Effect of Active Fire Protection

If it is possible to avoid the occurrence of any flashover situation by means of adequate active measures (automatic detection, adequate fire brigade, sprinklers; CO_2, Halon), the structure will never be endangered by fire.

So independently of any assessment method the fire resistance requirements may be reduced or mostly zero rated, as long as the probability of success of the active measures is high enough.

In fact an increasing number of countries like Sweden, Switzerland, and Germany allow for reduced or no Fire Resistance Requirements when the probability of avoiding flashover or of localising a fire in a small area is high enough. Some other countries are moving to this direction. These alternative concepts will normally be limited to occupancies which will not undergo significant change of use and to buildings with a limited number of storeys. The major argument brought forth against these Alternative Concepts refers to the reliability of the active measures in the sense that, if they fail to suppress an initial fire, then a reduced fire resistance of the structure could exhibit a considerable hazard. We should however consider the risks of failure case by case.

The fire modelling assessment method is an appropriate way to prove the effectiveness of alternative fire safety concepts. In fact the dependency of structural fire requirements on potential structural hazards is uncritically accepted whilst the dependency on non-structural measures (governing the frequency of severe fires) is often not generally acknowledged as a design parameter. Fire modelling will ultimately allow the influence of extinguishing actions (automatic devices such as Sprinklers and fire brigade actions) to be quantified and incorporated into the assessment.

For the Assessment Method 2 (T-equivalent-method), the CIB Design Guide for Structural Fire Safety proposes to multiply the value of t_e given under point 2.2. of this paper by a differentiation factor accounting for special fire-fighting provisions (active measures). This differentiation factor will vary in function of the safety level, effectiveness and liability of the chosen special fire-fighting provision and is always lower than the unity.

3. CONCLUDING SUMMARY

- The objectives of fire safety are to reduce the loss of life and property or financial losses in, or in the neighbourhood of, building fires.
- For this purpose fire safety concepts are used which are packages of integrated structural, technical and organisational fire precaution measures agreed by the owner, the fire authority and the designer.
- Different fire safety concepts are available including structural, monitoring , extinguishing concepts.
- The definition of the level of fire resistance requirements by architects and engineers takes into account the global fire safety approach used and the cost-effectiveness of modern fire safety concepts.
- Several types of assessment method for structural fire resistance are used, i.e. grading systems and engineering methods.

SOURCE http://www.kuleuven.be/bwk/materials/Teaching/master/wg04b/l0400.htm

Lecture 2 Background to Thermal Analysis

OBJECTIVE/SCOPE

To introduce a basic background of thermal analysis for fire situations.

SUMMARY

Thermal models are presented and simple rules are given for calculating the transient thermal response of steel elements, with or without a protective coating. The concept of the section factor of the steel section is introduced. Composite steel-concrete elements (columns and slabs) are also discussed.

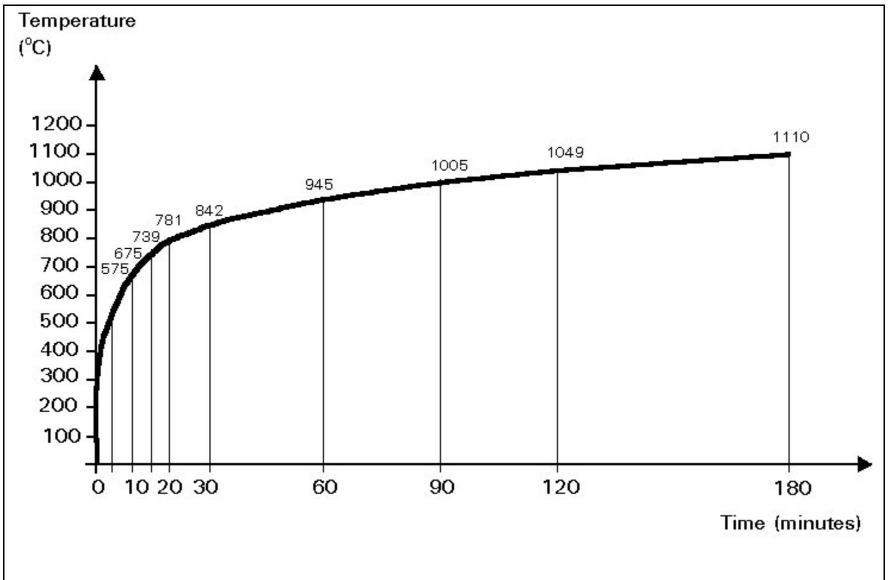
1. INTRODUCTION

Fire is a very complex phenomenon which can take many forms and involves different kinds of chemical reactions.

From a structural point of view, only the fires that can cause structural damage are of interest and, in this case, fire can be regarded as an accidental situation.

Design criteria for structural fire safety require some assumptions both for the structural and heating models.

Fire is usually represented by a temperature-time curve which gives the average temperature reached during fire in a small size compartment or in the furnaces used for fire resistance tests. International standards are based on the standard fire defined by the heat exposure given by the ISO 834 curve (Figure 1). In some cases reference can be made to natural fires which have different temperature-time relationships depending on fire load density and ventilation conditions (Figures 2a and 2b.



.Figure 1 ISO standard fire curve

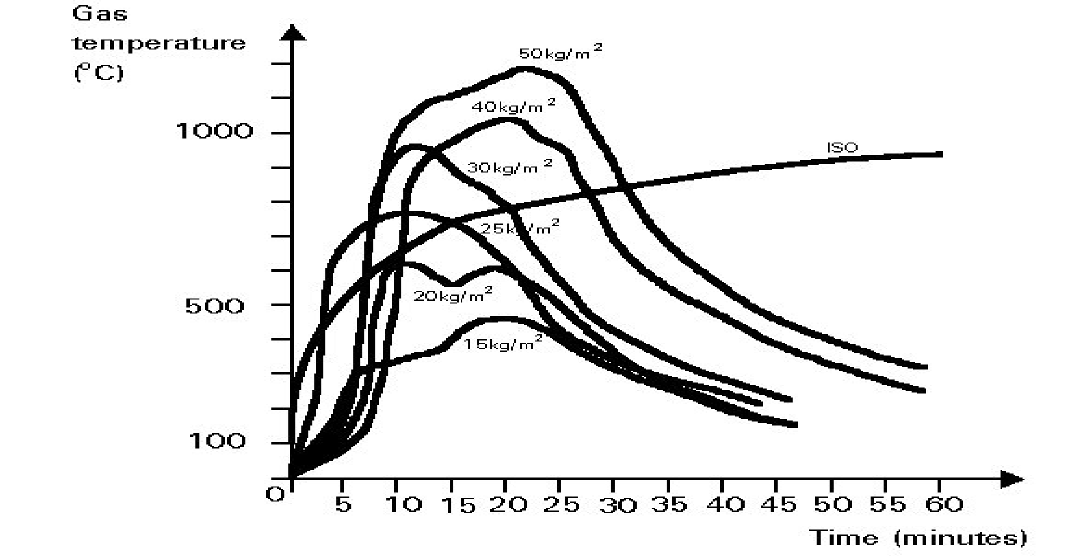


Figure 2a Evolution of the gas temperature for different fire load densities (Ventilation factor $f_v = 0{,}091\ m^{1/2}$)

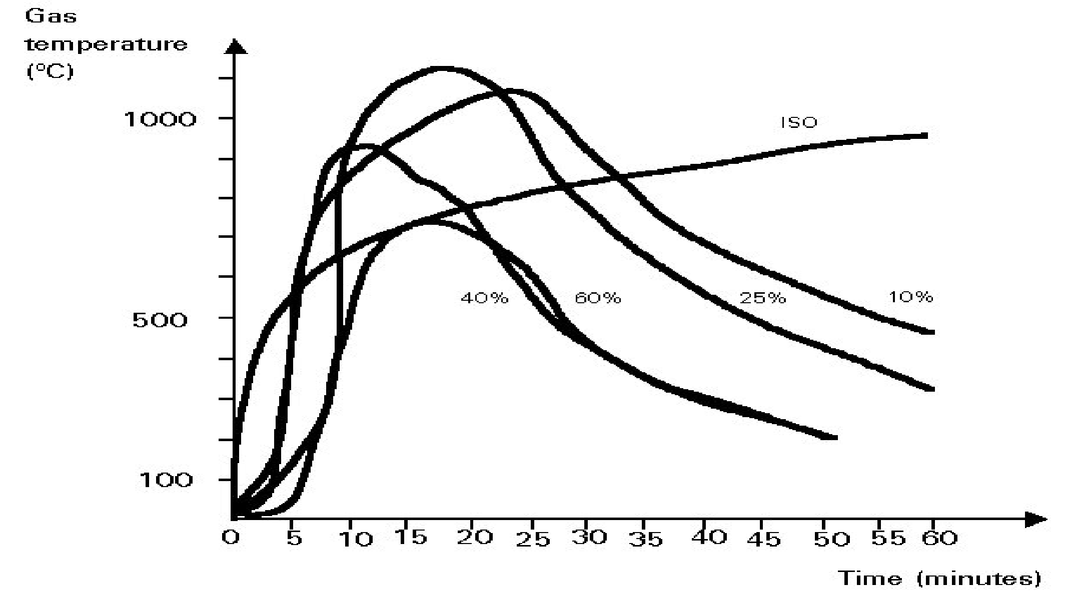


Figure 2b Evolution of gas temperature as a function of the ventilation (percentage of area of openings to total area of facade) for a fire load $q = 30\ kg/m^2$ of wood surface area

In more complex analyses different heating models can be considered to represent the temperature development in different zones of the fire compartment or in the neighbourhood of it. This is the case, for instance, for many large industrial buildings or for external columns near to the windows of a building.

The response of a structural member exposed to fire is governed by the rate that it is heated because the mechanical properties of materials decrease as temperature rises and, likewise, the structural resistance of a member reduces with temperature rise.

Collapse occurs at the time when the structural resistance reduces to the applied action effects. This fire resistance time can happen in a very short time when the increase of temperature is rapid. Steel elements have an unfavourable behaviour in this respect due to the very high thermal conductivity of the steel. A rapid heating of the whole profile takes place as a result. In comparison, composite elements have a favourable behaviour due to the great thermal inertia of the elements and the low thermal conductivity of the concrete.

In this lecture some basic aspects of thermal analysis are discussed. The general equation for heat transfer is presented, followed by the simplified method which may be adopted for steel members. Thermal gradients across the section and along the member are neglected.

In a fire, the temperature of the steel increases similarly but with some delay compared to the gas temperature of the fire (Figure 3). The delay depends on the thermal inertia of the element as well as on the intensity of heat flow passing through its external surface. If the element has an applied protective coating, this delay is longer. For bare elements the delay depends on the section factor of the element.

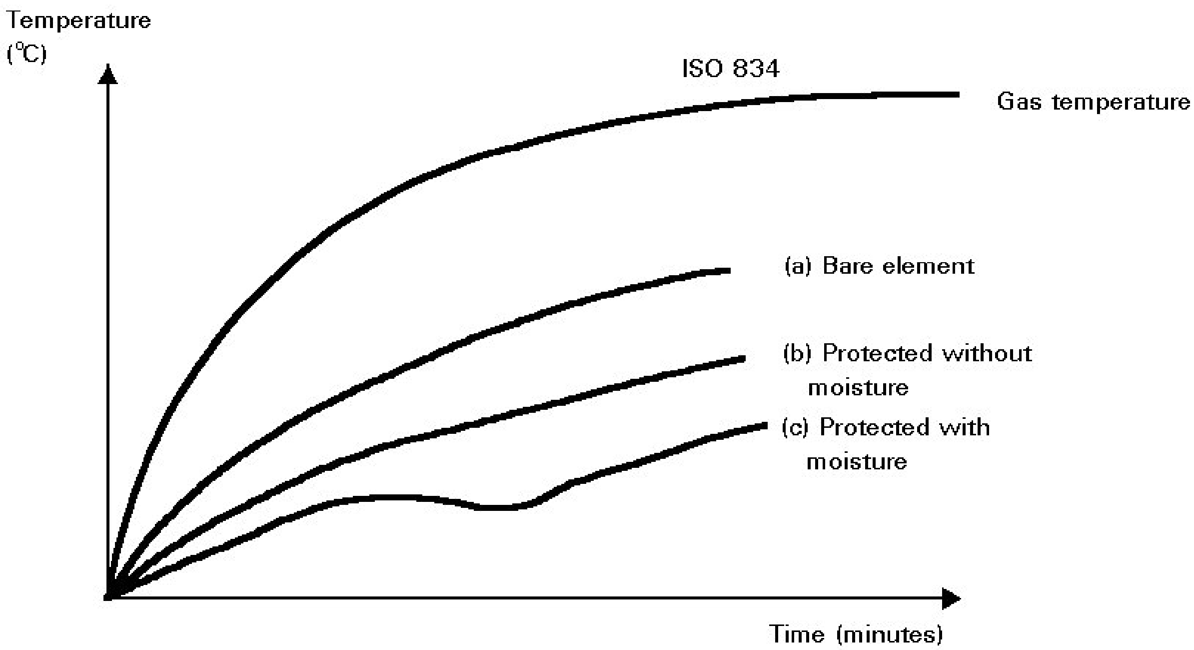


Figure 3 Influence of insulation on the heating rate

In Figure 3, the temperature rise in three different cases is compared for the same element. Curve (a) represents the delay for the bare element, while curves (b) and (c) apply to the cases of some protective coating, without and with moisture content.

2. HEAT TRANSFER EQUATION

The rise of temperature in a structural steel member depends on the heat transfer between the fire environment and the element.

According to the second law of thermodynamics, energy in the form of heat is transferred between any two elements which are at different temperatures. Conduction, radiation and convection are the modes by which thermal energy flows from regions of high temperature to those of low temperature.

On the external surface of building elements all three mechanisms are present. Inside the elements, heat is transferred from point to point only by conduction.

The general approach to studying the increase of temperature in structural elements exposed to fire is based on the integration of the Fourier heat transfer equation for non-steady heat conduction inside the member. The integration of this equation gives the energy balance between the net rate of heat flow into the element through its faces, the heat flow in the element per unit time and unit volume and the rate of change of internal energy. The change in internal energy causes the change in temperature.

The solution of this equation can be obtained when the initial and boundary conditions are known.

For fire, the initial conditions consist of the temperature distribution at the beginning of the analysis (usually the room temperature before fire); boundary conditions must be defined on every surface of the structure.

Usually fire simulations are based on the temperature history of the fire, for instance the standard fire curve of ISO 834. However, any other fire conditions can be assumed, using an input time-temperature history for the fire.

Numerical methods are necessary to solve the heat flow equation. Many computer programs are available and it is now possible to carry out thermal analysis for very complex structural elements.

In many cases, the general form of the equation can be greatly simplified. For instance, thermal conductivity, density and specific heat can be assumed to be independent of temperature; internal heat generation is absent or can be neglected; and three-dimensional problems can be studied as two-dimensional or one-dimensional idealizations.

3. HEATING OF STEEL SECTIONS

Since no heat is generated within the body of steel elements and since the material is isotropic, the Fourier heat transfer equation is:

$$\frac{k_s}{\rho_s c_s} \Delta^2 \vartheta = \frac{\partial \vartheta}{\partial t} \quad (1)$$

The quantity $k_s / \rho_s c_s$ is known as the thermal diffusivity and varies with the temperature.

The specific mass of steel (ρ_s) can be considered independently from the temperature (ρ_s=7850kg/m^3); while the thermal conductivity k_s and the specific heat c_s are dependent of the temperature (Figures 4 and 5) but for a simplified calculation it is possible to make reference to constant values (c_s=520 J/kg°C and k_s = 45 W/m°C, for all grades of steel).

