

STEEL COLUMN BEHAVIOUR IN FIRE CONDITIONS

by

Panayoula Braimi, Dip.Eng., M.I.Gre.E., M.N.F.P.A.

A thesis submitted in the fulfilment of the requirement for the Award of the
Degree of Master of Philosophy in Structural Fire Engineering.

Unit of Fire Safety Engineering
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DECLARATION

This thesis is the result of research work for the degree of Master of Philosophy undertaken in the Unit of Fire Safety Engineering, Department of Civil Engineering and Building Science, University of Edinburgh.

It is declared that all the work and results in this thesis have been carried out and achieved by the author herself and the thesis have been composed by her under the supervision of Dr. E.W. Marchant and Dr. N. Fairbairn.

Dedicated to my family

ACKNOWLEDGMENT

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ABSTRACT

Urban development has given rise to taller and more extensive buildings whose contents are of increasing commercial value and these offer an increasing risk to life at the same time. Therefore buildings should offer some resistance to the destructive effects of fire. These have led to the incorporation of design features to facilitate escape and in recent times resistance to the effects of fire. The practical features of the building which are involved in all these are walls, doors, floors, passages, corridors, beams, stairs, and columns. This project considers the study of columns exposed to standard fire tests (to BS 476 or to ISO 834), natural fires or real fires. The columns were pin-ended and axially loaded without applied moments.

TASEF-2 (Temperature Analysis of Structures Exposed to Fire) is a finite element computer program which has been used for calculating the temperature profiles throughout sections at any given time. Verification of the model has been done by comparing its results with those of experimental tests.

The column structural response to loading, which may be in terms of deflection or load bearing capacity is assessed using the following Codes: Eurocode 3: Part 10 [Eurocode 3, 1990], European Recommendations for the Fire Safety of Steel Structures [ECCS, 1983], BS 5950:Part 8: 1990 [BSI, 1990] and the Swedish Code [Pettersson, 1976]. Critical comparisons of the different Codes are also given and their advantages and disadvantages are discussed.

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CHAPTER ONE Introduction

1.1. The fire and its cost

The fire and its cost have increased over the centuries as society and technology have evolved. The concentration of people in towns and the growth of industries which handle flammable or explosive products such as oil, synthetics and chemicals increase the scale of fire risk.

Fire risk affects both human life and property.

The first category of risk can hardly be calculated in monetary terms. The statutory texts usually provide information to prevent loss of life in a fire incident.

In the case of damage of material goods as a result of combustion, corrosion or plant failure caused by high temperatures, it is the insurance company that defines the risks and determines premiums.

The possible effect of fire on the natural and social environment is a very important matter as well. A fire in industrial premises can lead to redundancies, while a fire in an oil refinery can result in pollution which is hazardous to the surrounding population.

1.2. Metal load - bearing structures

Though a number of different metals are used to some extent in building, only iron and steel (and to limited extent in recent years aluminium) are normally used for those parts which have to carry a load. They are not combustible and they present no risk of fire spread from direct burning. On the other hand, unprotected metal surfaces heat up in a fire and may cause fire spread by conduction. Load-bearing structural elements of unprotected metal collapse when excessively heated. Structural steel begins to lose strength above approximately 200-300°C but how quickly structural steel fails depends on its redundancy and degree of restraint. When metals are

heated, they expand. In a framed building, the failure of a single structural element will only cause local collapse. To reduce the chance of collapse, structural steel should be usually protected by a layer of non-combustible heat-insulating material.

The author's area of research is concerned with unprotected steel columns. The advantages of unprotected steel are reduced cost, specialist fire protection contractors are not required on site, less floor space is used, erection is fast and a good resistance to mechanical impact is achieved.

A Digest [BRE, 1986] published by the Fire Research Station states that large columns have a half-hour fire resistance inherently, without protection, provided the ratio of fire-exposed perimeter to cross-sectional area is sufficiently low (50m^{-1} or less). Smaller columns achieve the half-hour rating by using some form of protection, for example by filling the void between the flanges of the columns with a single layer of autoclaved aerated concrete blocks which protect the web and the inner flange faces from heat.

1.3. The structural fire protection

Fire safety systems are normally designed to minimize the occurrence of a disastrous fire and hazard potentials which are referred to as loss of lives, property, use and environmental mishap. Fire safety can be provided by active or passive measures or a combination of both. The active measures come into operation on the occurrence of a fire. They comprise fire detection and fire control systems. The passive measures are part of the built system and are functional at all times. They include building layout, design and construction. Measures may interact, the provision of the activation of a system can have beneficial effects on another. It is difficult for the designer to decide how safe the project is going to be, to distinguish the relative importance of fire safety strategies and to find out about the trade-offs involved.

The safety of a structure against fire - one of the fire protection strategies - is governed by:

- a/ the risks involved in case of a severe fire considered as an accidental situation,
- b/ the risk-reducing effect of conventional measures,
- c/ the risk-reducing effect of non-structural measures such as detection, alarm systems, sprinkler systems.

The provision of non-structural measures may result in a reduced level of structural fire safety becoming acceptable.

Structural fire protection engineering involves the design of structural building elements to resist the effect of fire. For individual structural elements, an increase in resistance is achieved by increasing member size or by providing adequate insulation in the case of steel and by increasing the thickness of concrete around the reinforcing and prestressing steel in the case of concrete.

Historically, the basis of the measurement of fire resistance for structural elements and assemblies has been the standard fire resistance test. A major difficulty with using the standard fire resistance tests' results or predicted results is that the response of full size assemblies to real fires cannot be measured. In recent years, there has been several attempts to predict test results by calculation as full scale testing is not practical.

In order to predict the response of a structure in a fire, three basic components has to be considered:

1. the fire,
2. the heat transfer to the structural members,
3. the affect of elevated temperature on structural performance.

The above components are not independent, they are interrelated in a very complex manner. A design method requires realistic simplification of the

complex problem in a manner which will yield consistent results with known levels of reliability.

Because the steel structural system has virtually no impact on a compartment fire, one can isolate the fire from the other two basic components (the heat transfer to the structural elements and the resulting structural performance). There are computer programs that predict the development of a fire and its different phases. In this research work, the temperature-time curves presented in literature such as the standard temperature-time curve or the hydrocarbon curve adopted by the U.K. Department of Energy are only used.

Once the fire is defined, the temperature distribution in the structure must be determined. The heat transfer problem is very complicated and the modes of heat transfer are dependent on the type of structural system. Different materials have different thermal protective systems. In the case of structural steel, three main systems are usually considered:

1. unprotected steel,
2. insulated steel,
3. membrane protected steel (such as suspended ceiling).

Usually, there are difficulties related to the input of data concerning the material properties of the insulating material because there is limited information on their temperature dependent material properties such as the density, conductivity and specific heat of the material.

The most difficult case to model is the membrane protected steel because in addition to the heat transfer properties, its thermal performance depends strongly on its integrity.

The behaviour of unprotected steel in a fire has been investigated using the model TASEF-2 [Sterner, 1990] to model the heat transfer from the fire to the unprotected structural steel. Results obtained using TASEF-2 are presented in Chapter Three (3).

Once the time dependent temperature gradient in a structural member has been determined, the effect on the structure may be analysed. A structure is said to have failed if either the ultimate limit or the serviceability limit is reached.

A serviceability limit is reached if the strains or the deflections are such that the columns deflect excessively producing an unsightly appearance.

The ultimate limit state is reached if excessive local damage causes deterioration of the material to such an extent that plastic hinges begin to form around the structure, reducing the statically indeterminate structure to a mechanism.

The response of steel columns to high temperatures was studied using different design Codes. No existing computer program was made available to the Unit for implementation and further conversion at a reasonable price. This is the reason why a computer program is not used for the structural analysis.

Non-linear instability effects are excluded from this study.

1.4 The objectives of this work

The author's research work focusses on steel columns and their behaviour in a fire environment. The study of three cases is presented in Chapter Three(3). These cases are :

1. a universal column exposed to fire from four sides in a fire compartment,
2. a universal column built next to a wall in a fire environment,
3. a universal column built into a wall which is part of a boundary which belongs to a fire compartment.

In the existing computer programs, the thermal analysis is not integrated with the mechanical analysis. Sometimes, the thermal and mechanical analyses

are carried out using different programs. According to such analyses, the time-temperature of the steel structures is calculated and is used as an input information for the analytical prediction of the mechanical behaviour. This is the reason why this thesis is divided into two parts elaborating the thermal and structural behaviour of columns respectively.

In the first part of the thesis (Part I), correlations are made among the standard tests' results taken from " The Compendium of U.K. Standard Fire Test Data" [Wainman, 1988] , the theoretical results obtained by using the computer program named TASEF-2 (Temperature Analysis of Structures Exposed to Fire) and the Pettersson's method [Pettersson, 1976]. Correlations between full scale test results [Almand, 1989] and results from TASEF-2 are presented as well.

In the second part (Part II), the existing Codes are presented and analysed.

The existing Codes studied are:

- BS 5950: Part 8: Code of practice for fire resistant design [BSI, 1990],
- ECCS: European Recommendations for Steel Structures [ECCS, 1983],
- Swedish Code for Design of Steel Structures [Pettersson, 1976].

A design example using the above Codes is presented in Chapter 6. Non linear instability effects are excluded.

1.5 Future trends

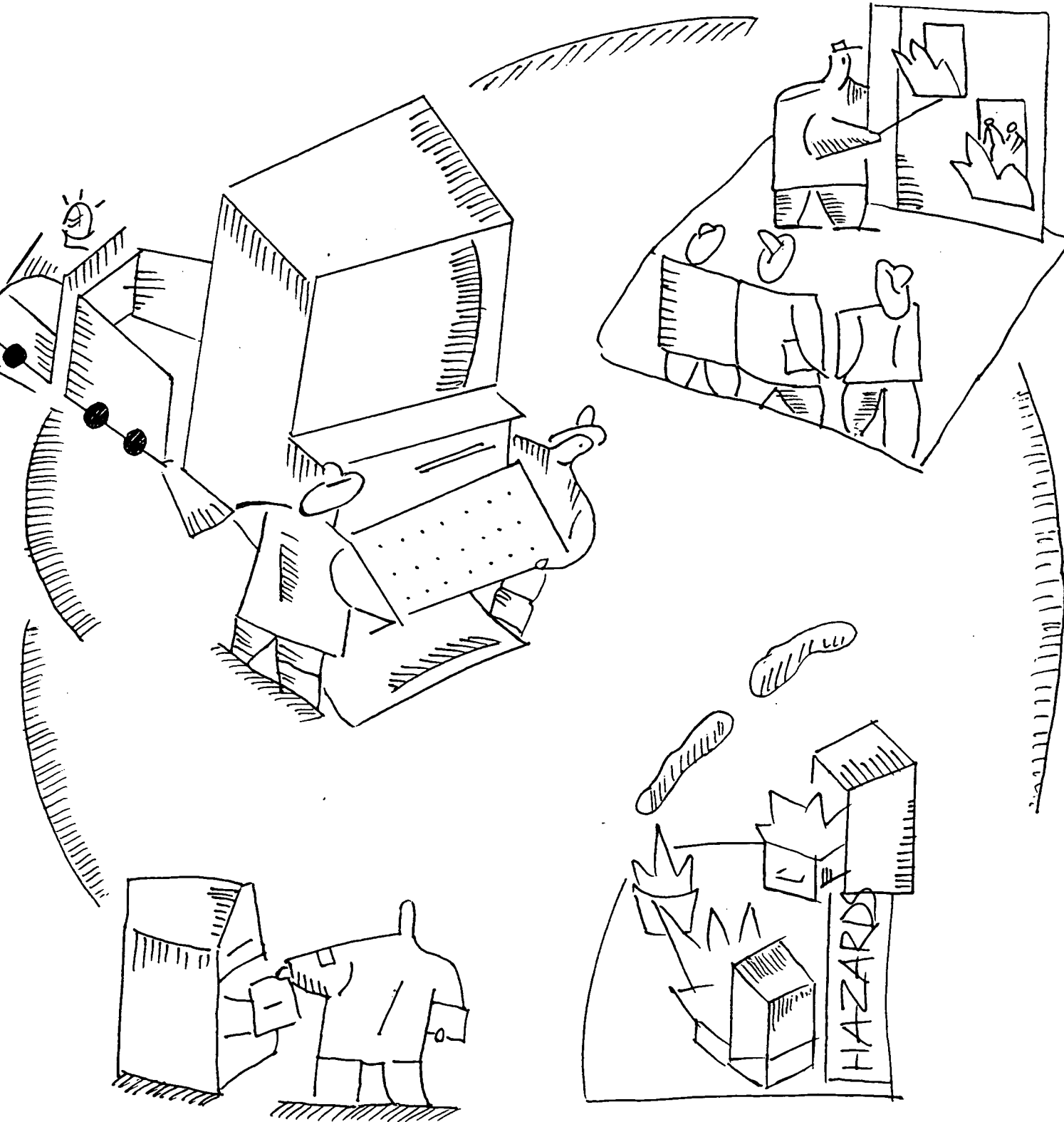
The computer programs have still to be developed.

Unknown mechanical properties of various building materials at elevated temperatures must be investigated in different fire conditions and be used as input to the structural programs.

A three- dimensional framework program to take account of fire will be an important development of the existing computer programs.

Factors like plastic hinge rotation, finite deformations leading to the problem of buckling and instability in individual members and in part or all of a framework must be investigated further and incorporated in the analysis of the computer programs.

PART I: Thermal Problem



CHAPTER TWO Physical and Thermal Properties

The material properties can be divided into four groups [Malhotra, 1982]:

- Chemical (decomposition, charring),
- Physical (density, expansion, softening, melting, spalling),
- Mechanical (yield strength, elasticity, tensile strength, creep),
- Thermal (thermal conductivity, specific heat capacity, thermal diffusivity, thermal inertia).

In the case of steel, the chemical properties are not of interest - only wood is subject to decomposition and charring. Softening of steel will occur at temperatures higher than 800°C. Melting of steel is unlikely to happen at the maximum temperatures (1200°C) usually experienced in fires.

The discussion of the mechanical properties is given in Chapter Four of this study.

2.1 Physical Properties

2.1.1 Density.

For all steel qualities, the value of $\rho=7850 \text{ kg/m}^3$ should be taken for the calculations, regardless of the temperature history.

2.1.2 Thermal strain.

When a material is heated, its thermal movement depends on the coefficient of linear expansion and on the restraint imposed on its ends. If the material is unrestrained and under constant temperature, there is no stress associated with the strain. Where the material is restrained, for materials that follow Hooke's law in the elastic range - like metals - there is less complication than for inorganic composite materials where Hooke's law does not apply due to cracking and phenomena associated with loss of water and phase changes in the cement matrix and aggregate upon heating.

If the temperature of the solid material is raised by T , the material elongates according to the equation:

$$L_t = L_o(1 + T + \alpha_1 T^2 + \alpha_2 T^3) \quad (2.1)$$

where: L_t = length under a temperature rise of T ,

L_o = length at the initial temperature,

α = coefficient of thermal expansion.

For most purposes, the following equation is adequate:

$$L_t = L_o(1 + \alpha T) \quad (2.2)$$

This happens because, for pure metals, the constants $\alpha, \alpha_1, \alpha_2$ have values $10^{-5}, 10^{-11}, 10^{-14}$ respectively and the values of α_1, α_2 are considered negligible in comparison with α . The coefficient of thermal expansion is determined in tests under stress-free conditions.

According to BS 5950: Part 1: 1985 the coefficient of thermal expansion at ambient temperatures is defined by $\alpha = 1.2 \times 10^{-5} / ^\circ\text{C}$.