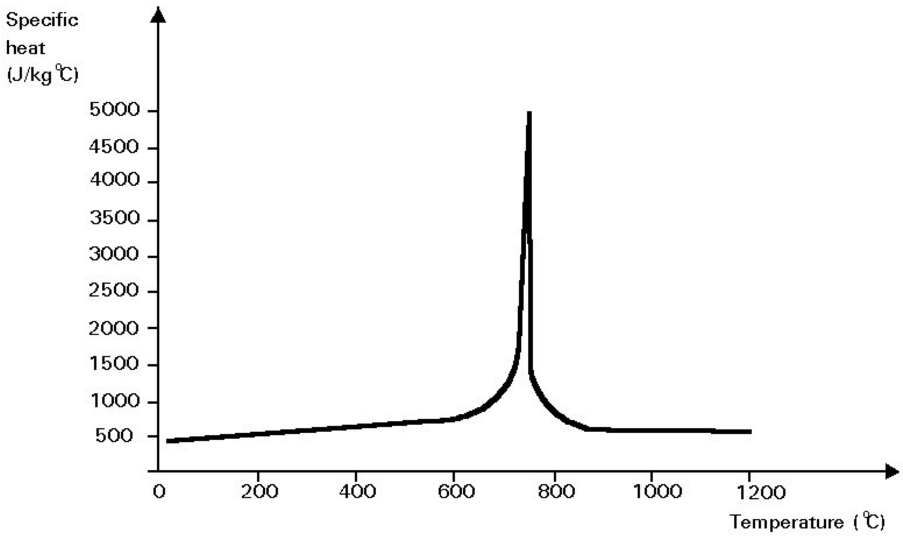


Figure 4 Specific heat of steel as a function of the temperature

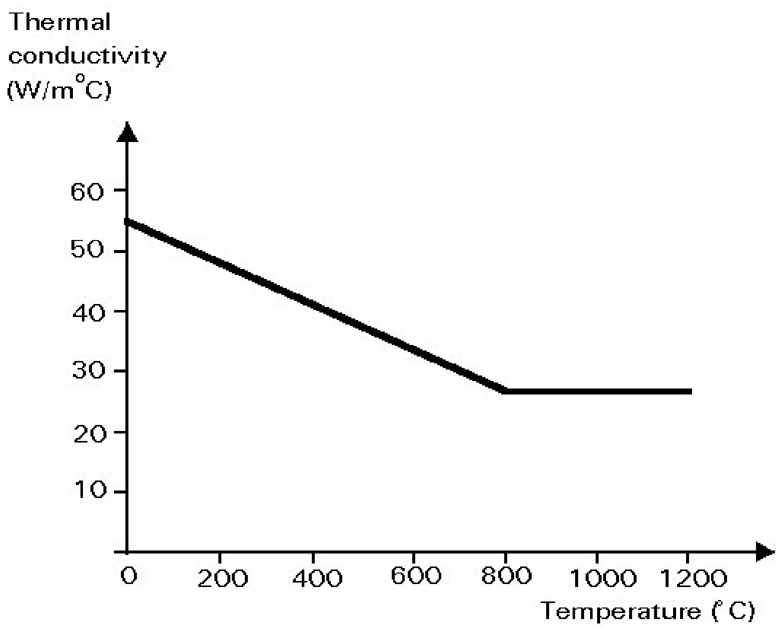


Figure 5 Thermal conductivity of steel as a function of the temperature

The solution of the thermal transient can be obtained by numerical methods as in the general case; but thermal conductivity is high enough to allow differences of temperature in the cross-section to be neglected.

This assumption means that thermal resistance to heat flow is negligible. Any heat supplied to the steel section is considered to be instantly distributed to give a uniform steel temperature. With this assumption the energy balance can be made not only with reference to an infinitesimally small element, but also to the whole section of the exposed steel element (Figure 6).

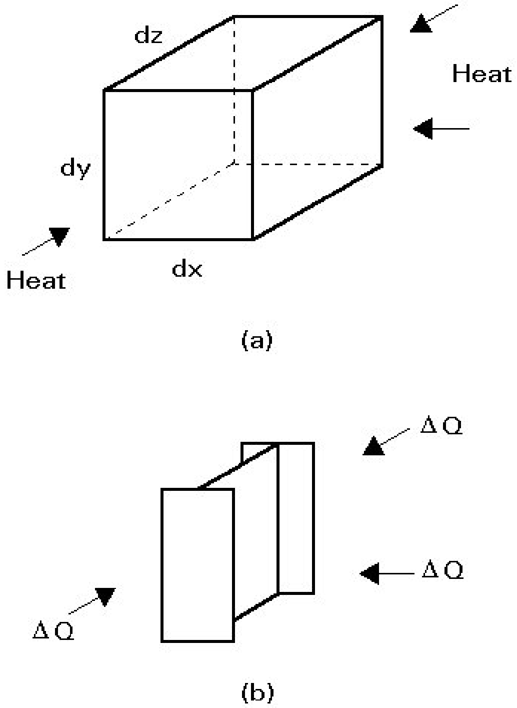


Figure 6 Elements for heat flow analysis
(a) element in continuum,
(b) length of steel section

The quantity of heat transferred per unit length in the time interval Δt is:

$\Delta Q = K \cdot A_m \cdot (\theta_f - \theta_s) \cdot \Delta t$ (2)

where:

K is the total heat transfer coefficient ($W/m^{2\circ}C$)

A_m is the perimeter surface area per unit length exposed to fire (m^2/m)

θ_f is the temperature of hot gases (°C)

θ_s is the temperature of steel during the time interval Δt (°C)

If this quantity of energy is entirely absorbed by the section, i.e. no loss of heat is considered, the internal energy of the unit length of a steel element increases by the same quantity:

$\Delta Q = c_s \cdot \rho_s \cdot A \cdot \Delta\theta_s$ (3)

where:

A is the cross-sectional area of the member (m^2).

The temperature rise of the steel is given by combining Equations (2) and (3) as follows:

$\Delta\theta_s = [K/(c_s/\rho_s)].[A_m /A].(\theta_f - \theta_s).\Delta t$ (4)

Solving this incremental equation step by step gives the temperature development of the steel element during the fire. To assure the numerical convergence of the solution some upper limit must be taken for the time increment Δt. In Eurocode 3 Part 1.2 [1] it is suggested that:

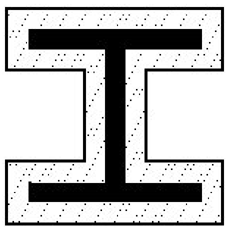
$$\Delta t \leq \frac{2{,}5.10^4}{A_m/A}$$

where:

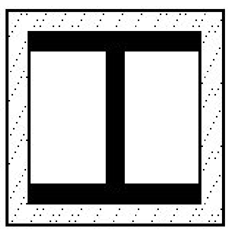
Δt is in seconds

A_m /A is in m^{-1}

It is apparent that an important parameter in determining the rise of temperature of the steel section is A_m /A. This is often known as the "section factor" (sometimes given as F/V, or A/V, or H_p /A in different countries), see Figure 7.



(1) Contour encasement or spraying $A_m/A = \dfrac{\text{Perimeter of steel section}}{\text{Steel cross-section}}$



(2) Hollow encasement $A_m/A = \dfrac{\text{Interior perimeter of encasement}}{\text{Steel cross-section}}$

Figure 7 Section factors for insulated sections

When the profile is in contact with another element, for example, a concrete slab, which has a thermal conductivity greatly lower than the thermal conductivity of the steel, the effective exposed perimeter A_m must be calculated taking into account only the part of the surface directly exposed. This requires an assumption of an adiabatic condition at the contact surface. The result is a safe solution: in fact some thermal energy passes through the colder body and, if it is neglected, the increase of the temperature in the steel element is higher.

It is very important to understand this point, because it gives the key to deciding if the simplified solution of the thermal problem is appropriate or if it is necessary to solve the complete heat transfer equation.

For instance it is appropriate to consider the thermal gradients in the steel cross-section or the heat flux transmitted from the steel to the concrete where the concrete slab is supported by a profiled steel sheet or in composite elements (Figures 8 and 9). In this case a finite element model can be used.

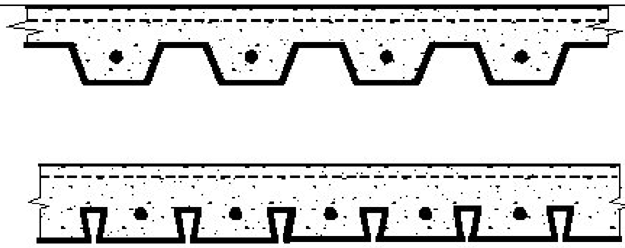


Figure 8 Typical cross-sections of composite steel-concrete floors

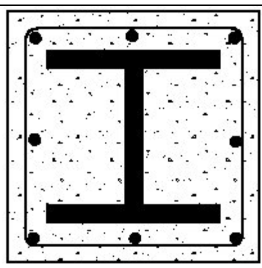


Figure 9 Cross-section of column with encased H-steel profile reinforced with longitudinal steel bars

Eurocode 3 Part 1.2 permits many practical problems to be solved in a simplified way [1].

Heat transfer information is presented for bare elements, as well as for protected elements. Two types of coating are considered for protected elements: dry insulation materials, and materials containing a significant amount of moisture.

The ECCS publication 'European Recommendations for Fire Safety of Steel Structures' [2] also gives a simplified formula which expresses the relationship between the time, t, of exposure under a standard fire (expressed in minutes), the critical temperature $\theta_{s,cr}$ of the element, the section factor A_m /A and the properties of the insulation materials, their thickness d and their thermal conductivity λ_i.

For unprotected elements the equation is:

$t = 0{,}54\ (\theta_{cr} - 50)\ (A_m /A)^{-0{,}6}$

It can be solved in two ways:

$\theta_{cr} = 1,85\ t\ (A_m / A)^{0,6} + 50$

or

$A_m / A = 0,36\ [(\theta_{cr} - 50)/t]^{1,67}$

and is valid within following ranges:

t = 10 to 80 min

θ_{cr} = 400 to 600°C

Similarly, for sections protected by a light insulation material, the equations are:

$t = 40(\theta_{cr} - 140).[dA/\lambda_i A_m]^{0,77}$

or

$d = 0,0083\ [t/(\theta_{cr} - 140)]^{1,3}\ .[A_m/A].\lambda_i$

In the above equation d is the protection thickness (in metres) and λ_i is the thermal conductivity of the material (in W/m°C).

These equations can be expressed also in a nomogram which is very practical for design purposes (Figure 10).

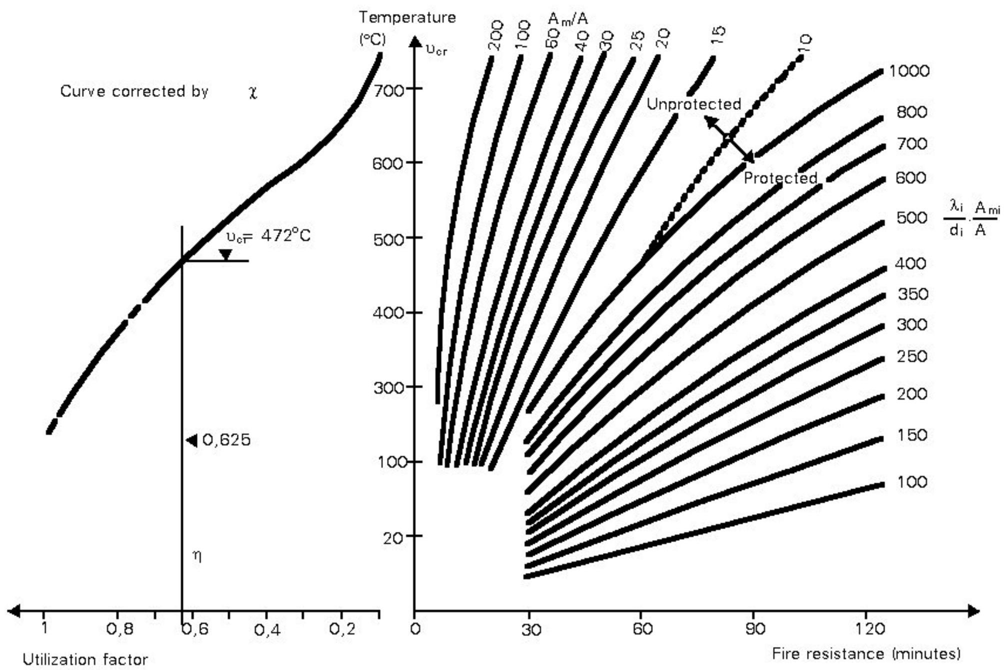


Figure 10 Relation between the utilization factor, section factor and fire resistance

Equation (4) shows the three principal factors on which the increase of temperature of steel depends: the total heat transfer coefficient the ratio between the exposed perimeter of the element A_m and its cross-section A; and the difference between the temperatures of the hot gases and the steel element.

The total heat transfer coefficient, K, depends on the heat transfer coefficients for convection and radiation, α_c and α_r, and, if there is a protective coating, on the thermal conductivity of the protective material and its thickness, such that:

$$K = \{1/(\alpha_c + \alpha_r) + d/\lambda_i\}^{-1} \qquad (5)$$

For the usual condition of fire convection (as in a fire test furnace) the value of α_c can be assumed:

$$\alpha_c = 25 \text{ W/m}^{2\circ}\text{C} \ (6)$$

while α_r can be calculated by the expression:

$$\alpha_r = 5{,}77\varepsilon_r \ \{(\theta_f + 273)^4 - (\theta_s + 273)^4\}\times 10^{-8}/(\theta_f - \theta_s)$$

where ε_r is the resultant emissivity of the flames, combustion gases and steel surface. Its value can be assumed according to Eurocode 3 [1], i.e.

$$\varepsilon_r = 0{,}5$$

The value of the section factor (A_m /A) can vary over a very large range. The rate of temperature rise in a small thick section will be slow, whilst in a large thin section it will be more rapid.

The differences between section factors are the principal reason for the different behaviour of different steel elements exposed to fire. If the thermal inertia is larger, the increase of temperature is slower and the fire resistance is higher under the same loads as a result.

Values of section factors can be found in many publications. It must be noted that the section factor A_m /A represents the ratio of the effective surface exposed to fire to the volume of the element. Where there is a protective coating, the surface to take into account is not the external surface of the profile, but the inner as, for instance, in the case of boarded encasement (Figure 7).

4. THERMAL RESPONSE OF COMPOSITE STEEL - CONCRETE ELEMENTS

4.1 Introduction

Two different composite elements are to be considered; composite columns and composite slabs.

For composite columns (Figure 11), a distinction is made between:

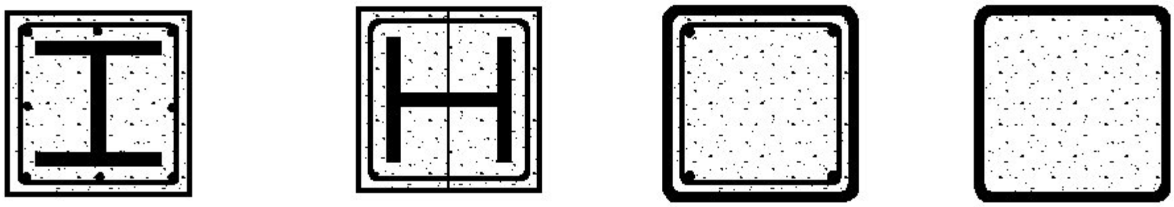


Figure 11 Typical cross-sections of composite columns

a. rolled I-profiles encased in concrete

b. rolled I-profiles with concrete between the flanges

c. concrete filled steel sections with or without reinforcement.

For composite slabs, the discussion is limited to composite concrete slabs with profiled steel sheet. Some typical cross-sections are shown in Figure 8.

The rules are based on the ECCS-Technical Notes for the calculation of the fire resistance of composite columns and composite concrete slabs with profiled steel sheet, exposed to the standard fire [3, 4]. These Technical Notes provided for designers reflect the present state of knowledge based on recent research results.

4.2 Thermal Response of Composite Columns

ISO standard fire exposure on all sides of the column is taken as the starting point. A uniform temperature distribution is assumed over the height of the column. Under practical fire conditions, however, a significant non-uniform temperature distribution in the concrete over the cross-section must be expected, as shown in Figure 12 for a concrete filled steel column.

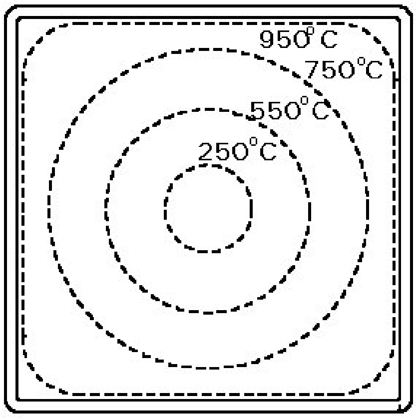


Figure 12 Measured temperature distribution over the cross-section of a concrete filled steel column (200x200x6.3 m³) after 90 minutes standard fire exposure on four sides

As a consequence, two dimensional heat flow models must be used. The calculation of the temperature field over the cross-section is only possible by means of a computer. In practice this means that a separate thermal analysis must be made for all relevant cross-sections.

4.3 Thermal Analysis of Composite Slabs

Composite slabs have not only a load bearing function but also a separating function in comparison to many traditional building components. As a result the insulation and integrity criteria should be considered when determining the fire resistance of composite slabs. For composite steel-concrete floors the integrity criterion is not difficult to fulfil. Normally the floor slab is cast in situ producing joints which are adequately sealed. Cracks which may occur in the concrete during fire exposure are unimportant because the steel sheet will prevent penetration by flames and hot gases. For these floors an explicit check on integrity is generally not necessary.

The following discussion considers the analysis for the criteria of insulation. Rules are presented for determining the temperature distribution as far as such information is necessary to evaluate the load bearing resistance. The rules concentrate on the required additional reinforcement, since without such reinforcement the fire resistance of composite slabs is only about 30 minutes.

As for composite columns, two-dimensional heat flow models are necessary for concrete slabs with profiled steel sheets. These models are however too cumbersome for every day design. To overcome this problem, the profiled slab is schematized to a flat slab with an effective thickness equal to a weighted average of the real slab thickness. For various periods of standard fire exposure, temperature distributions can then be determined. From such distributions a rule for minimum slab thickness necessary to fulfil the criterion of insulation can be derived. This rule is given in Figure 13 together with some test results. It is seen that conservative solutions are obtained.

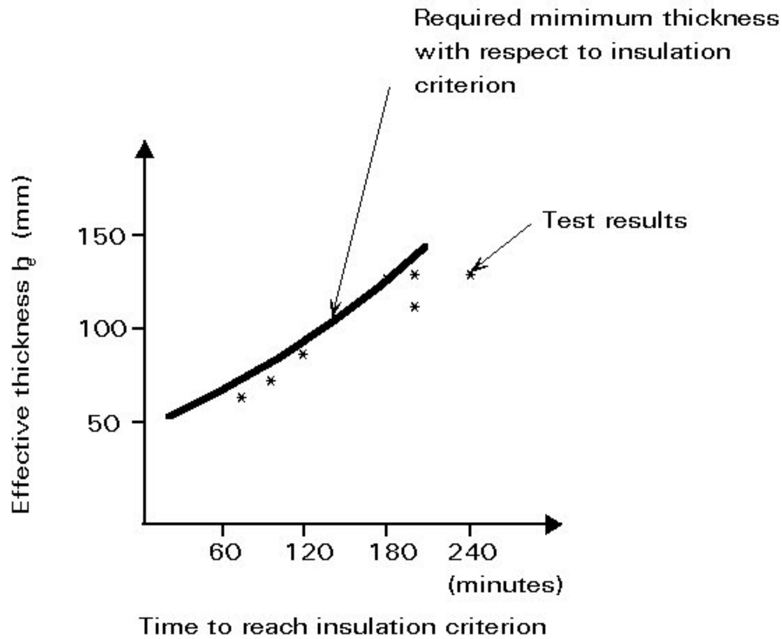


Figure 13 Minimum effective slab thickness to meet the insulation criterion (Δt-140 °c)

The temperature of the additional reinforcement plays a crucial role in structural analysis. Therefore, experimentally determined equations have been established from which this temperature may be calculated as a function of the position of the reinforcement bar in the slab, (given by u_1, u_2, u_3) and the period of standard fire exposure, t. Figure 14 shows the results of some validation tests.

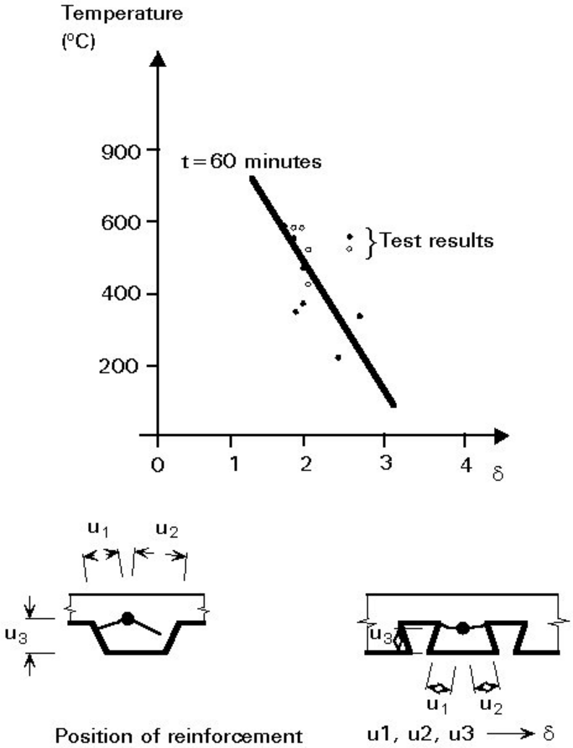


Figure 14 Temperature in the additional reinforcement

5. CONCLUDING SUMMARY

- The response of a structural member exposed to fire is governed by the heating rate of the element which is directly related to the section factor of the element.
- The general approach to study the increase of temperature in structural elements is based on the Fourier heat transfer equation. The general solution of the heat transfer equation is possible by means of computer programs.
- In numerous practical cases simplified solutions can be used to find the temperature reached by steel profiles under standard fire exposures.

SOURCE http://www.kuleuven.be/bwk/materials/Teaching/master/wg04b/l0400.htm

Lecture 3 Background to Structural (Mechanical Fire) Analysis

OBJECTIVE/SCOPE

Demonstration of calculation of load bearing resistance of structural elements submitted to an increase of temperature. Guideline for the calculation of the fire rating of steel and composite elements.

SUMMARY

The failure resistance of a structural element subjected to fire is calculated from the applied load during the fire and plastic theory. Differentiation is made between bending elements and axially loaded elements and between uniformly heated sections and sections with thermal gradients. The main factors influencing stability in fire are presented. Structural analysis of composite columns and composite slabs is also discussed.

1. INTRODUCTION

The increase of the temperature of steel and concrete in composite steel-concrete elements, leads to a decrease of mechanical properties such as yield stress, Young's modulus, and ultimate compressive strength of concrete (Figures 1 - 4) Thus, when a steel or a composite structure is submitted to a fire action, its load bearing resistance decreases. If the duration and the intensity of the fire are large enough, the load bearing resistance can fall to the level of the applied load resulting in the collapse of the structure (Figure 5). This state is illustrated by:

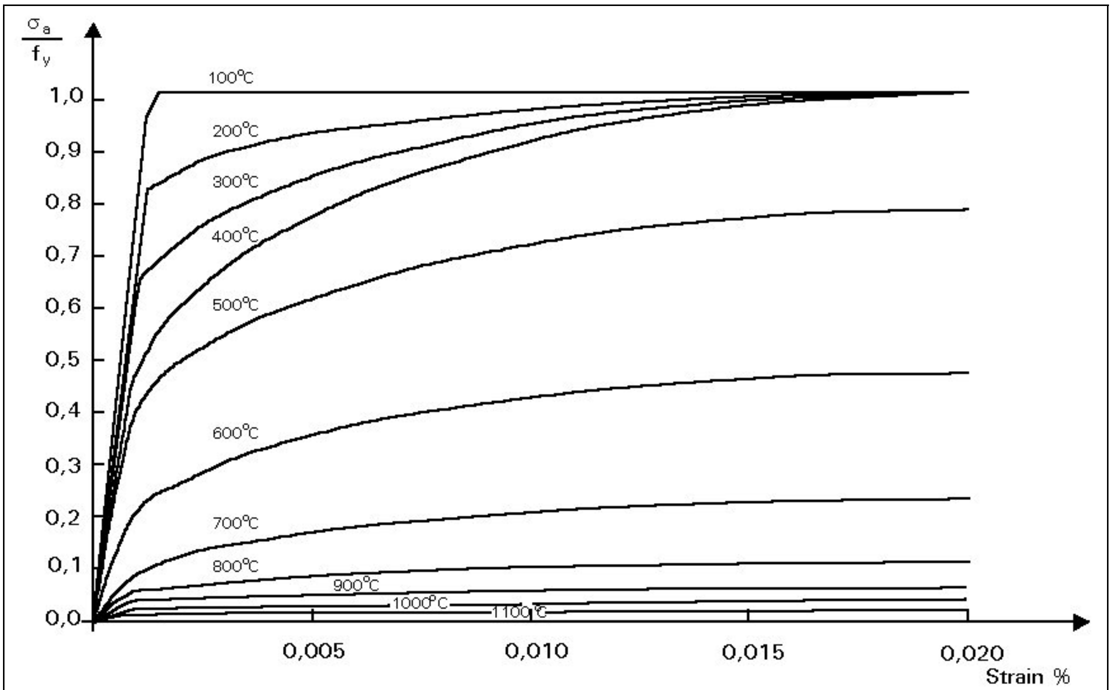


Figure 1 Stress-strain relationship of steel

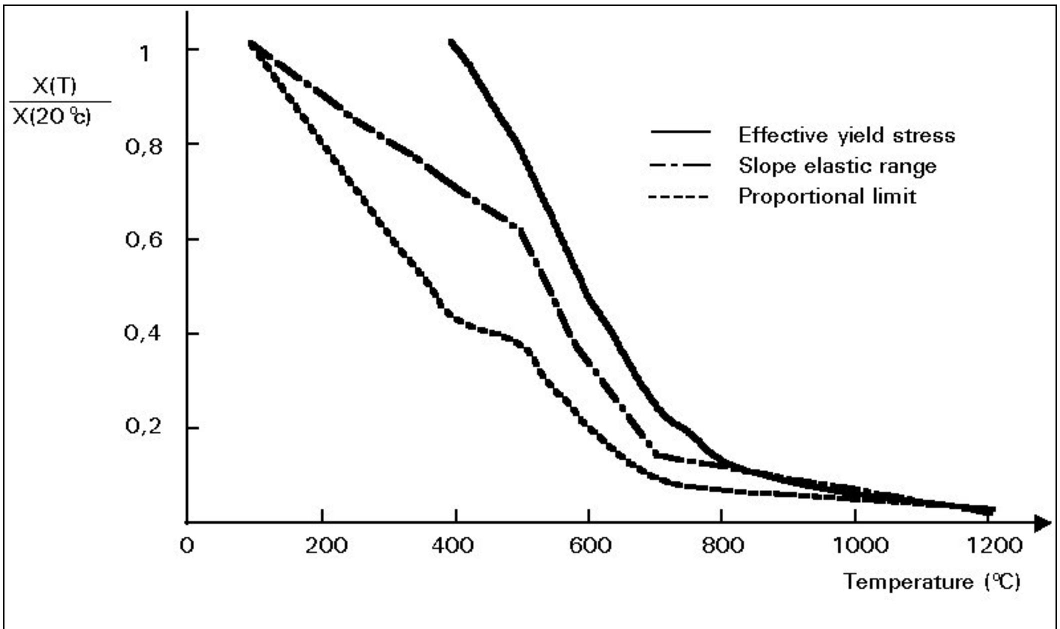


Figure 2 Parameters of structural steel at elevated temperature

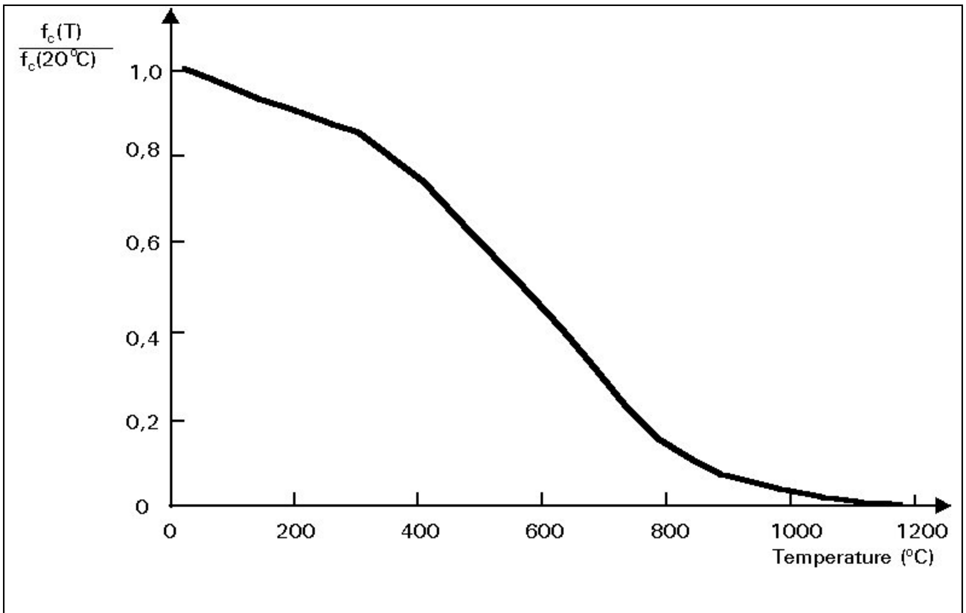


Figure 3 Compressive resistance of concrete

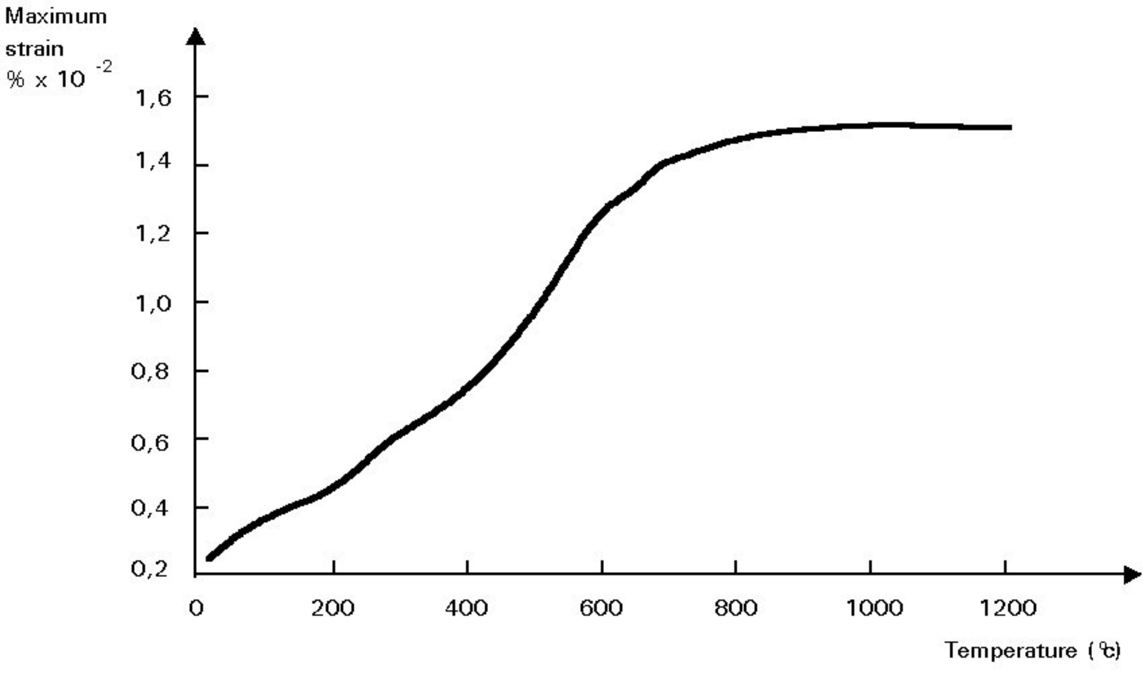


Figure 4 Maximum strain of steel.