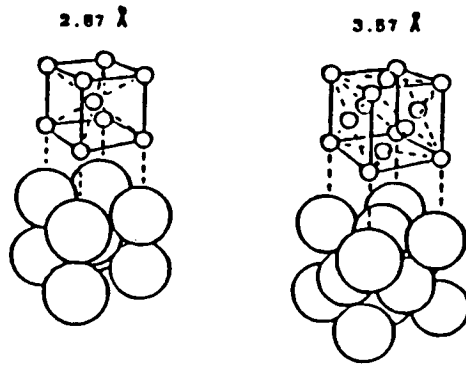
2.1.2.1 Variation of thermal expansion with temperature

The coefficient of thermal expansion depends on steel temperature. In 1953, the National Physical Laboratory reported on the thermal movement of 22 different steels at elevated temperatures (Cooke, 1988). The research was sponsored by the British Iron and Steel Research Association. According to this research, for steel with a carbon content of 0.23% - which is nearest to structural steel in chemical composition - the coefficient of thermal expansion varies from a mean value of $12.18 \times 10^{-6} / ^\circ\text{C}$ in the range 0-100°C to $14.81 \times 10^{-6} / ^\circ\text{C}$ in the range of 0-1200°C. Because structural steel begins to lose strength at temperatures above 550°C, the temperatures of main interest are in the range 0-550°C over which

the mean value of the coefficient of thermal expansion is $\alpha=14.17 \times 10^{-6} / ^\circ\text{C}$ (Cooke,1988). From the same reference and for temperatures greater than 700°C , it is observed that the coefficient of thermal expansion reduces with further increase in temperature. This phenomenon is called phase transformation, it is caused by the transformation of pearlite to austenite and is accompanied by a rearrangement in the atomic structure from the body-centred cubic structure to the face-centred cubic structure (Fig. 2.1)[Walker J.,1984 - Kennedy R. et al., 1970]. The magnitude of the actual shrinkage and temperature of onset of phase transformation depend on the chemical composition (Fig. 2.2). According to tests that BSC were commissioned to conduct, the variation of heating rate does not affect the temperature at which the phase transformation commences and ceases but does affect the magnitude of shrinkage (Fig.2.3.). A high precision MMC High Speed Vacuum Dilatometer was used for these experiments. Two specimens were taken from the same piece of structural steel containing 0.28% C and 0.67% Mn. It is certain that ignorance of phase transformation leads to strain overestimate at temperatures where the onset of phase transformation occurs. However, in practice, the steel structural members fail before reaching the phase transformation temperature. In that case, phase transformation is not considered as sufficiently an important factor to affect the behaviour of structural steel.

Codes

The European Recommendations for the Fire Safety of Steel Structures by the European Convention for Constructional Steelwork (1983), BS 5950:Part 8:1990, and Eurocode: Part 10,1990: Structural Fire Design consider, as an approximation, that the coefficient of thermal expansion is independent of steel temperature and may be taken as $\alpha=14 \times 10^{-6} / ^\circ\text{C}$. ECCS recommends this value for the grades of steel Fe 310, Fe 360, Fe 510 as specified in the Euronorm 25-72. BS 5950 recommends it for hot finished structural steels that comply with BS



Ferrite B.C.C. Austenite F.C.C.

Fig. 2.1 Crystal structures of iron (Kirby,1986)

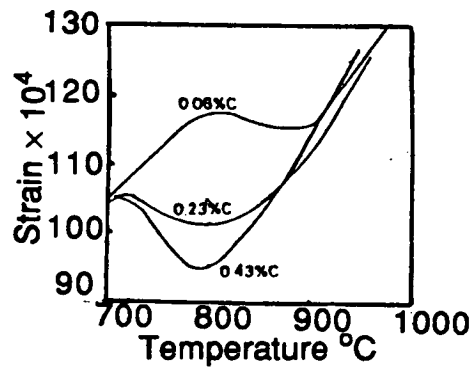


Fig. 2.2 Thermal expansion - temperature curves for low and medium carbon steels in the phase transformation range (Cooke, 1988).

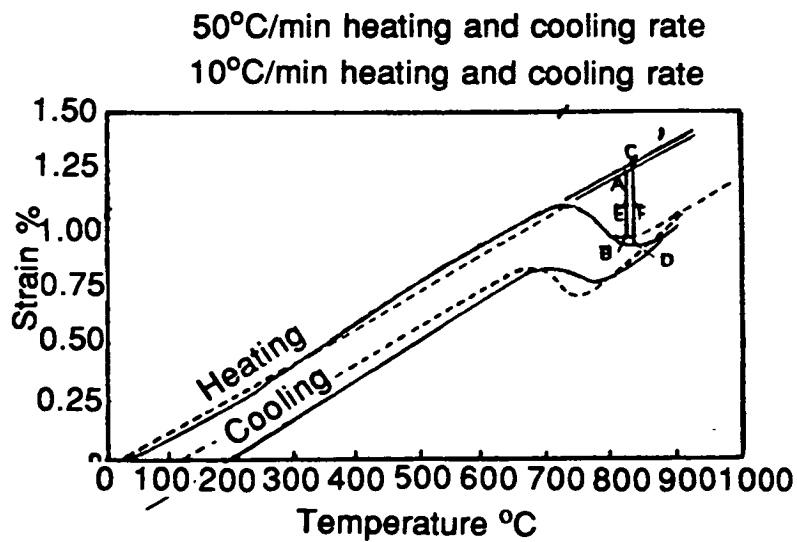


Fig. 2.3 Dilatometer curves for a mild steel showing effect of different heating and cooling rates (Cooke, 1988).

4360 at elevated temperatures. EC3 recommends that this value is valid for steel qualities Fe 310, Fe 360, Fe 430, Fe 510 as specified in the Euronorm 10025 and for Fe 460 as specified in EN 10113. In all other cases the reliability of the given design values must be demonstrated explicitly.

As an alternative, ECCS advises that the thermal elongation may be calculated by the following equation:

$$\frac{\Delta l}{l} = 0.4 \times 10^{-8} \theta_s^2 + 1.2 \times 10^{-5} \theta_s - 3 \times 10^{-4} \quad (2.3)$$

where: l = length at room temperature

Δl = temperature induced expansion

θ_s = steel temperature

The equation is illustrated in Fig. 2.4 for a practical temperature range. It does not illustrate the phenomenon of phase transformation.

EC3 recommends, for more explicit design, that the thermal elongation may be as well given by:

$$\begin{aligned} \frac{\Delta l}{l} &= -2.416 \times 10^{-4} + 1.2 \times 10^{-5} \theta_s^2 && \text{for } \theta_s \leq 750^\circ\text{C} \\ \frac{\Delta l}{l} &= 11 \times 10^{-3} && \text{for } 750^\circ\text{C} < \theta_s < 860^\circ\text{C} \\ \frac{\Delta l}{l} &= -6.2 \times 10^{-3} + 2 \times 10^{-5} \theta_s && \text{for } 860^\circ\text{C} < \theta_s < 1200^\circ\text{C} \end{aligned} \quad (2.4)$$

where: l = length at room temperature (m)

Δl = temperature induced expansion (m)

θ_s = steel temperature ($^\circ\text{C}$)

The graphical representation of elongation varying with temperature is given in Figure 2.4. It does take into account the phase transformation by defining the coefficient of thermal expansion as a constant equal to 11×10^{-3} over the temperature range 750°C - 860°C .

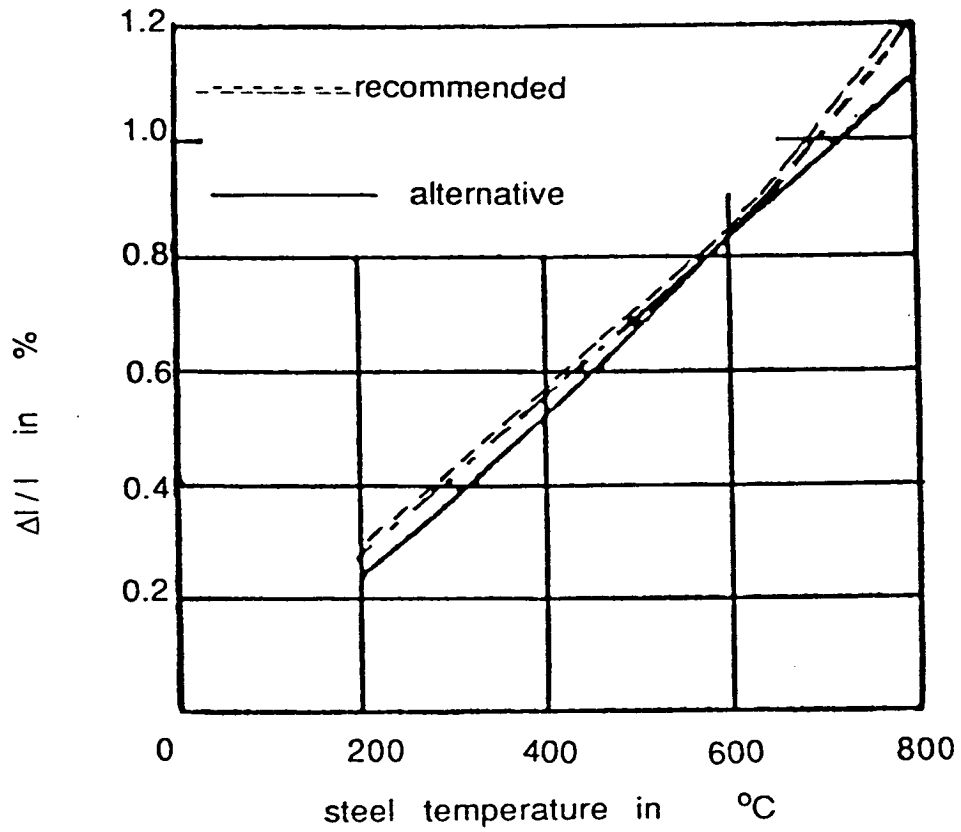


Fig. 2.4 Thermal expansion of steel as a function of the steel temperature (ECCS, 1983).

2.2 Thermal Properties

The thermal properties of materials which affect their behaviour when exposed to fire are:

- thermal conductivity;
- specific heat capacity;
- thermal diffusivity;
- thermal inertia.

2.2.1 Thermal Conductivity

Thermal conductivity characterizes the capability of various materials for transmitting heat. In buildings, the materials may experience three modes of heat transfer which are:

- thermal conduction by atomic and molecular vibration;
- thermal conduction by radiation;
- thermal conduction due to mass transfer (gaseous conduction).

The bonding between the molecules of an element influence its ability to transmitting heat when subjected to a heat flux. The heat energy is transmitted by very high frequency elastic waves. Because the frequency of variation for rigidly-bound molecules is high, the rate at which the energy is transferred is also high. The weakly- bonded molecules pass less heat energy through their bulk than the rigidly- bonded ones. The quantum of energy associated with such wave motion is called the phonon. A metal is considered to be a good conductor because the electrons move from regions of high temperature to regions of low temperature. In an insulation-type material, the conduction is dominated by phonon-phonon collisions or phonon-lattice framework collision processes.

With an increase in temperature, the interaction between phonons increases and causes the thermal resistance to increase. So, the thermal conductivity decreases with an increase of temperature.

Surfaces which have a temperature above absolute zero are capable of transmitting and absorbing radiation. This happens because when radiation passes through solids, it undergoes scattering at structural imperfections, crystal boundaries and pores.

According to Stefan Boltzmann's law, the total emissive power by unit area of a black body is given by:

$$E = \sigma T^4 \quad \text{kW m}^{-2} \quad (2.5)$$

where: $\sigma = 56.7 \times 10^{-12}$ (Stefan-Boltzmann Constant),

$T = \text{temperature } (^{\circ}\text{C})$.

A black body is a surface which absorbs all radiation incident upon it.

This type of conductivity becomes quite significant at temperatures above 500°C for highly porous materials. For opaque-type materials, it becomes significant at about 1000°C or above.

In a material under thermal gradient with pores which are filled with air, the air can ease the heat energy to pass through the material. It has been shown that for a porous material the gaseous conduction component of the thermal conductivity makes a moderate contribution to the overall conductivity compared to the radiant component.

The thermal conductivity varies with temperature, density and moisture content. Example of the variation of thermal conductivity with temperature and density are given in Figures 2.5, 2.6 respectively.

When water displaces the air in the pore spaces, the thermal conductivity of a porous-type material becomes greater.

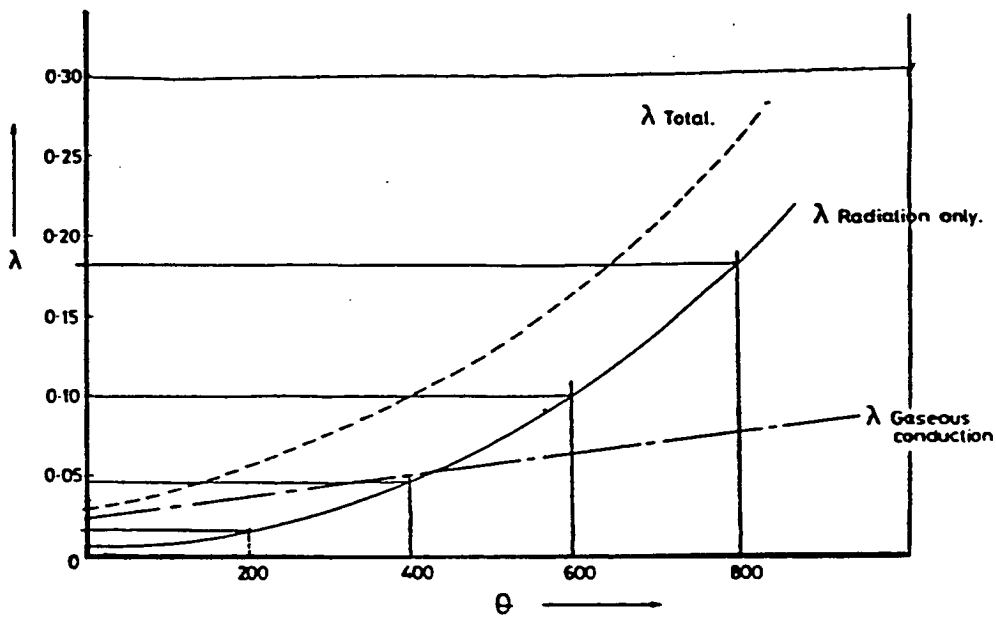


Fig. 2.5a Variation of the thermal conductivity with temperature for a medium - density fibrous insulant (Shield, 1987).

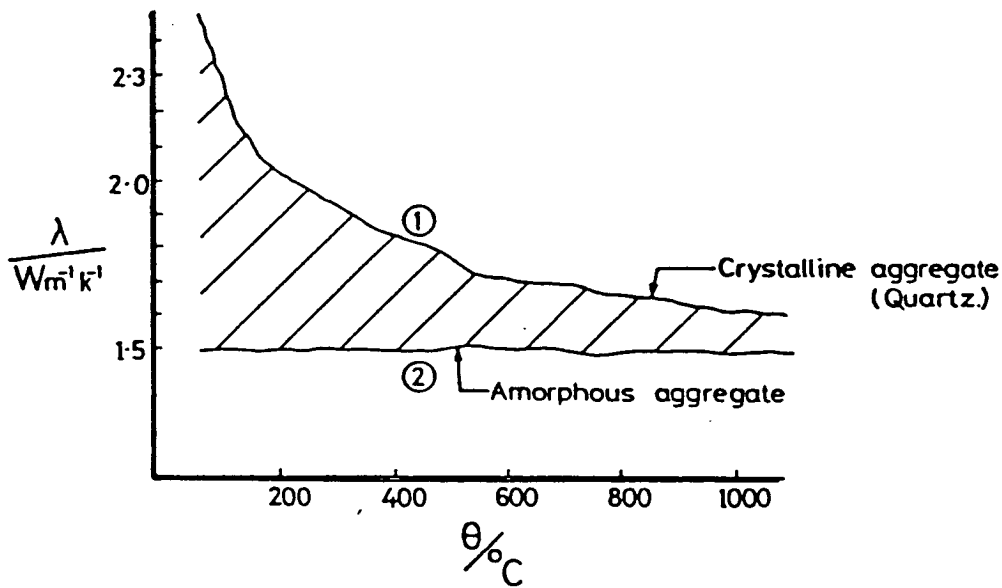


Fig. 2.5b Comparison of thermal conductivity variation with temperature for amorphous and crystalline materials (Shield, 1987).