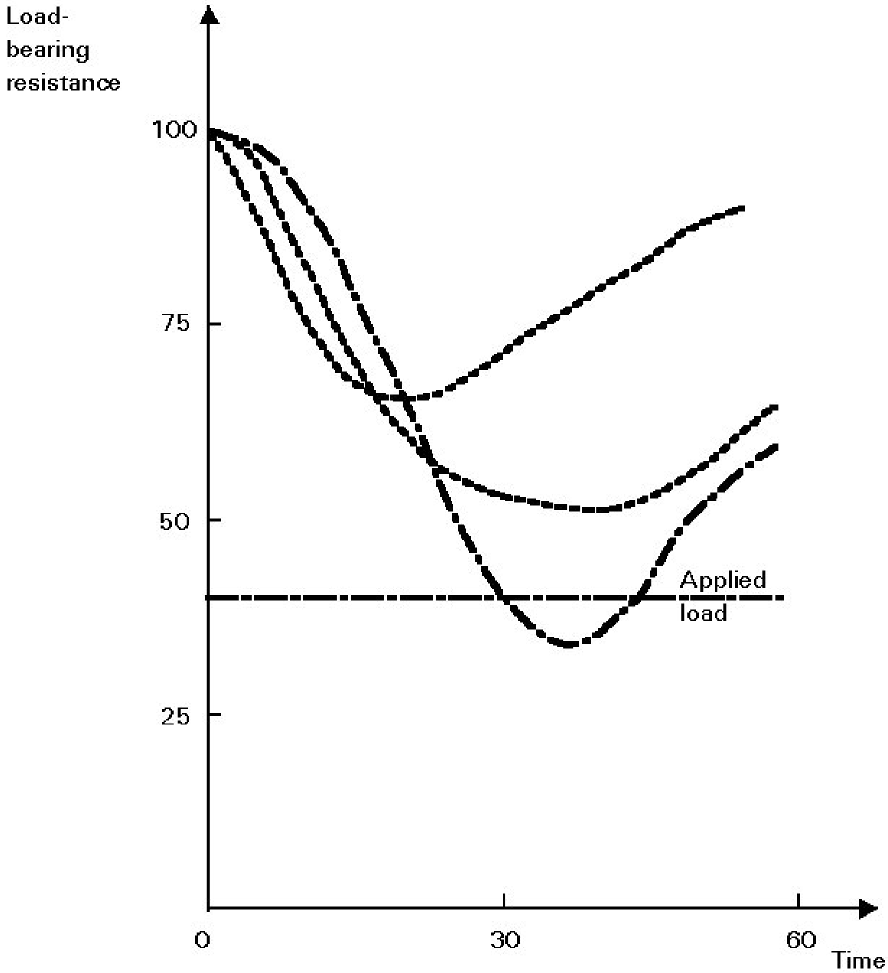


Figure 5 Examples of decrease of load-bearing resistance of a structure during exposure to fire.

$P_u\ (\theta_{cr}) = P$

where:

P is the applied load in fire conditions

$P_u\ (\theta_{cr})$ is the load bearing resistance for a temperature θ_{crit} (the critical temperature), and

P_u is the load bearing resistance at room temperature.

The objective of this lecture is to give the background for structural analysis of this situation.

2. APPLIED LOAD

The applied load is obtained by considering the accidental combination of the mechanical actions such as: dead load, live load, wind (only for bracing), snow.

Due to the low probability that both fire and extreme severity of external actions occur at the same time, only the following accidental combinations are considered:

$1{,}0\ G_K + \psi_1\ Q_{K,1} + \Sigma\ \psi_{2,i}\ Q_{K,i}$

where:

G_K is the characteristic value of permanent actions

$Q_{K,1}$ is the characteristic value of the main variable actions

$Q_{K,i}$ is the characteristic value of other variable actions

ψ_1 is the frequent value of the main variable actions

$\psi_{2,i}$ is the average of the other variable actions.

Generally, in fire: $\psi_1 = 0{,}5$ and $\psi_{2,i} = 0$

Apart from bracings, $Q_{K,1}$ and $Q_{K,2}$ generally correspond to imposed loads and snow loads.

3. DETERMINATION OF THE FIRE LOAD BEARING RESISTANCE

The calculation of the load bearing resistance of a structure submitted to fire can be made in several ways depending on the kind of structure and the requirement for the duration of stability in fire.

The simplest method of calculation is an analysis in which the structure is represented by individual members considered directly exposed to fire. In such calculations, support and boundary conditions should be assumed as for normal conditions of use. External forces and moments on the structural member are deduced from a global structural analysis for normal conditions of use.

This analysis is generally sufficient when requirements of fire stability are expressed in terms of duration of a standard fire.

The load bearing resistance can also be deduced from a sub-assembly analysis or a general structural analysis by taking into account interaction between the various members, expansion, and localisation of the fire.

Such sub-assembly analysis leads to a more accurate knowledge of the behaviour of the structure in fire. However these analyses require the use of computer modelling.

4. LOAD BEARING RESISTANCE OF STEEL MEMBERS

The critical temperature (θ_{crit}) which leads to the failure is calculated for a steel structure assuming a uniform temperature distribution along and across the members.

Some examples of calculation of the critical temperature where the theory of plasticity applies are given below.

Four kinds of structural elements are considered: tensile members, columns, beams and beam columns [1].

4.1 Tension Member

At room temperature, the ultimate tensile resistance is given by:

$N_p = A \cdot f_y$

where:

A is the cross-section of the member,

f_y is the yield stress.

At a given uniform temperature θ, through the member, the ultimate tensile resistance is:

$N_p(\theta) = A \cdot \psi(\theta) \cdot f_y$

$\psi(\theta)$ is the strength reduction of steel at θ, and is given by Figure 2.

The collapse of the member will occur at the temperature θ_{crit} when:

$N_p(\theta_{crit}) = N$

where: N = the applied load in fire conditions

This formula can also be written as:

$A \cdot \psi(\theta_{crit}) \cdot f_y = A \cdot \sigma$

where: σ = applied stress in fire conditions

Thus $\psi(\theta_{crit}) = \sigma/f_y$

or, $\psi(\theta_{crit}) = A\,\sigma/A.f_y = N/N_P = P/P_u$

Therefore, knowing P/P_u it is possible to determine, using Figure 2, the value of the steel critical temperature (θ_{crit}) for which $\psi(\theta_{crit})$ is equal to P/P_u.

4.2 Columns

A similar calculation as for tensile members applies but the analysis has to include the effects of column buckling. This is taken into account by modifying the ultimate load bearing resistance by the buckling coefficient. In order to correlate test results on columns with the basic performance of steel at elevated temperatures, it is necessary to consider a correction factor, κ, such that:

$\psi(\theta_{crit}) = \kappa P/P_u$

Both P and P_u should be evaluated using the appropriate bucking coefficient. The buckling coefficient for a column at a temperature θ is given by:

$\chi(\theta) = 1/\{\phi(\theta) + [\phi(\theta)^2 - \bar{\lambda}(\theta)^2]^{1/2} \leq 1$

where: $\phi(\theta) = 0{,}5\,(1 + \alpha(\bar{\lambda}(\theta) - 0{,}2) + \bar{\lambda}(\theta)^2)$

and $\bar{\lambda}(\theta) = \dfrac{\lambda}{\pi}\sqrt{\dfrac{f_y(\theta)}{E(\theta)}} \approx \sqrt{\dfrac{f_y}{E}}$

The end conditions of the column have to be taken into account. Generally the cold parts at the ends of the column lead to a lower value of buckling slenderness, λ, than under normal conditions.

The value of the correction factor, κ, is equal to 1,2. It is used to compensate for the choice of f_y which is related to the effective yield stress (the stress level at which the stress-strain relationship of steel tends to a yield plateau for a certain temperature) and not to the yield stress at 0,2% strain.

4.3 Beams

4.3.1 Simply Supported Beam

The maximum bending moment of a simply supported beam uniformly loaded (by load P over its length) is:

$M = PL/8$

and the corresponding maximum stress is:

$\sigma = M / S_e$

where:

S_e is the minimum elastic modulus of the section

To obtain collapse according to plastic theory, it is necessary that a plastic hinge forms at mid-span. The failure will occur when the total load on the beam is:

$P_u = 8\ M_u/L$

where:

M_u is the plastic bending moment resistance given by:

$M_u = Z\ .\ f_y$

and Z is the plastic modulus of the section

When the temperature is equal to θ, this plastic bending moment resistance is equal to:

$M_u(\theta) = Z\ .\ \psi(\theta)\ .\ f_y$

For a beam subject to a load of P, the collapse will occur at θ_{crit} when:

$P_u(\theta_{crit}) = P$ or $M_u(\theta_{crit}) = M$

i.e. when $\psi(\theta_{crit}) = S_e\ .\ \sigma/Z\ .\ f_y = \sigma/(f\ .\ f_y) = P/P_u$

where $f = Z/S_e$ is the shape factor of the steel section (~ 1,10 to 1,3).

4.3.2 Continuous Beam

When the beam is statically indeterminate, several plastic hinges are necessary to obtain collapse. For beams designed accordingly to elastic theory, a special coefficient (χ) has to be taken into account in the calculation of the critical temperature for the collapse condition. This coefficient takes account of the redistribution of moments in the intermediate structure (plastic analysis).

For example a continuous beam on three supports, uniformly loaded, has a maximum bending moment (M) at the middle support, where:

$M = PL/8$

i.e. a value equal to that at midspan of a simply supported beam.

In fire a plastic hinge will form at this middle support as the temperature increases when:

$M_u(\theta_1) = M$

For the simply supported beam the failure occurs at this temperature (θ_1), whereas the continuous beam still needs an increase of temperature up to θ_{crit} in order that other plastic hinges form in the spans (Figure 6), leading to collapse of the beam.

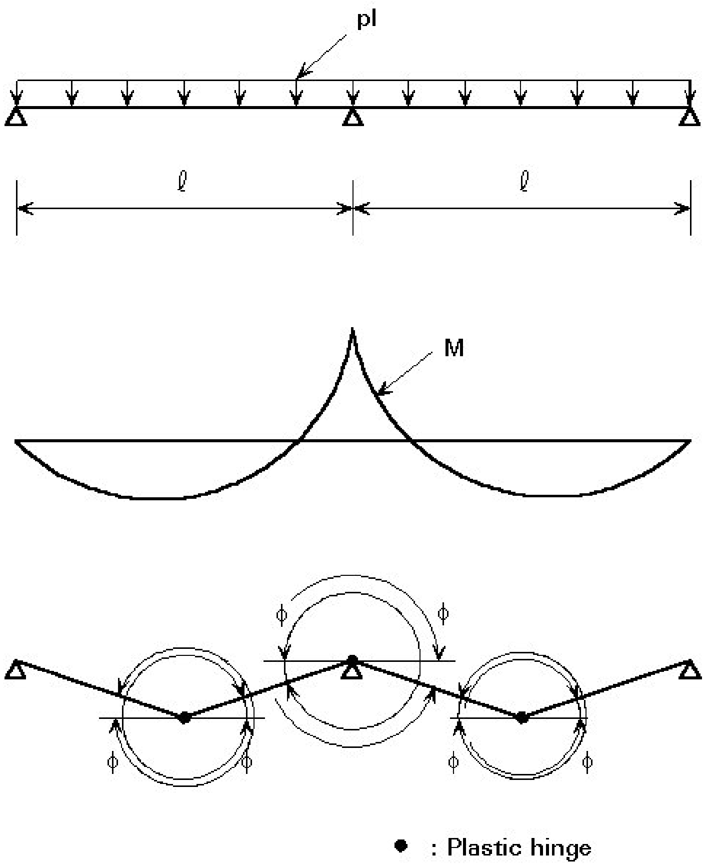


Figure 6 Plastic collapse of continuous beam

The load bearing resistance of this continuous beam is:

$P_u(\theta_{crit}) = 12 . M_u(\theta_{crit})/L$

so $\psi(\theta_{crit}) = 8.S_e.\sigma/12 . Z . f_y = 8.\sigma/12.f.f_y \quad = P/P_u$

The ratio 12/8 = 1,5 = χ is the statically indeterminate coefficient, or plastic redistribution coefficient.

4.4 Beam Column

When axial force and bending moment act together on the same structural element, its critical temperature can be obtained from the following formula:

$$\psi(\theta_{crit}) = \frac{N}{\chi_{min}\, N_p} + \frac{k_y \cdot M_y}{M_{p\ell,y}} + \frac{k_z\, M_z}{M_{p\ell,z}}$$

where:

χ_{min} is the lesser of the buckling coefficients χ_y and χ_z about the yy or zz axis and

k_y and k_z are the reduction factors for the yy and zz axes respectively (see Lectures 7.10)

4.5 Main Parameters

The various parameters which have a strong influence on the critical temperature may be found by study of the above mentioned formulae.

The general formula for bending elements is:

$\psi(\theta_{crit}) = \sigma/\chi.f.f_y$

χ is the coefficient given in Section 4.3.2 above.

and for columns and beam columns is:

$\psi(\theta_{crit}) = N/\chi_{min}N_p + \Sigma(k_iM_i/M_{pi})$

These show that ψ will decrease and subsequently the critical temperature will increase, when:

- the stress (= applied load) decreases or,
- the yield stress of the steel grade increases or,
- the shape factor (f) increases or,
- the statically indeterminate coefficient increases.

The critical temperature can also be increased by using other steel grades with better behaviour when heated.

4.6 Steel Elements with Non-uniform Temperature Distribution

In reality it is very seldom that temperature across and/or along elements is uniform.

Thermal gradients occur for several reasons, for example, the presence of a slab or a wall near the flange of a steel profile, localised fire, connection between column and beam (leading to a concentration of steel), element located outside a fire compartment. Thermal gradients have different effects on the mechanical behaviour of structural elements.

For beams, a lower temperature in the upper flange will lead to an increase of the ultimate bending moment (Figure 7). For continuous beams, a lower temperature in the area of the middle support will lead, either, to more time to reach the temperature at which the plastic hinge occurs, or, to a displacement of this hinge to a cross-section where an optimum ratio between bending moment and temperature is obtained.