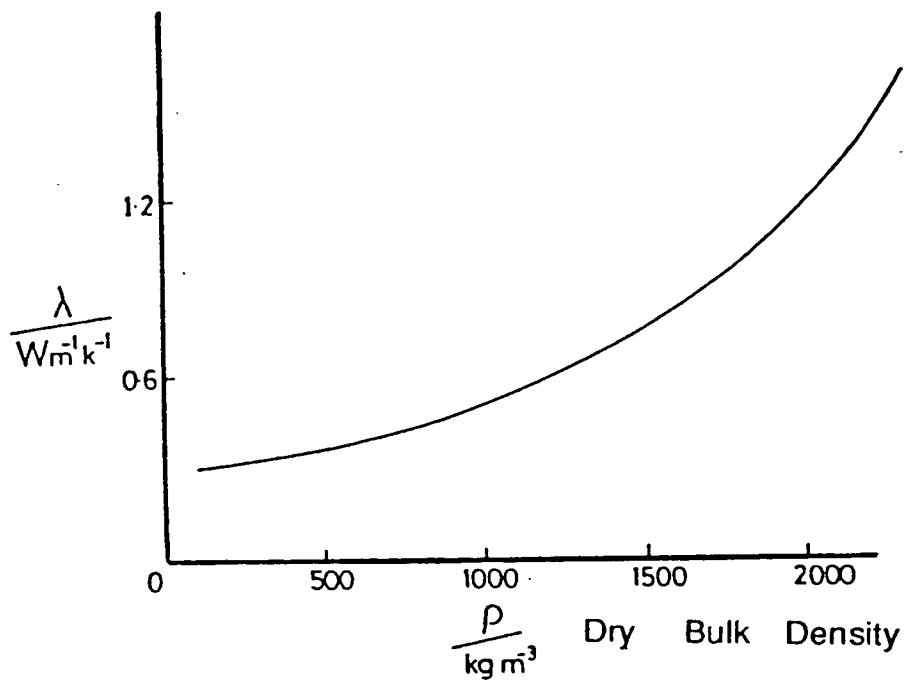


Fig. 2.6 Variation of thermal conductivity for porous materials with density (Shield, 1987).

BS 5950 :Part 8 [BSI,1990] recommends for the thermal conductivity of hot-finished steels (complying with BS 4360) a value of 37.5 [W/m °C]. This value is independent of temperature and is to be used in the fire calculations.

2.2.2 Specific Heat Capacity

Specific Heat Capacity is the quantity of heat that must be supplied to a unit quantity of substance in a particular process in order to change the temperature of the substance by one degree. Since the unit quantity of substance may be different e.g. 1 kg, 1 mole, 1m³, a distinction is made between the mass specific heat (J/kg K), the molar specific heat (J/mol K) and the volumetric specific heat (J/m³ K).

The variation of the specific heat capacity with temperature for chemically stable materials is given in Figure 2.7a. The same relation for complex materials such as gypsum is given in Figure 2.7b. It can be seen that the specific heat increases up to the temperature of 100°C which is the boiling point. This happens due to the removal of water and vapour in the pores, making the energy needed to raise 1 kg of a material by 1 K at this temperature much greater than would normally be the case.

ECCS [ECCS,1983] recommends, as an approximation, that the specific heat be considered independent of the steel temperature. In calculations and for all grades of steel, it may be taken as:

$$c_s = 520 \quad (\text{J / kg } ^\circ\text{C})$$

The same document gives an analytical relationship for the specific heat capacity to be determined as a function of temperature. The equation is:

$$c_s = 38 \times 10^{-5} \theta_s^2 + 20 \times 10^{-2} \theta_s + 470 \quad [\text{m/m}] \quad (2.6)$$

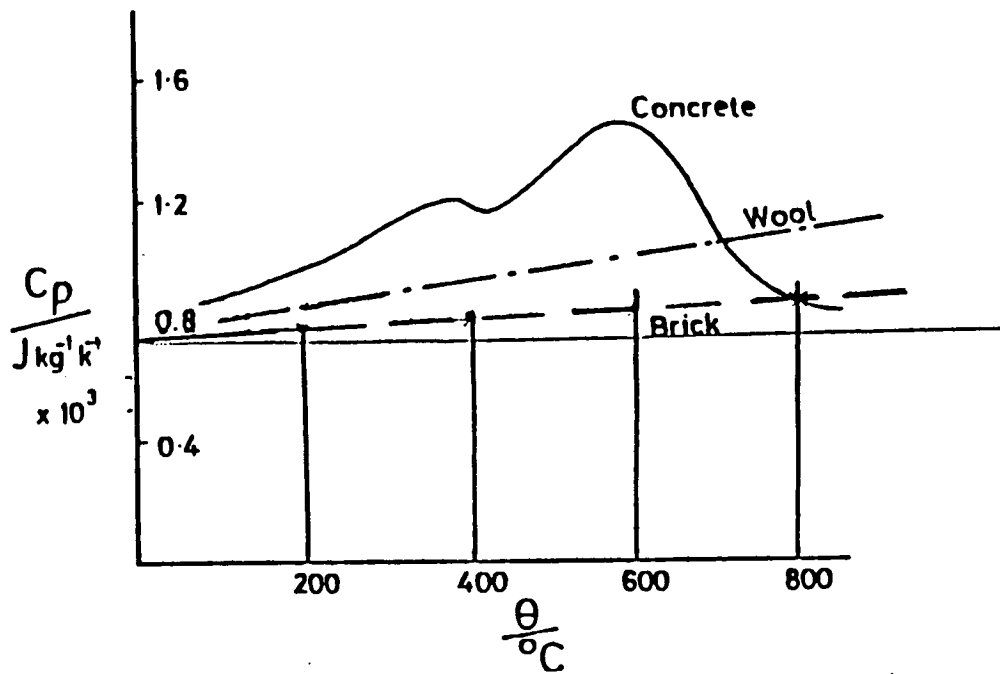


Fig. 2.7a Variation of specific heat capacity of chemically stable materials (Shield, 1987).

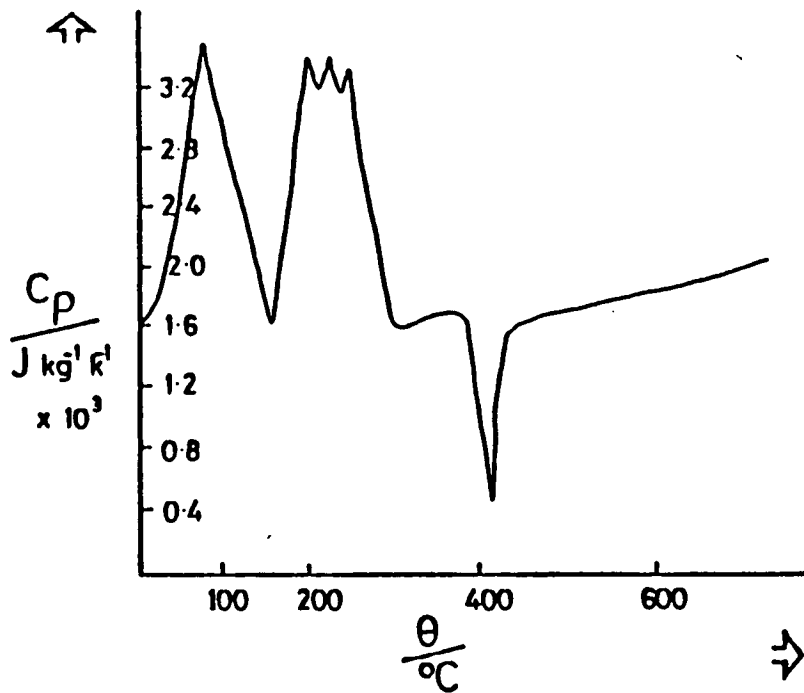


Fig. 2.7b Variation of specific heat capacity of gypsum with temperature (Shield, 1987).

The relationship is illustrated in Figure 2.8 for a practical temperature range. The dashed line in the same figure represents the approximation of a specific heat capacity value independent of temperature.

BS 5950: Part 8 [BSI, 1990] recommends that a value for the specific heat capacity of hot finished structural steels (complying with BS 4360) of 520 [J/kg °C] be used in fire calculations.

2.2.3 Thermal Diffusivity

Thermal Diffusivity is a measure of a material's ability to conduct heat energy in relation to its thermal storage capacity. It also gives a measure of the rate at which thermal energy can travel through a material which in turn controls the rate of temperature rise within the material.

The thermal diffusivity may be defined as follows:

$$\alpha = \frac{\lambda}{\rho C_p} = \frac{\text{thermal conductivity of material}}{\text{density of material} \times \text{specific heat of material}} \quad (2.7)$$

The variation of the diffusivity of steel with temperature is given in Figure 2.9a,b. ECCS [ECCS, 1983] recommends density a value of 7850 kg/m³. This value is independent of temperature and valid for all grades of steel.