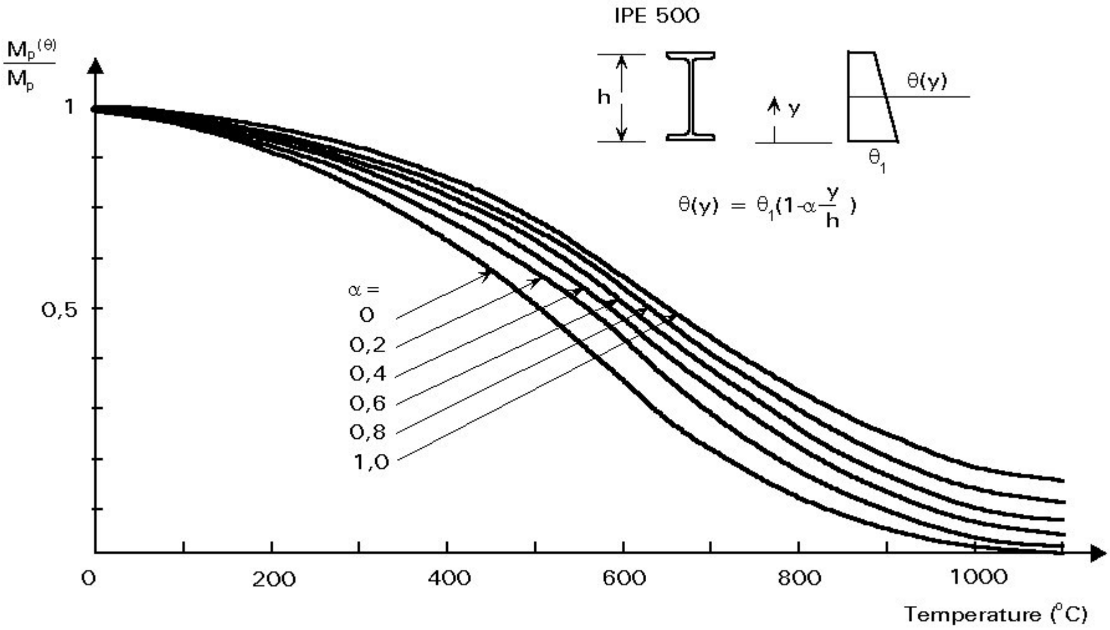


Figure 7 Effect of non-uniform temperatures on bending capacity.

Thus the reduction of the temperature in a part of a beam leads to an increase of its load bearing resistance.

Such thermal gradients can be taken into account, either by calculating the load bearing resistance as explained in Section 5 or by using the global coefficient, called the Kappa-factor. The Kappa-factor is a global coefficient to account for the beneficial influence of thermal gradient for beams. For this purpose the general formula for a beam then becomes [2].

$\psi(\theta_{crit}) = \kappa.\sigma/\chi.f.f_y$

where:

- κ= 1 for simply supported beams exposed to fire on all sides
- κ= 0,7 for simply supported beams exposed on 3 sides
- κ= 0,85 for continuous beams exposed on all sides
- κ= 0,60 for continuous beams exposed on 3 sides.

For columns, a lower temperature at the ends will effectively decrease the buckling length. However a thermal gradient in a cross-section, especially when this section is near the mid-height, will cause curvature of the column. An additional bending moment will be created, increasing the stress in the column. However, in general, non-uniform heating increases the strength of the columns because the colder parts are still able to resist compression.

5. LOAD BEARING RESISTANCE OF COMPOSITE MEMBERS

Composite sections in which concrete and steel are used, are subject to thermal gradients when heated on one or several of their sides. The load bearing resistance of beams and slabs can be determined on the basis of simple plastic theory. To calculate the load bearing resistance of a member with a thermal gradient, the simplest approach is to divide the section into elements each with its appropriate temperature and mechanical properties.

5.1 Composite Beam

A composite beam (steel section and flat concrete slab) generally has a distribution of temperature after a given time of fire exposure as shown in Figure 8.

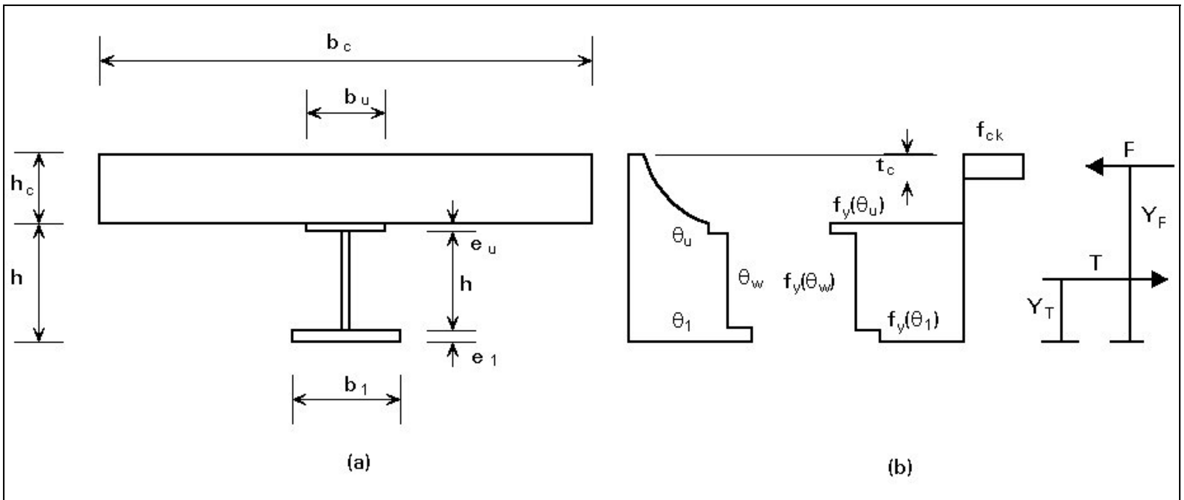


Figure 8 Composite section after a given time of fire exposure

In a positive moment zone, the ultimate bending moment resistance, assuming the neutral axis is in the thickness of the concrete slab, is calculated by considering an equilibrium between tensile force (in the steel section) and compressive force (in the upper part of the concrete slab).

The tensile force summed over the three parts of the steel section is:

$$T = \Sigma^{3}_{i=1} A_i \, \psi(\theta_i) f_y$$

where:

A_i is the area of lower flange, web and upper flange of the steel profile

θ_i is the respective temperature

The point of application of this force is the plastic neutral axis at elevated temperature of the 3 parts of the steel action.

In order to balance this tensile force, a layer of the concrete slab is compressed (Figure 8b) such that:

$$T = C = b \; t \; f_{ck}$$

where:

b is the effective width of the slab

t is the thickness of the compressive zone

f_{ck} is the ultimate strength of concrete

This equation is only valid when the temperature of the compressive zone is approximately uniform. If a strong thermal gradient exists over the height of this zone, it is necessary to divide it into different layers having approximately uniform temperature and to sum the contribution of these layers.

However normally this composite floor generally has also to achieve insulation criteria, i.e. the temperature on its external face has to be less than 140°C. At this temperature, it can be assumed that the concrete strength remains as it is at room temperature.

The ultimate bending resistance of the composite section is:

$$M^{+}_{u(\theta)} = T \cdot z$$

where z is the distance between the points of application of the tensile and the compressive forces.

For continuous beams, the determination of the full load bearing resistance also requires the calculation of the negative plastic bending moment ($M^{-}_{u(\theta)}$).

For this negative plastic moment, the tensile force is taken by the reinforcement bars located in the upper part of the concrete slab and the compressive force is taken by the steel profile and, if necessary, by a lower part of the concrete slab.

For this situation, it is assumed that stability in fire is maintained if the isostatic bending moment (M) of the applied load in a span is lower or equal to the sum of the positive and negative moment resistances of the composite section, as follows:

$$M^{+}_{u(\theta)} + M^{-}_{u(\theta)} \geq M$$

5.2 Composite Slabs

The calculation of the fire behaviour of composite concrete slabs with profiled steel sheet is made using the same theories as for composite beams.

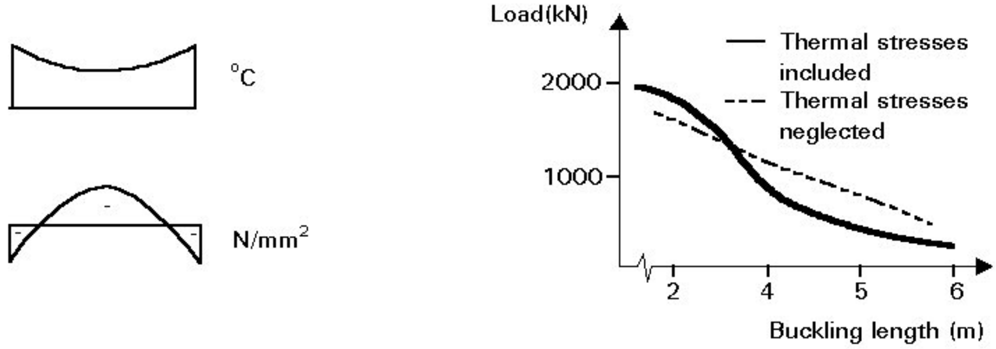
The only major modification is that after 30 minutes of ISO fire, the steel sheet is not taken into account when calculating the mechanical behaviour of the element.

For this situation, only the re-bars can be used to compensate the tensile force because the tensile strength of concrete does not contribute to the load bearing resistance at elevated temperature and is ignored. Only 30 minutes fire resistance can generally be achieved for non-reinforced slabs.

5.3 Composite Columns

Simple plastic theory cannot be used for columns (in contrast to beams and slabs) and an incremental elasto-plastic approach is necessary.

For composite columns another complication arises as a direct consequence of the non-uniform temperature distribution over the cross-section. The distribution causes additional stresses in the cross-section due to restrained thermal elongation (Figure 9).



(a) Distribution of temperature and thermal stresses (schematic)

(b) Buckling curve at elevated temperatures

Figure 9 The effect of restraint of thermal elongation on the load-bearing resistance of concrete filled steel columns exposed to fire

These thermal stresses may have a significant influence on the load bearing resistance of composite columns, as is illustrated by the buckling curves (Figure 9). Both curves are for a reinforced, concrete filled steel column after 90 minutes standard fire exposure. The continuous curve is based on a calculation model which takes the thermal stresses into account; the dashed curve neglects the effect of the thermal stresses.

Structural analysis of composite columns should therefore, ideally, be based on refined models, i.e. models allowing for a precise thermal and mechanical analysis.

The numerical complexity of such physical models however, quickly increases with the growing precision of the analysis. The complexity has drawbacks, for example where design information requires a great number of systematic calculations. For this reason limited, more approximate, models have been developed. These simpler methods occasionally require the introduction of semi-empirical correction factors. They should therefore be used with caution when extrapolating outside the range of experimental evidence.

The ECCS-Technical Note on the calculation of the fire resistance of composite columns provides design information in the form of buckling curves for various cross-section dimensions, profiles and reinforcement and for periods of standard fire exposure of 30, 60 ,90 and 120 minutes [3]. An example of a design chart is given in Figure 10.

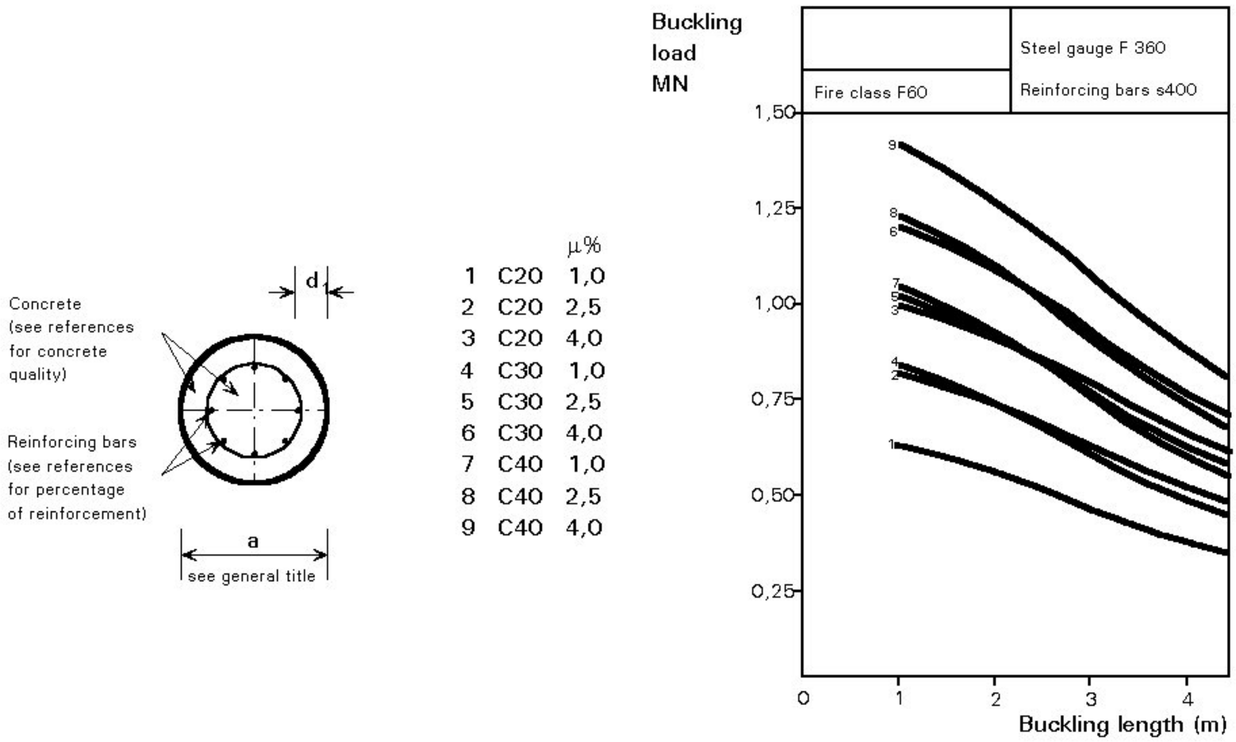


Figure 10 Buckling curves for concrete filled HSS-profiles after 60 minutes standard fire exposure

6. CONNECTION BETWEEN MEMBERS

There is no special problem in determining the fire resistance of connections between columns and beams. Due to the concentration of steel in this area, the temperature of the connection is lower than that of the adjacent members. It may be possible to consider some positive effects of partial continuity of beams when there is a connection which is only designed to support shear forces at room temperature.

7. CONCLUDING SUMMARY

- The increase of the temperature of steel and concrete leads to a decrease of mechanical properties (Young's modulus, yield strength, ultimate strength).
- The applied loads considered under fire conditions are obtained by using the accidental combination of the actions.
- The critical temperature of elements is easy to calculate by equating the fire load bearing resistance and the applied loads.
- Composite columns require complicated calculations but design charts are available for usual applications.

SOURCE http://www.kuleuven.be/bwk/materials/Teaching/master/wg04b/l0400.htm

Lecture 4 Practical Ways of Achieving Fire Resistance of Steel Structures

OBJECTIVE/SCOPE

To survey the practical means of achieving fire resistance of steel structures with examples of their application. To describe the essentials of European fire resistance design.

SUMMARY

The mechanical properties of all common building materials decrease with elevation of temperature. Steel structural elements should possess an appropriate fire resistance to resist collapse, flame penetration or excessive temperature rise on the unexposed faces. The inherent fire resistance of unprotected steelwork is introduced and the influence of a variety of insulating systems, of partial member exposure and of composite action are discussed. Reference is also made to the effects of water cooling on temperature control.