2.2.4 Thermal Inertia

Thermal Inertia may be defined as the product of thermal conductivity, density and specific heat capacity ($\lambda\rho C_p$). It can be easily calculated if the variation of thermal conductivity and specific heat with temperature are known.

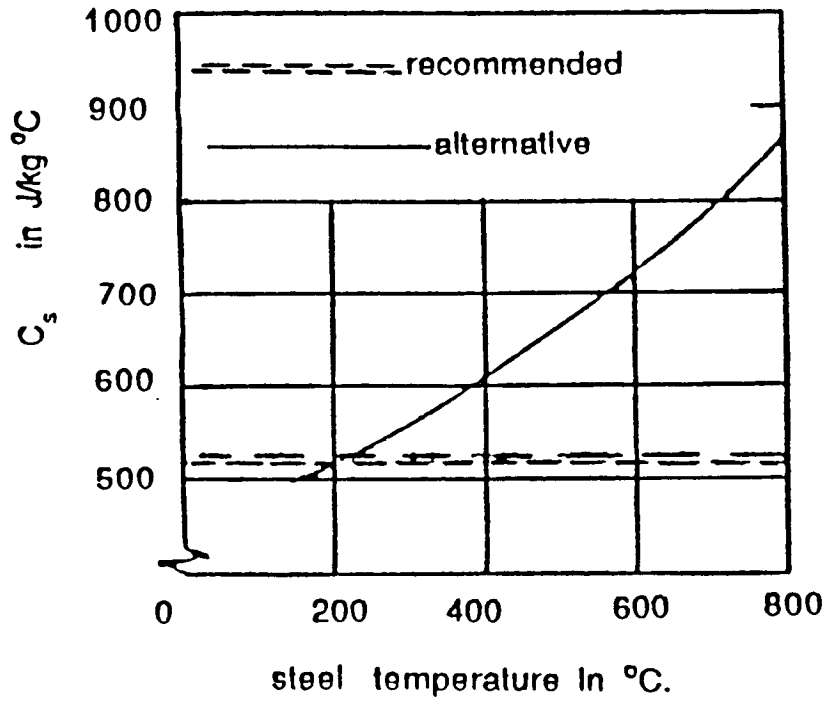


Fig. 2.8 Specific heat of steel as a function of the steel temperature (ECCS, 1983).

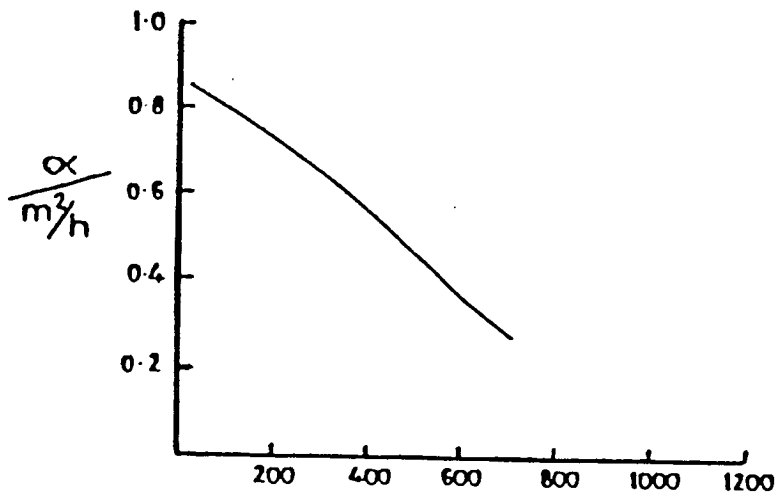


Fig. 2.9a Variation of the diffusivity of steel with temperature (Shield, 1987).

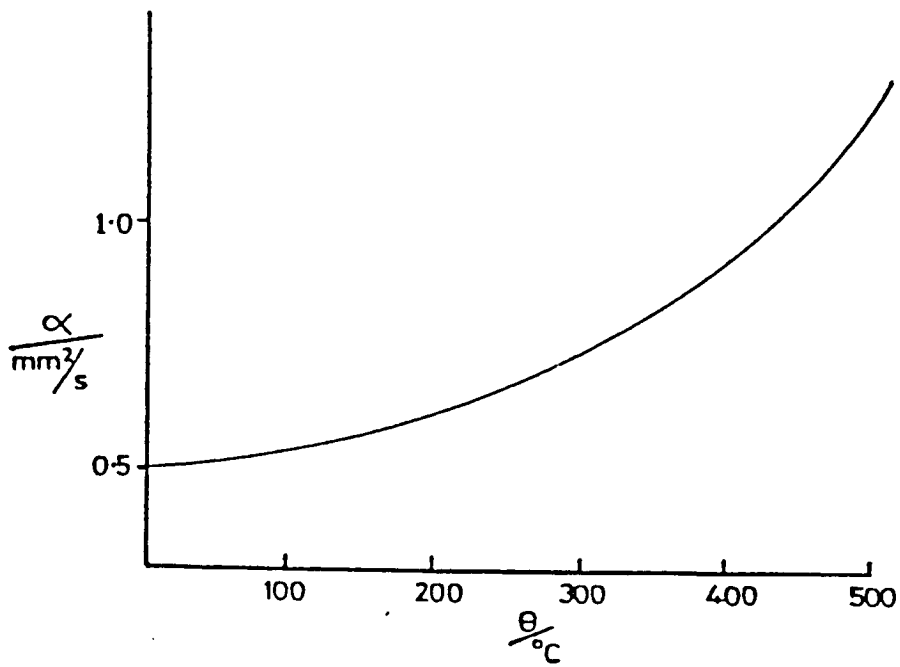


Fig. 2.9b Variation of the diffusivity of masonry materials with temperature (Shield, 1987).

CHAPTER THREE Heat transfer analysis

3.1 Mechanisms of Heat Transfer

Three basic mechanisms of heat transfer can be distinguished:

- conduction;
- convection;
- radiation.

Conduction is the method of heat transfer effected by microparticles of bodies (molecules, atoms and electrons) which possess different levels of energy and can exchange this energy as they move and interact. It is the basic mode of heat transfer within solids. It occurs in gases and liquids but it is not the principal mode of heat transfer.

Convection is the transfer of heat by the mass of a liquid or gas as it moves from a region of one temperature to a region of a different temperature. It is the basic mode of heat transfer for liquids and gases.

Radiation is the process of transfer of the internal energy of a body in the form of radiation energy. It requires no intervening medium between the source and the receiver.

Under real conditions, heat transfer often occurs by two or even three mechanisms simultaneously.

3.1.1 Theoretical Model

The non-linear heat flow equation must be solved to predict the distribution of the temperature in the structure exposed to fire. Because analytical solutions of such equations exist only for idealized cases, finite element or finite difference methods must be employed to approximate heat conduction.

The governing equations for heat conduction are given below:

- the heat balance equilibrium equation

$$-\nabla^T \mathbf{q} + \dot{e} - Q = 0 \quad (3.1)$$

- the Fourier law

$$\mathbf{q} = -\underline{\mathbf{k}} \nabla T \quad (3.2)$$

where:

\mathbf{q} = the heat flow vector,

$e = \frac{\partial e}{\partial t}$ = the rate of specific volumetric enthalpy change,

Q = the rate of internal generated heat per unit volume,

$\underline{\mathbf{k}}$ = a symmetric positive definite thermal conductivity matrix,

T = temperature,

t = time,

∇ = the gradient operator.

From the above:

$$-\nabla^T (\underline{\mathbf{k}} \nabla T) + \dot{e} - Q = 0 \quad (3.3)$$

For isotropic materials, the thermal conductivity matrix is given as follows:

$$\underline{\mathbf{k}} = k \underline{\mathbf{I}} \quad (3.4)$$

where:

$\underline{\mathbf{I}}$ = the identity matrix

The specific volumetric enthalpy is defined as follows:

$$e = \int_{T_0}^T c \rho \, dT + \sum_i l_i \quad (3.5)$$

where:

T_0 = the reference temperature (usually zero),

c = the specific heat,

ρ = the density,

$l_i =$ the latent volumetric heat due to phase changes at various temperature levels

The time derivative of the above equation is:

$$\dot{e} = c \rho \dot{T} \quad (3.6)$$

where: $\dot{T} = \frac{\partial T}{\partial t}$ = the rate of temperature change

Substituting, the conventional form of the transient heat flow equation is given as follows:

$$-\nabla^T (\underline{k} \nabla T) + c \rho \dot{T} - Q = 0 \quad (3.7)$$

In order to solve the above equations, one must specify initial and boundary conditions.

An initial condition is given by specifying the distribution of temperature in a body at zero reference time .

Boundary conditions are given as temperature or heat flow on parts of the boundary (∂V_T and ∂V_q respectively):

$$\partial V = \partial V_T + \partial V_q \quad (3.8)$$

where:

$T = T(x, y, z, t)$ = temperature,

$q_n = \underline{n}^T \underline{q} = -\underline{n}^T \underline{k} \nabla T$ = prescribed heat flow ,

with \underline{n} = the outward normal to the surface.

Heat transfer phenomena it is difficult to model. If approximate formulas are used the convection and radiation heat transfer is given by the equations.

The convective heat transfer is given as follows:

$$\hat{q}_n^c = \beta (T_s - T_g)^\gamma \quad (3.9)$$

where:

\hat{q}_n^c =the rate of heat transferred by convection,

β, γ =the convection factor and power respectively,

T_s =surface temperature,

T_g =surrounding gas temperature.

The radiation heat transfer is given as follows:

$$\hat{q}_n^r = \epsilon_r \sigma (\bar{T}_s^4 - \bar{T}_g^4) \quad (3.10)$$

where:

σ =the Stefan-Boltzmann constant,

T_s =absolute surface temperature,

T_g =absolute surrounding gas temperature.

ϵ_r =resultant emissivity

The resultant emissivity depends on the surface properties and geometric configuration. In fire engineering design, when assessing radiation between flames and structures, it may be assumed for the calculation that the case is similar to radiation between two infinitely long parallel planes. For the latter case, the emissivity is given by the following equation:

$$\epsilon_r = \frac{1}{\frac{1}{\epsilon_s} + \frac{1}{\epsilon_g} - 1} \quad (3.11)$$

where:

ϵ_g = appropriate gas or flame emissivity.

The total heat flux at a boundary is calculated as follows:

$$\hat{q}_n = \hat{q}_n^c + \hat{q}_n^r \quad (3.12)$$

3.2 Temperature Analysis of Steel Columns

The temperatures attained by a structural element may be assessed in four different ways:

- conducting a standard fire test;
- conducting a full scale test;
- using the codes;
- using the existing computer programs.

3.2.1 Standard fire tests

The fire resistance of load bearing structural elements is currently assessed in the U.K., in accordance with the British Standard 476: Part 20,21,22 [BSI, 1987].

Generally, a fire resistance test is carried out on a specimen which is as far as possible, representative of the structural element in terms of its size, materials and workmanship.

Loads are also applied to simulate the same magnitude and type of stresses generated in practice.

The test specimen is heated in a gas fire furnace in which the temperature is controlled to vary with time in accordance with the BSI recommendations [BSI, 1987]. The fire test is terminated either at the request of the sponsor or when the limiting requirements for maintaining the relevant criteria for stability, integrity, insulation are achieved. At the end of the heating period, as previously defined, the loads applied to the structural element under consideration may be removed and reapplied after twenty four (24) hours. The reload test is optional. If collapse occurs during the test procedure, the

DATA SHEET NUMBER **40a**

Column

DIMENSIONS AND PROPERTIES

SECTION SERIAL SIZE AND TYPE mm	DIMENSIONS AND PROPERTIES	MASS PER METRE kg	DEPTH OF SECTION mm	WIDTH OF SECTION mm	THICKNESS		ELASTIC MODULUS		PLASTIC MODULUS		MOMENT OF INERTIA	
					WEB mm	FLANGE mm	AXIS XX cm ³	AXIS YY cm ³	AXIS XX cm ³	AXIS YY cm ³	AXIS XX cm ⁴	AXIS YY cm ⁴
305 x 305	NOMINAL	198	339.9	314.1	19.2	31.4	2991	1034	3436	1576	50832	16230
COLUMN	ACTUAL		341	314	*	*						

CHEMICAL COMPOSITION (PRODUCT ANALYSIS - Wt.%) (*)

SECTION	STEEL QUALITY	C	Si	Mn	P	S	Cr	Mo	Ni	V	Cu	Nb	Al	N
COLUMN	GRADE 43A													

ROOM TEMPERATURE TENSILE PROPERTIES (*)

POSITION	LYS N/mm ²	TS N/mm ²	ELONG %
FLANGE			

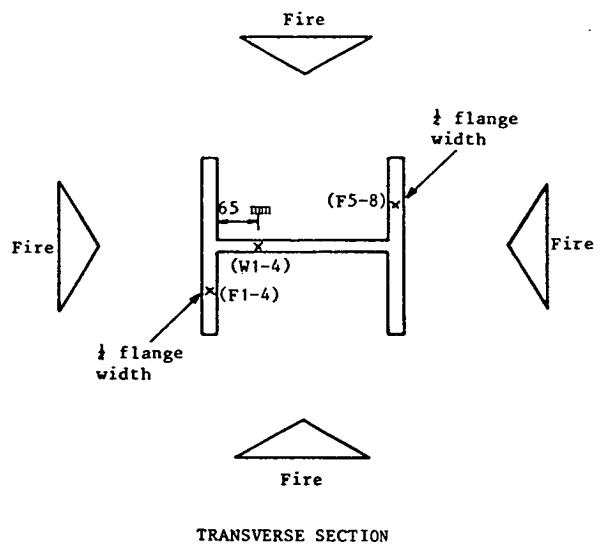
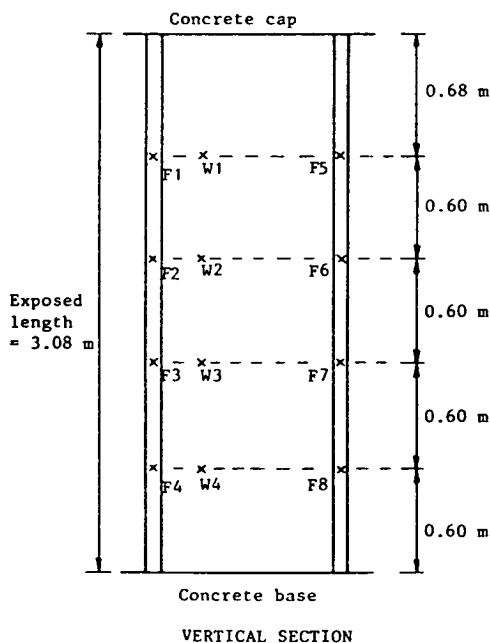
NOTES

- (a) After 20 minutes the loading frame collapsed, thereby prematurely terminating the loaded fire test. At that time the section had bowed approximately 10 mm. However the heating cycle was continued until 33 minutes.
- (b) Temperatures accurate to the nearest 5 deg. C
- (c) Initial ambient temperature = 10 deg. C
- (*) Data not available

TEST CONDITIONS

EXPOSED LENGTH	: 308	cm
EFFECTIVE LENGTH	: 215.6	cm
RADIUS OF GYRATION (y-y)	: 8.02	cm
SLENDERNESS RATIO	: 26.88	
MAXIMUM AXIAL STRESS	: 144	N/mm ²
AREA OF CROSS SECTION	: 252	cm ²
MAXIMUM LOAD	: 3629	kN
LOAD APPLIED	: 3630	kN

THERMOCOUPLE POSITIONS



(Not to scale)

TEST CENTRE : FIRTO -- BOREHAMWOOD
 TEST DATE : 17th. MARCH 1980
 TEST NUMBER : TE 3646

BS 476 : PART 8 : 1972 ASSESSMENT

RE-LOAD TEST : SATISFIED
 STABILITY : 20 MINUTES
 FIRE RESISTANCE : 20 MINUTES

DATA SHEET NUMBER **40b**

(a)