1. INTRODUCTION

The mechanical properties of all common building materials decrease with elevation of temperature. Structural elements should possess an appropriate fire resistance to resist collapse; in addition fire resisting partitioning walls and slabs should resist flame penetration or excessive temperature rise on their unexposed faces in order to contain the fire in its original location. The fire stability of a structure is especially important and any failure of the structure in the fire zone should be gradual, involving large plastic type deformations. The parts of the building away from the fire should remain intact.

Fire resistance requirements are fixed by National Codes in terms of the time an isolated element should resist the action of a Standard Fire as defined by the heat exposure given by ISO834, (Figure 1). Fire resistance times of 15/30/60/90/180 and 240 minutes are specified depending upon the number of storeys; these times can also be a function of the occupancy of the building and of the fire load.

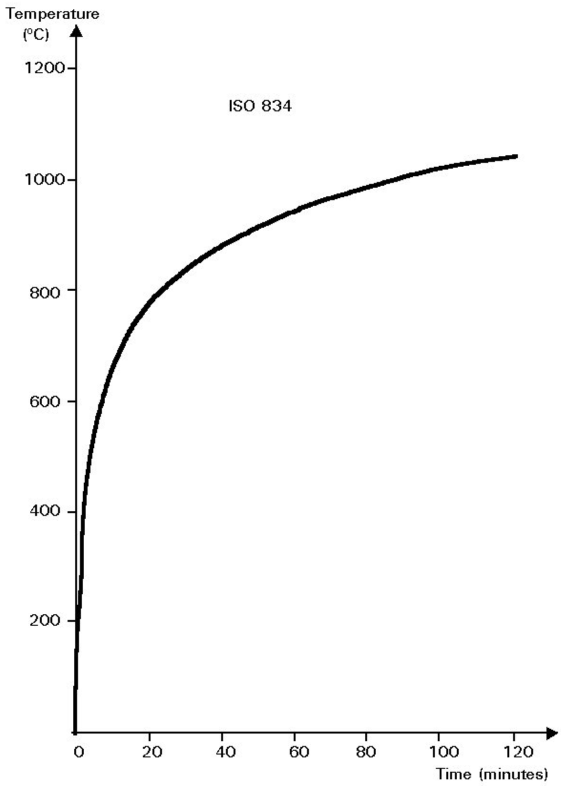


Figure 1 Standard time - temperature curve in fire test

Steel members will collapse in a fire when their temperature reaches a "critical" level. This critical temperature varies according to the load conditions, the cold design theory adopted and the temperature distribution across the section, which typically is in the range 500 to 900°C.

The fire resistance time is the time, in the standard ISO834 fire test, taken by the member to reach the critical temperature. This time varies according to the section size. In a building in which a natural fire occurs the heating rate is also influenced by the member location. The thicker the steel the slower is the heating rate and therefore the greater is the fire resistance time.

The heating rate is quantified by the Section Factor, known as the A_m /A ratio, where A_m is the perimeter of the steel member exposed to the fire, and A is the total cross-sectional area of the section. Consequently, a heavy member with a low A_m /A ratio will be heated more slowly than a light member with high value of the section factor. Tables are published giving values of section factors for standard section sizes.

For a member to fulfil a given fire resistance requirement, it is necessary to ensure that the temperature developed in the member at the required fire resistance time (taking into account its Section Factor and any insulation which may be applied) is less than the critical temperature necessary to cause failure (also known as the "critical temperature").

For short periods of fire resistance (15, 30 minutes) stability may be attained by unprotected steelwork. A fire resistance time of 60 minutes may sometimes be obtained without applying fire protection by utilising the thermal and/or structural interaction between steel and concrete. For longer periods of fire resistance time, the steelwork can be protected by applying an insulating material, by using screens, or, in the case of hollow sections, by the recirculation of water. Composite steel-concrete structures can also exhibit significant fire resistance.

A brief survey of the simpler practical means of achieving structural fire resistance in steel structures is presented. It is important to recognise, however, that considerable research and development work (fuel loads based on natural fires) is being undertaken in Europe. This work aims to optimise the process of the fire resistant design of structural steelwork leading to further economies in construction.

2. BARE STEEL STRUCTURES

Bare steel structures may satisfy fire resistance times of 30 and 60 minutes if one or more of the following conditions are met:

- low load level.
- low value of the section factor, A_m /A.
- high degree of static redundancy (viz: may be influenced by the design of the connection).

Figure 2 shows an example in which the fire resistance of bare steel beams is given as a function of the section factor, for different values of the ratio between the actual load and the collapse load under room temperature conditions.

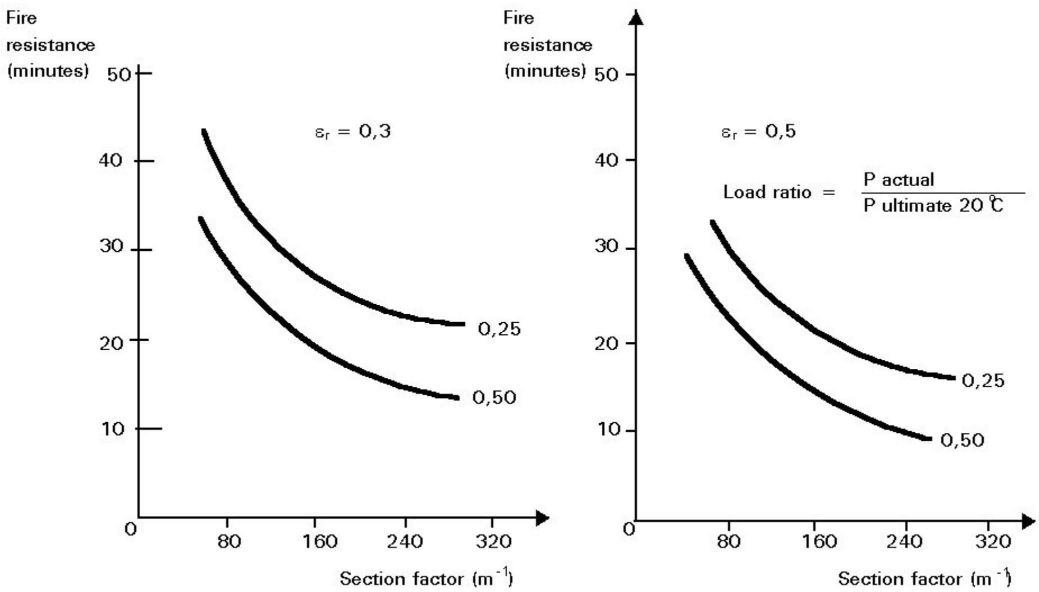


Figure 2 Fire resistance of bare steel beams as a function of section factor, different load levels and different resultant emissivity values

When the ratio between the applied load and the collapse load is reduced, the failure temperature, and thus the fire resistance time, is increased. The fire resistance time can therefore be increased by oversizing the members, by maintaining the member size but using a higher strength steel, by utilising the restraining effects of connections, or by a combination of these methods.

Heating rates of fire exposed members may be calculated on the basis of European Recommendations [1, 2] which are incorporated in Eurocode 3[3]. These rates hold for continuous beams. The calculations assume a uniform temperature distribution across the steel member. However, research has shown that the temperature profile has an important influence on fire resistance when non-uniform temperature distributions are developed. For example, in a beam supporting a concrete slab, the fire resistance is increased due to the transfer of load from the hotter to the cooler part of the section. This effect is accounted for by use of a modification factor 'κ' in the calculation method.

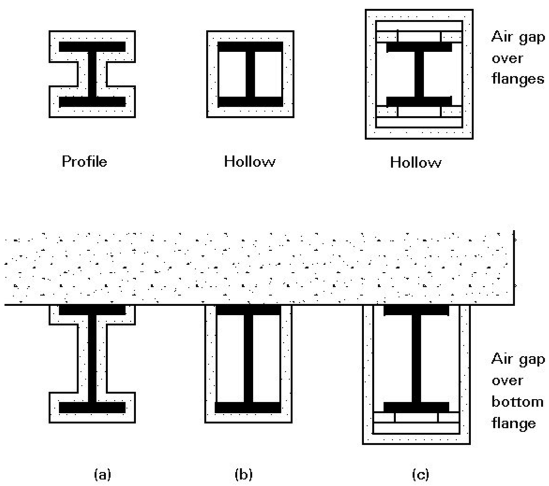
In a fire, heat is transferred to the steelwork predominantly by radiation and the rate of heat transfer is described by the resultant emissivity ε_r. The value of ε_r will change according to the characteristics of the furnace used for standard fire tests on beams and their position in relation to the flames. Typically, ε_r will be between 0,3 and 0,5, the lower value resulting in an increase in the measured fire resistance time. The effect of this variation on the fire resistance of bare steel beams is shown in Figure 2.

The fire resistance of bare steel columns exposed to heat on four sides also depends on the section factor and the applied load. Bare columns with section factors up to 30 m^{-1} have a fire resistance of 30 minutes when working at full design loading, based on Eurocode 3 [3].

3. PROTECTED STEEL STRUCTURES

In many steel framed buildings, structural fire protection is required to meet the requirement of legislation and to prevent failures of major building components in fires. A wide range of fire protection systems are available. The generic forms, such as concrete, brickwork and plasterboard are well established. The materials available also include sprayed materials, dry products in the form of boards and batts, intumescent products which form a carbonaceous char when exposed to heat, and compounds which absorb heat and undergo chemical changes in fire.

The thickness of the insulation must be such that the temperature of the steel at the required fire resistance time (taking into account its section factor) does not exceed the critical (or limiting) temperature. Government agencies and approved private laboratories have established programmes of fire tests for passive and intumescent protection systems on loaded and unloaded specimens. These tests are designed to determine both the insulation characteristics of a fire protection material and its physical performance under fire conditions for a range of steel sizes. Analytical methods from which reliable assessments of the thickness of the protection medium can be made are now available. The fire protection can be applied to the structural steel member in a variety of ways, as shown in Figure 3.



(a) - Spray or intumescent
(b) - Board
(c) - Board

Figure 3 Ways of applying fire protection

Sprayed Protection

The various spray systems include mineral fibre products, vermiculite-based products which include either cement or gypsum, perlite/cement products and chemical compounds that absorb heat, such as magnesium oxychloride. The majority of these systems form a slurry in a mixer which is pumped through the nozzle onto the steel substrate. The mineral fibre/cement mixture is mixed with a water spray at the nozzle head. The thicknesses of these materials vary from 10 to 100 mm with specific mass in the range of 200 kg/m^3 to 1000 kg/m^3.

To achieve the required degree of fire resistance, it is important that the specified thickness of coating is applied. Inspection of the quality of the coating and thickness checking is therefore required. However, no specific guidance is currently available on the number of positions where thicknesses should be checked and the acceptable tolerance limits.

These sprayed materials have several advantages. They are fast to apply, inexpensive and can be adapted to cover complicated shapes including the voids between metal deck floors and steel beams. Their disadvantages are that they are messy, can cause damage due to overspray and are sometimes susceptible to cracking and shrinkage. They do not provide an attractive surface finish unless trowelled smooth.

These systems of protection are generally applied to hidden elements, e.g. beams above suspended ceilings. It may be possible with the aid of colouring to integrate these coatings to the architectural aspect of the structure. The spray composition must be compatible with the substrate, be it primed or unprimed steelwork.

The abrasion and impact resistance of sprayed insulation is improved with an increase in its cohesive strength and density. The coatings are difficult to repair and therefore it is important that any attachments to the steelwork are made prior to the installation of the fire protection.

Dry Systems

These include board systems based on mineral fibre or vermiculite, mineral fibre batts and ceramic fibre blankets. Board materials can either be glued in place using noggings, or screwed to a framework or to other boards. The specific mass of the board materials vary between 165 to 800 kg/m^3.

These products are generally easy to use. The extent of checking required during installation is much less than that needed with a spray-applied coating as the products are manufactured with reliable thicknesses. They provide some degree of flexibility in programming, are clean, cause little damage to surrounding constructions and offer a good surface finish. Some board products are soft or brittle and are susceptible to mechanical damage; others are susceptible to water damage and are only suitable for internal use. Installation is not easily adaptable around complex shapes. Few problems are encountered by compatibility with substrates.

Recent developments have seen an increase in the use of mineral fibre batt materials. These materials have a specific mass around 100 kg/m^3 and are held in place using pins, welded at regular intervals onto the steel surface, and retaining washers.

The desirable properties of both the spray and dry systems of protection are as follows:

- good thermal insulation, i.e. low thermal conductivity and/or high thermal capacity.

- satisfactory mechanical resistance to shock and impact.
- good adhesion to the element to prevent separation of the protection material by rising temperature and deformation of the structural member.

In order to facilitate the use of sprayed and sheet materials, special graphs have been prepared by authorised fire testing laboratories. These graphs give the thickness of a specific material as a function of the section factor, the critical temperature of the structural member, and the required fire resistance period, as shown in Figure 4.

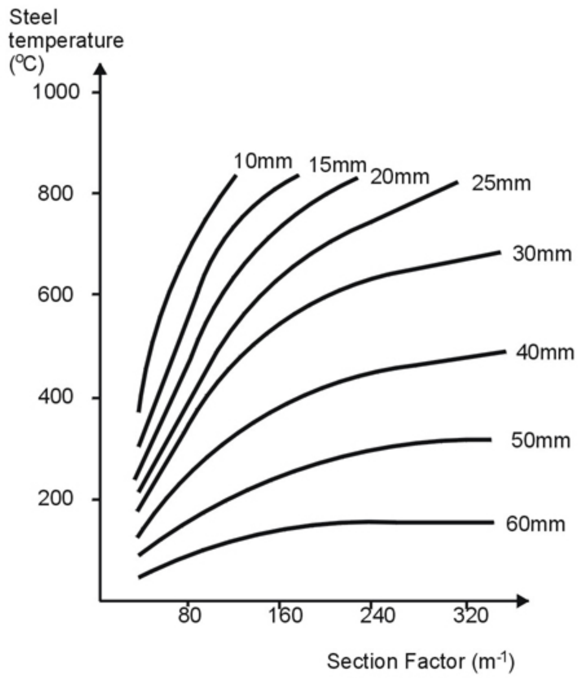


Figure 4 Derivation of required insulation thickness for a fire resistance of 90 minutes

Intumescent Systems

These materials are used to provide a decorative finish to a structure. A range of thin film coatings are available that can satisfy up to 90 minutes fire resistance. These products are mainly suitable of internal use. A range of thick film coatings based upon the epoxy chemicals can satisfy up to 120 minutes fire resistance. These coatings exhibit satisfactory ageing characteristics when used externally.

Typical Intumescent Coating Thicknesses Used on I-Sections

Coating type	Range of basecoat thickness for different fire resistance periods* (mm)			
	30 mins	60 mins	90 mins	120 mins
Solvent based thin coats	0,25 - 1,0	0,75 - 2,5	1,50 - 2,50	-
Epoxy resin- based thick coats	4,0 - 5,0	4,0 - 11,0	6,0 - 16,5	6,0 - 16,5

* Data from UK practice

The thin film coatings or mastics foam and swell under the influence of heat to produce an insulating char layer up to 50 times thicker than the original film thickness. These products can be applied by spray, brush or roller. In order to apply thicker coats multiple treatments are necessary. Control measurements on thickness are required using proprietary measuring equipment which has been developed for assessing paint thickness. Only a limited amount of investigation of durability has been conducted on the ability of certain products to be used externally. Most of the products have good resistance to impact and abrasion.

A simplified graph showing fire resistance periods provided by a single coat of intumescent paint is shown in Figure 5.

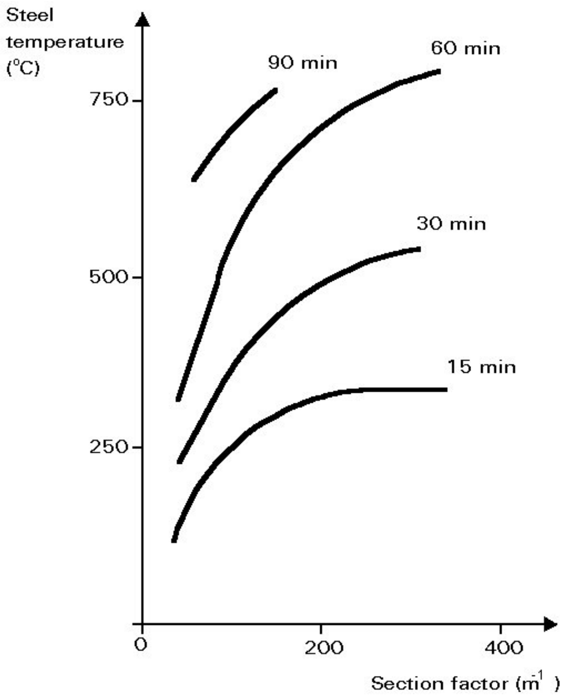


Figure 5 Fire protection provided by a single coat of specified thickness of an intumescent paint

Tests have shown the need to evaluate the performance of intumescent coatings over a variety of shapes and orientations of the substrate. In the long term it is anticipated that this form of fire protection may be installed by the steel fabricator.

Although these materials have resistance to impact and abrasion, mechanical damage can occur, particularly on columns, requiring maintenance of the paint system to be carried out.

Specification of Fire Protection Thickness

In Eurocode 3: Part 1.2 [3] an equation is given to calculate the rise in temperature of protected steelwork. The thermal conductivity of the insulation material, λ_i, and its thickness d_i are taken into account as λ_i / d_i. The heat capacity of the insulation is also included. The thermal conductivity of insulating materials changes with their mean temperature. This change can be taken into account in more precise calculations.

However, if no detailed information is available and if only an approximate answer is required, the analysis may be based on average values of λ_i, which are assumed to be valid for the whole temperature range during a fire. It may be shown that under such circumstances the time to attain a certain steel temperature is governed by the factor

$\lambda_i A_m / d_i A$

The required thickness of insulation for a structural steel member may be determined by using a nomogram which relates critical temperature, applied load, section factor and fire resistance. For example, consider an IPE 500 beam supporting an actual load/collapse load, η = 0,625 requiring a fire resistance of 120 minutes and exposed to heat on 3 sides. The nomogram for protected steel is reproduced in Figure 6 for a simply supported beam supporting a concrete slab (k factor of 0,7). The product of η x k = 0,625 x 0,7 = 0,438 and the section factor, A_m /A = 132 m^{-1}, for a beam exposed to heat on three sides.

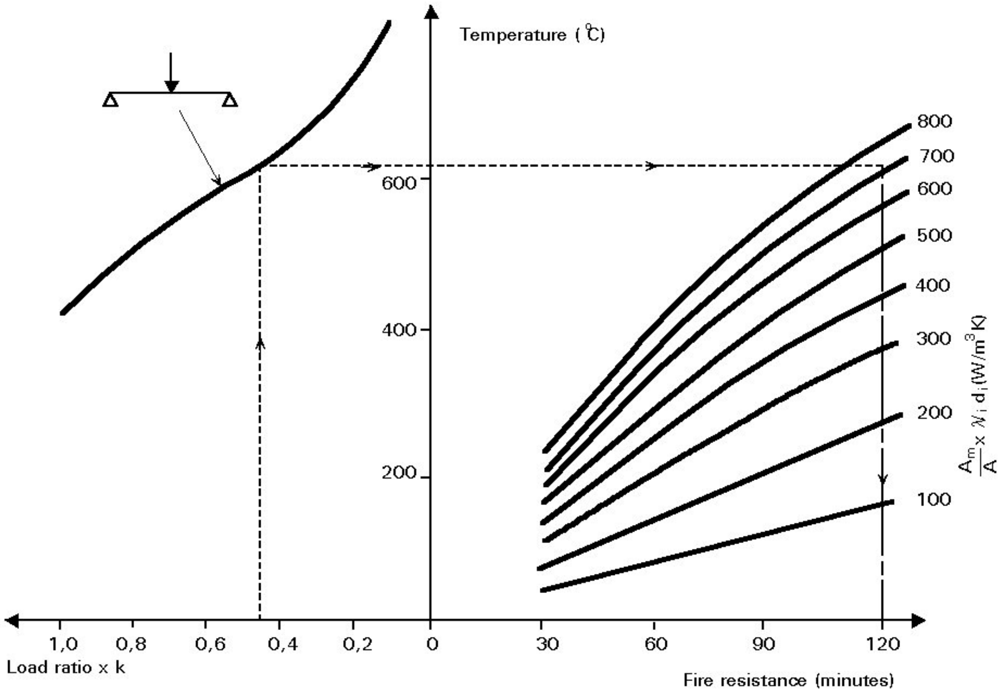


Figure 6 Use of nomogram to predict required thickness of insulation

From the nomogram the value of the factor {$\lambda_i A_m / d_i A$} is 690 W/(m^3.K) to satisfy 120 min fire resistance with η x κ = 0,438

$$\frac{d_i}{A_i} \geq \frac{132}{690} = 0{,}19 \ \frac{m^{2}\,{}^\circ C}{W} \quad (1)$$

Values of thermal conductivity, λ_i, may be obtained from manufacturers' data and examples are given in the nomogram. For instance, when λ_i = 0,1 W/m°C (typical of many protection materials), the required thickness of insulation, $d_i \geq$ 0,1 x 0,19 = 19 mm.

4. COMPOSITE CONSTRUCTION

The use of composite steel/concrete components in buildings is becoming increasingly important in fire resistant design because they offer several choices for influencing the rise in temperature of the steel [4, 5]. One is the position and mass of the concrete and a second option is the possibility of redistributing the internal stresses to protected and cooler parts of the section.

Concrete-Filled Hollow Steel Columns

The cross-section of this type of column is either rectangular or circular as shown in Figure 7(a, b, c). The performance in fire depends mainly on the member size and the tensile and flexural properties of the concrete. If non-reinforced concrete is used the fire resistance is normally 30 minutes (Figure 7a). However, a rating of 120 minutes can be achieved by the inclusion of reinforcing bars or steel-fibre reinforcement (Figure 7b).

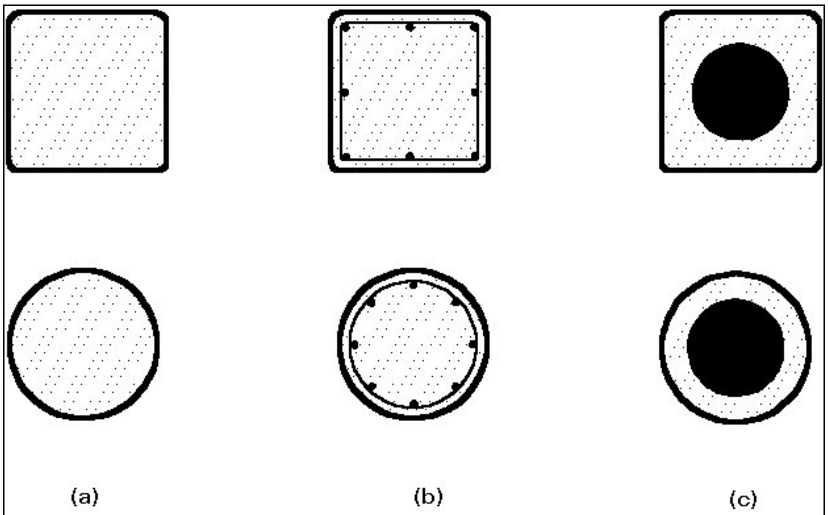


Figure 7 Concrete-filled hollow columns

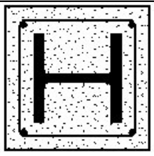
The steel core column (Figure 7c) is a further development of the concrete filled hollow section but with the main part of the load-carrying steel cross-section protected against fire by layers of concrete. The fire resistance of this type of column varies from 60 minutes to higher values depending upon the thickness of the concrete layer. These columns are used as centrally loaded members with small load eccentricities.

Rolled Steel Sections Encased in Concrete

Different types of composite construction utilising steel sections are manufactured into fire resistant columns and beams.

One of the great advantages of composite columns is their uniform outside dimensions in multi-storey buildings. By varying the thickness of the steel section, the material qualities of both steel and concrete, and the percentage of reinforcement, the cross-section of the column may be adapted to support an increased load without significant changes in the outer dimensions. Each type of composite column has specific advantages and ranges of application.

The oldest type of composite column, Figure 8a, is the steel section encased in concrete. Its advantages are a high allowable load level in fire conditions and a high load-carrying resistance not only for centrally applied loads, but also for bending moments. The fire resistance is normally 90 minutes or more.


(a)

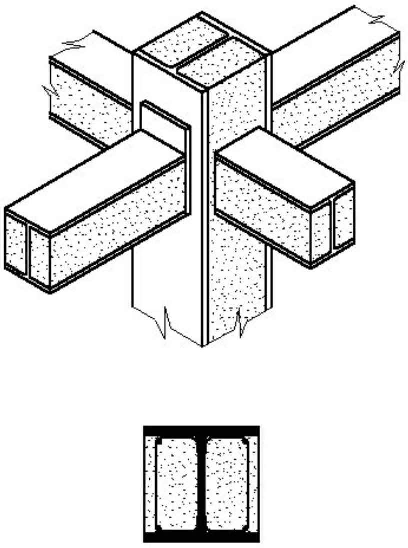

(b)

Figure 8 Rolled steel sections and concrete

The second type, namely the steel section with concrete between the flanges, can support considerable central loads and high bending moments. The amount of shuttering is significantly reduced. Other advantages are a good resistance to mechanical damage without the need for corner reinforcement and the ability to use conventional steel connections between the columns and steel beams similarly concreted and reinforced between the flanges, as shown in Figure 8b. Such composite sections may reach any desired fire resistance level.

Composite Steel Deck Floors

Composite floors utilising profiled steel decks are very frequently used in building as shown in Figure 9. These floors can have a fire resistance of up to four hours without any fire protection applied to the soffit. Floors with minimal reinforcement have at least 30 minutes fire resistance and a single layer of reinforcement can give up to two hours fire resistance. For longer periods of fire resistance for floors with high loading and long spans, additional reinforcement may be necessary.

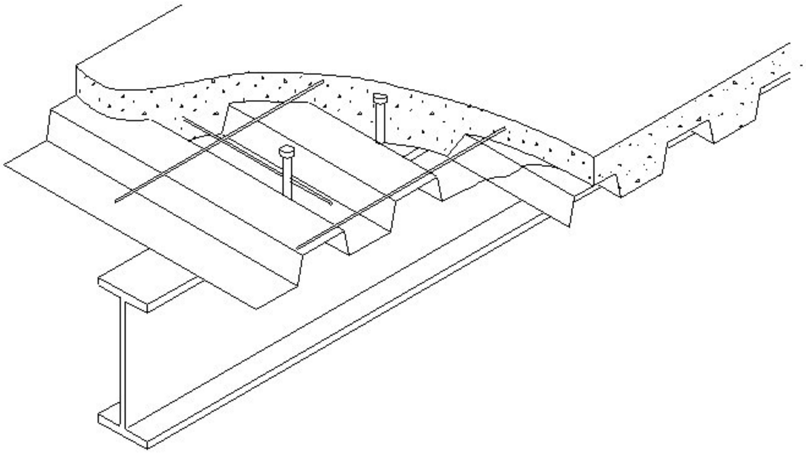


Figure 9 Composite steel deck floors

5. PARTIALLY EXPOSED STEEL SECTIONS

Members partially exposed because they are embedded in walls, floors or other elements of structure achieve a significant fire resistance by redistribution of stress from hot exposed regions to cooler (non-exposed) areas of the section. This effect occurs whether there is composite interaction or not. Research and analysis are in progress to quantify this effect.

A relatively inexpensive method of improving the fire resistance of free-standing universal columns without the specialist application of fire protection can be achieved by blocking in the volume between the flange and web with non-load bearing conventional lightweight building blocks, as illustrated in Figure 10. Fire tests have demonstrated that universal column sections from 203 mm x 203 mm x 52 kg/m upwards achieve in excess of 30 minutes fire resistance under full design loading.

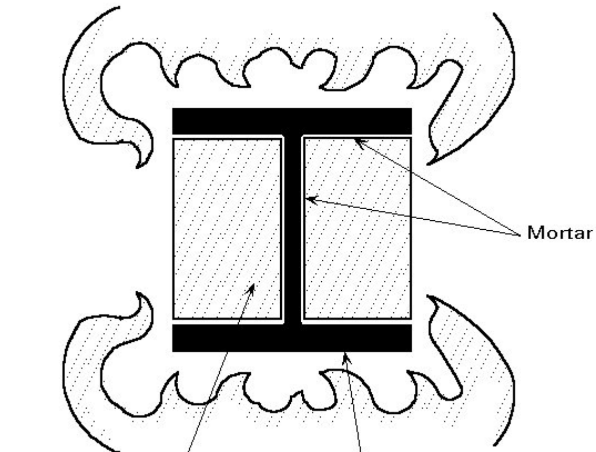


Figure 10 Steel column with blocked-in web

Another particular example is the shelf angle floor beam illustrated in Figure 11. The time taken for a steel beam to reach its limiting temperature in a fire can be extended by protecting the beam from direct attack by the flames. One economic method of providing this protection is by means of the shelf angle floor design where precast concrete floor slabs rest on steel angles attached to the web of the beam so shielding the upper flange and part of the web from the fire. The resulting decrease in heating rate of the upper part of the beam significantly extends the fire resistance time of the steel beam.

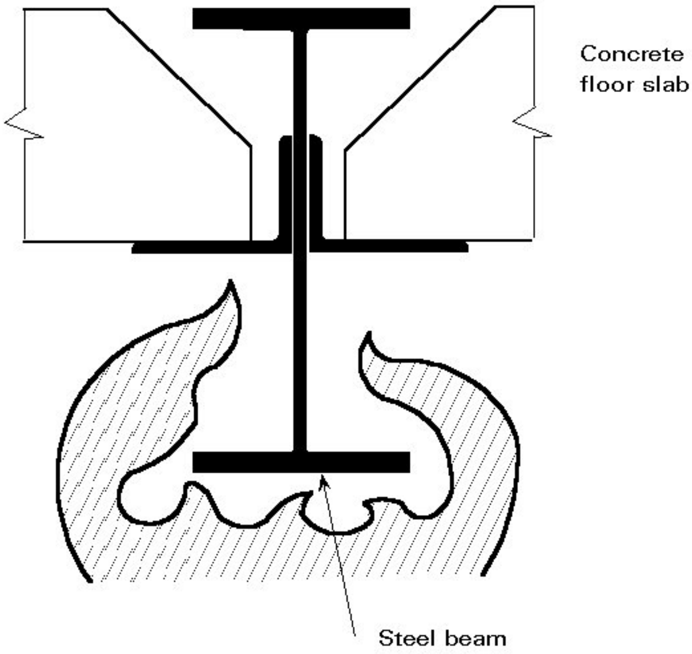


Figure 11 Shelf angle floor arrangement

Research results indicate that by selecting suitable combinations of steel beam size and depth of concrete floor unit, fire resistance periods of 30, 60 and 90 minutes are possible without the need for applied lightweight fire protection.

6. PROTECTION BY SCREENS

It is not necessary to apply protection to each member in the steel frame of a building before it is completed. When suspended ceilings or partition walls (Figure 12) are used they can offer cost advantages by combining their normal functions with fire protection. The screens must be able to ensure the integrity, insulation and stability necessary to prevent the fire from spreading into the void. Special attention should be paid to the method of assembly and in particular the joints and connections. Any desired fire resistance level can be obtained.

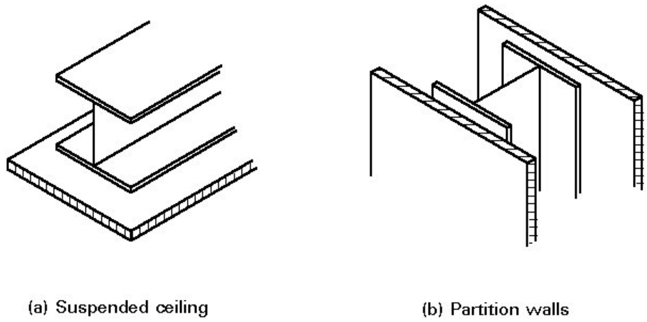
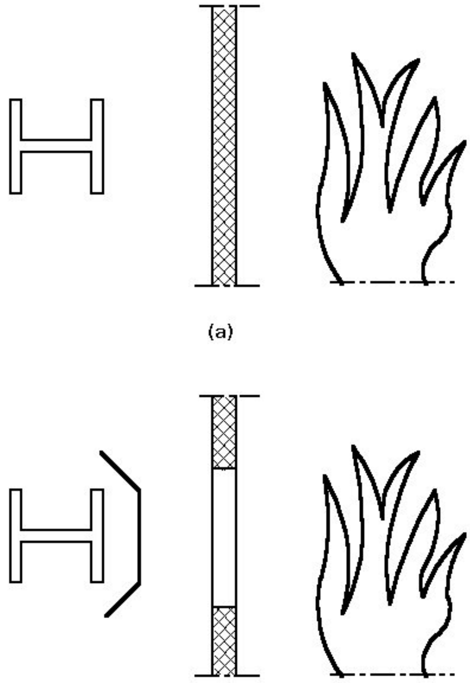


Figure 12 Fire screens

7. EXTERIOR STEELWORK

Columns positioned outside a building remain cooler during a fire than those positioned inside. In this way fire stabilities much greater than 30 minutes can be achieved. Existing calculation methods for the mechanical behaviour of such loaded elements in fire have led to the following recommendations:

- the best position for the column is as far as possible from openings and, either shielded by a wall which has an adequate fire stop rating, shown in Figure 13, or protected by a screen if the column is in front of a window.
- if there is a risk of severe thermal loadings, rigid connections between columns and beams are preferred.
- generally, the load-carrying floor beams need no protection over their external parts.

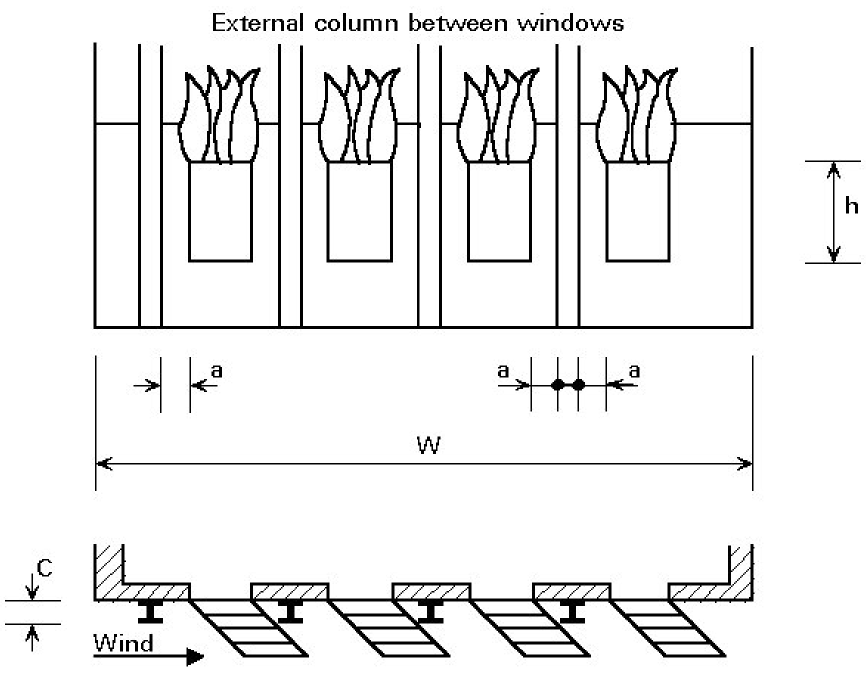


(a)

(b)

Figure 13 External column

A simplified design example is given in Figure 14. The position of the external columns to avoid excessive rise in temperature is indicated for a building which has all the windows on one wall and no through draught.



Plan - shows flame deflection by wind
A = a or C, whichever is the larger

Window height h (mm)	Values of A for compartment width W (m)			
	9	18	36	72
1	1,4	2,3	2,3	2,3
2	0,8	1,1	1,1	1,1
3	0,6	0,8	1,0	1,0
4	0,3	0,7	0,9	0,9
5	0,3	0,7	0,8	0,8

Figure 14 Design example for external columns between windows

8. WATER COOLING

The fire resistance of tubular members can be improved by utilising the hollow interior to cool the load-bearing steelwork. Filling such members with water gives extremely high fire resistance when circulation is maintained, Figure 15.

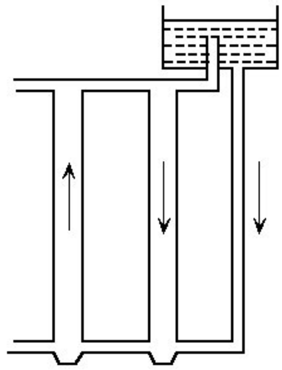


Figure 15 Water-filled hollow sections

Circulation can be achieved by natural convection using a number of interconnecting members (not all of them fire exposed) with an adequate high level storage tank, by direct connection to water mains and drainage, or by pumps. Research is currently being carried out into static unreplenished systems. Chemicals are added to the water to inhibit corrosion (Potassium Nitrate) and freezing (Potassium Carbonate). With any system, although the water temperature can exceed 100°C depending on the pressure, the steel will remain below its critical temperature.

The outward appearance of the steelwork is unaffected which has advantage architecturally. The design is, however, complex and the method expensive. It is normally confined to prestige buildings or structures requiring extreme levels of fire resistance.

Standard rolled beam and column sections may be cooled by water spray. The procedure is activated as soon as the ambient temperature exceeds a predetermined value. The water flow produced by a number of sprinklers must be in a continuous film over the entire length of the member.

9. CONCLUDING SUMMARY

- The heating rate of a steel profile is specified by the section factor, A_m/A. Low ratios lead to small heating rates.
- For short periods of fire resistance, stability may be attained by unprotected steel elements.
- For long periods of fire resistance, the steelwork must be protected by applying an insulating material, by screens or by recirculation of water in the case of hollow sections.
- The choice of the fire protection must take into account the localisation of the element, aesthetic requirements and economy.

SOURCE http://www.kuleuven.be/bwk/materials/Teaching/master/wg04b/l0400.htm

Lecture 5 Calculation Examples

OBJECTIVE/SCOPE

To make designers familiar with simple methods of calculation of fire resistance time and thickness of insulation for columns and beams (respectively steel and composite steel-concrete).

SUMMARY

Calculation examples are presented for the following:

- Critical temperatures of tension members, beams and columns (Examples 1, 2, 3).
- Moment resistance of composite beam in fire conditions (Example 5).
- Time equivalent of actual fire (Example 6).
- Fire protection to steel beam (Example 4).

The examples use the principles and design equations presented in the preceding lectures.

EXAMPLE 1 CRITICAL TEMPERATURE OF TENSION MEMBER

Strength reduction of steel at elevated temperatures						
Temperature θ	400	450	500	550	600	650
Strength Reduction ψ(θ)	1,00	0,93	0,78	0,63	0,47	0,33

For $\frac{P}{P_u} = \frac{\text{Load at fire limit state}}{\text{Tensile resistance under normal temperature conditions}} = 0{,}5$

As the performance of a member in tension is equivalent to the basic performance of the steel, it follows that:

$$\psi(\theta) = \frac{P}{P_u} = 0{,}5$$

By linear interpolation from the above table, the critical temperature θ_{cr} = 590°C

EXAMPLE 2 CRITICAL TEMPERATURE OF BEAM

It is assumed in this example that the beam supports a concrete slab, and hence the upper flange remains cooler than the rest of the section. This benefit is taken into account by use of a load multiplier, or kappa factor, κ, such that:

$$\psi(\theta) = \kappa \cdot \frac{P}{P_u}$$

κ = 0,7 for a beam supporting a concrete slab

Use the same degree of loading as in Example 1.

In this case, $\frac{P}{P_u} = \frac{\text{Load on beam at fire limit state}}{\text{Load equivalent to failure under normal conditions}}$

For $\frac{P}{P_u}$ = 0,5, it follows that:

$\psi(\theta)$ = 0,7 x 0,5 = 0,35

By linear interpolation of the strength reductions in Example 1, the critical temperature of the beam, θ_{cr} = 645°C.

It follows that the critical temperature of a beam supporting a concrete floor slab exceeds that of a member in tension, i.e. uniformly heated, by 55°C for the same degree of loading.

EXAMPLE 3 CRITICAL TEMPERATURE OF COLUMN

It is assumed in this example that the column is restrained against buckling. The load multiplier, κ, for columns is 1,2. This value takes into account the influence of high strains in the column at failure in fire conditions.

For $\frac{P}{P_u} = 0{,}5$, as in previous examples

$\psi(\theta)$ = 1,2 x 0,5 = 0,6

By linear interpolation of the strength reductions in Example 1, the critical temperature of the column, θ_{cr}=560°C.

It follows that the critical temperature of a column is less than that of a member in tension by 30°C for the same degree of loading.

EXAMPLE 4 FIRE PROTECTION TO STEEL BEAM

From Example 2, the critical temperature of the beam is 645°C. From lecture 2 the required thickness of fire protection (in metres) is:

$d = 0{,}0083\ \lambda_i\ (A_m/A)\ \{t/(\theta_{cr} - 140)\}^{1{,}3}$

Where

A_m/A is the section factor of member (m^{-1})

λ_i is the thermal conductivity of protection material (W/m°C)

t is the fire resistance period (mins)

θ_{cr} is the critical temperature of beam (°C)

In this Example, use the following parameters:

$A_m/A = 200m^{-1}$ (typical of IPE beams)

$\lambda_i = 0{,}15$W/m°C (typical of many protection materials)

t = 60 mins

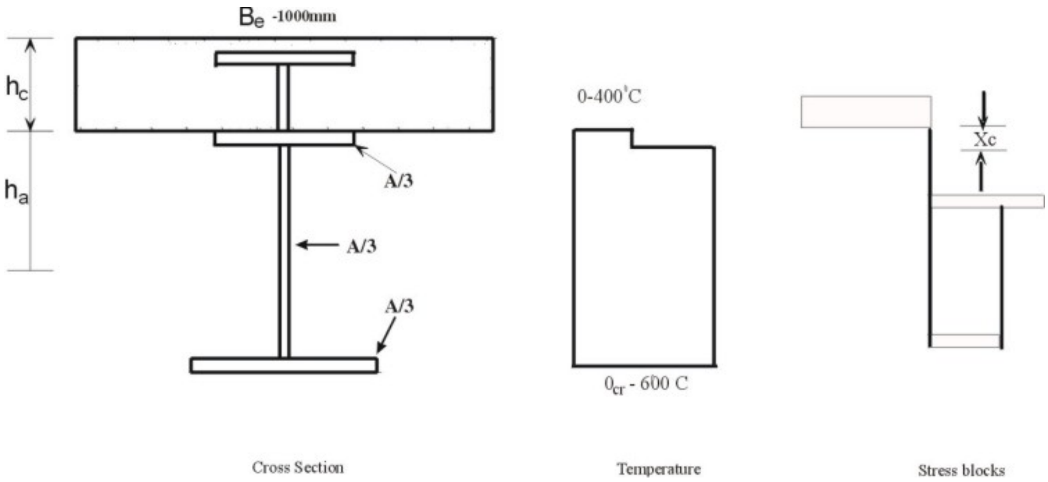
$\theta_{cr} = 645$°C

$d = 0{,}0083 \text{ x } 0{,}15 \text{ x } 200 \text{ x } \{60/(645 - 140)\}^{1{,}3} \times 10^{-3}$

= 15,6 mm (say 16 mm)

EXAMPLE 5 MOMENT RESISTANCE OF COMPOSITE BEAM

This Example follows the use of the principles of plastic analysis to calculate the moment resistance of a composite beam in fire conditions. The following properties are assumed:



Cross-section Temperature Stress blocks

Cross-sectional area of web = area of flange

Temperature of top flange = 2/3 x temperature of web and bottom flange

Effective breadth of slab = 1000 mm

Compressive strength of concrete f_c = 30 N/mm^2 (Note: for short term load in fire conditions γ_{mc} = 1,0)

Yield strength of steel f_y = 235 N/mm^2

Critical temperature of beam θ_{cr} = 600°C (assumed)

Neutral axis depth, x_c, in concrete is obtained by equating tension and compression. Hence:

$$f_c\, x_c \times 10^3 = \psi(600)\left(\frac{A}{3}+\frac{A}{3}\right) f_y + \psi(400)\,\frac{A}{3}\, f_y$$

$\psi(600) = 0,47$ }

} from Example 1

$\psi(400) = 1,00$ }

$$x_c = 0,64\,\frac{A\, f_y}{f_c\, B_e}$$

Moment resistance of composite section is obtained by taking moments about the mid-length of the concrete in compression:

$$M = \psi(600)\, A\, f_y \left(\frac{h_a}{2} + h_c - \frac{x_c}{2} \right)$$

$$+ \{\psi(400) - \psi(600)\} \frac{A}{3} f_y \left(h_c - \frac{x_c}{2} \right)$$

$M = 0{,}47\ A\ f_y\ (0{,}5\ h_a + 1{,}44\ h_c - 0{,}72\ x_c)$

For the following data:

$h_a = 400$ mm

$h_c = 120$ mm

$A = 1000$ mm^2

$x_c = 0{,}64 \times \dfrac{1000 \times 235}{1000 \times 30} = 5{,}0$ mm in fire conditions

$M = 0{,}47 \times 1000 \times 235 \times (0{,}5 \times 400 + 1{,}44 \times 120 - 0{,}72 \times 5) \times 10^{-6}$

= 40,8 kNm

By comparison under normal conditions, using $\psi = 1{,}0$, moment resistance is 74,2 kNm (note: this value may be calculated using the partial safety factors appropriate for normal conditions, as covered in the lecture on Composite Beams).

Therefore $M/M_u = 40{,}8/74{,}2 = 0{,}58$ for $\theta_{cr} = 600°C$

But $\psi(600) = 0{,}47$

If $\psi(600) = \kappa \times M/M_u$, it follows that:

$\kappa = 0{,}47/0{,}58 = 0{,}81$ for composite beams (compare to 0,7 for non-composite beams)

EXAMPLE 6 TIME-EQUIVALENT OF NATURAL FIRE

Refer to lecture1 Assume that the fire compartment may be characterized by the following parameters. The time equivalent is:

$T_e = c\ w\ q_f$ minutes

c = 0,10 for typical compartment properties

w = 1,5 for typical ventilation conditions

q_f = 450 MJ/m^2 for office buildings

T_e = 0,1 x 1,5 x 450 = 67,5 minutes

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