THERMOCOUPLE LOCATION	TEMPERATURE Deg. C AFTER VARIOUS TIMES (MINUTES) (b)											
	3	6	9	12	15	18	21	24	27	30	33	
EXPOSED PLANGES	F1	50	85	140	215	300	380	465	530	585	625	665
	F2	40	70	140	210	300	395	485	560	620	660	700
	F3	30	75	145	240	320	445	540	610	660	695	725
	F4	30	85	150	230	345	405	490	565	620	665	700
	F5	35	75	130	200	280	360	435	495	560	610	650
	F6	30	60	120	200	285	375	455	520	590	635	670
	F7	40	75	150	235	305	410	495	570	640	680	715
	F8	35	90	155	240	330	405	475	535	600	635	670
	MEAN	35	75	140	220	310	395	480	550	610	650	685
EXPOSED WEB	W1	50	95	160	255	360	445	525	590	645	690	725
	W2	45	80	145	245	345	445	535	605	650	685	715
	W3	40	80	145	250	365	470	555	630	685	720	740
	W4	45	115	185	280	390	475	560	630	685	715	740
	MEAN	45	95	160	260	365	460	545	615	665	705	730
MEAN FURNACE GAS		435	500	620	730	740	765	795	810	835	830	845
STANDARD CURVE (c)		492	593	653	695	729	756	779	799	816	832	846
EXTENSION (mm)		(*)										

DATA SHEET NUMBER **48a**

Column in wall

DIMENSIONS AND PROPERTIES

SECTION SERIAL SIZE AND TYPE mm	DIMENSIONS AND PROPERTIES	MASS PER METRE kg	DEPTH OF SECTION mm	WIDTH OF SECTION mm	THICKNESS		ELASTIC MODULUS		PLASTIC MODULUS		MOMENT OF INERTIA	
					WEB mm	FLANGE mm	AXIS XX cm ³	AXIS YY cm ³	AXIS XX cm ³	AXIS YY cm ³	AXIS XX cm ⁴	AXIS YY cm ⁴
203 x 203	NOMINAL	52	206.2	203.9	8.0	12.5	510.4	173.6	568.1	263.7	5263	1770
COLUMN	ACTUAL		206	204	8.0	12.5						

CHEMICAL COMPOSITION (PRODUCT ANALYSIS - Wt.%)

SECTION	STEEL QUALITY	C	Si	Mn	P	S	Cr	Mo	Ni	V	Cu	Nb	Al	N
COLUMN	GRADE 43A	0.27	0.041	0.94	0.010	0.022	0.01	0.005	0.026	0.005	0.018	0.005	0.01	0.0033

ROOM TEMPERATURE TENSILE PROPERTIES (a)

POSITION	LYS N/mm ²	TS N/mm ²	ELONG %
FLANGE	285	492	25.5
WEB	287	472	28.0

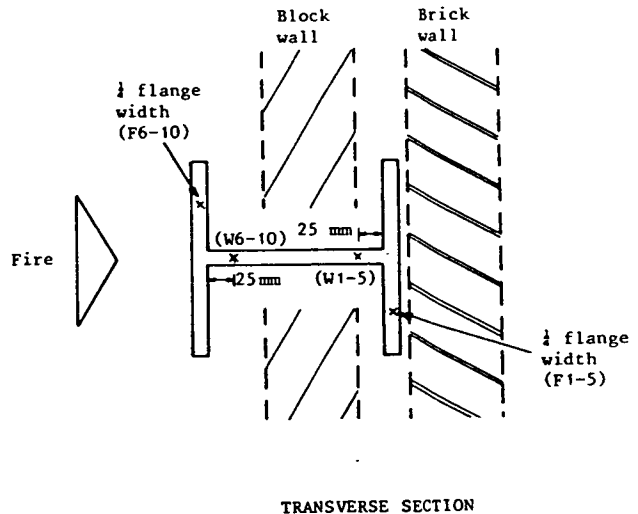
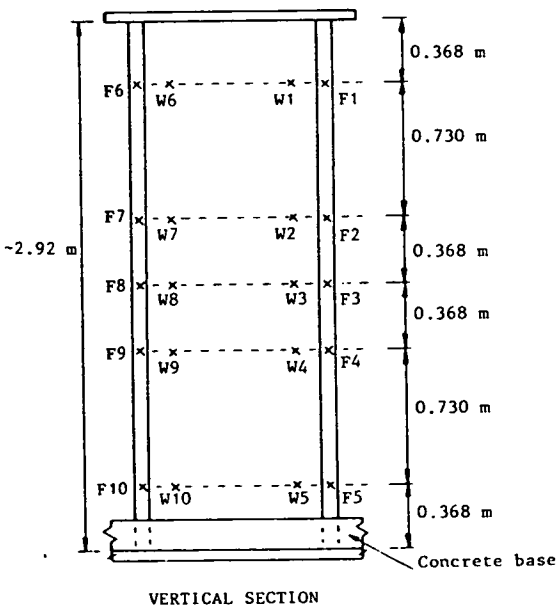
NOTES

- (a) For elevated temperature anisothermal tensile properties see Data Sheet No. 88
- (b) Equals 50% of the maximum permissible load according to BS 449 : Part 2 : 1969
- (c) Examination of the test behaviour by the British Steel Corporation suggests that for the x-x axis the effective length factor = 1.0 - 1.2 (estimate) and for the y-y axis the effective length factor = 1.0
 Effective length (x-x) = 300 - 360 cm
 Effective length (y-y) = 300 cm
 Slenderness ratio (x-x) = 33.71 - 40.45
 Slenderness ratio (y-y) = 58.14
 Hence the y-y axis governs collapse and the maximum allowable axial stress on the gross section = 127 N/mm²
 Maximum permissible load per column = 843.3 kN
 Therefore the load applied per column (476.5 kN) equals 56.5% of the maximum permissible load according to BS 449 : Part 2 : 1969
- (d) Initial ambient temperature = 19 deg. C
- (e) Accurate to the nearest 0.5 mm
- (*) Not measured

TEST CONDITIONS

HEIGHT OF COLUMNS	: 300	cm
EFFECTIVE LENGTH	: 255	cm
RADIUS OF GYRATION (x-x)	: 8.90	cm
RADIUS OF GYRATION (y-y)	: 5.16	cm
SLENDERNESS RATIO (x-x)	: 28.65	
MAXIMUM AXIAL STRESS	: 143.5	N/mm ²
AREA OF CROSS SECTION	: 66.4	cm ²
MAXIMUM LOAD PER COLUMN	: 953	kN
LOAD APPLIED PER COLUMN	: 476.5	kN (b)(c)

THERMOCOUPLE POSITIONS



(Not to scale)

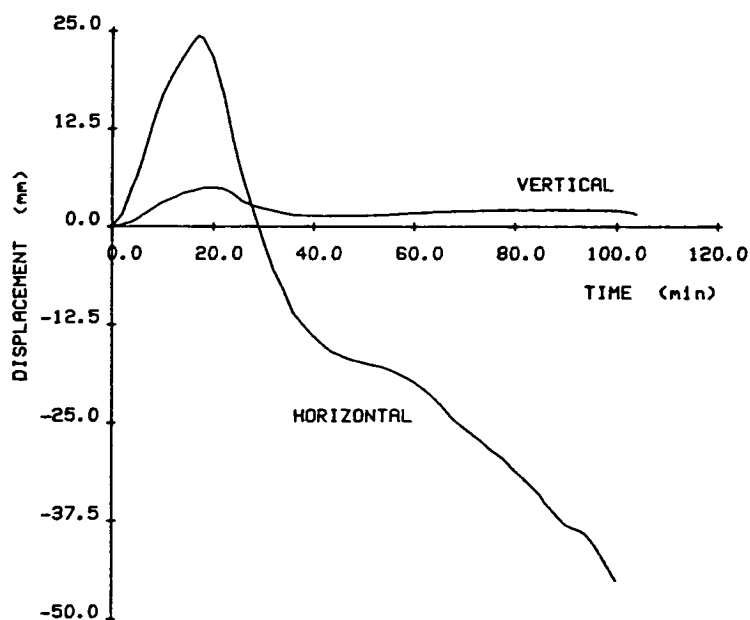
TEST CENTRE : FIRTO -- BOREHAMWOOD
 TEST DATE : 3rd. NOVEMBER 1981
 TEST NUMBER : TE 4081

BS 476 : PART 8 : 1972 ASSESSMENT

RE-LOAD TEST : SATISFIED
 STABILITY : 104 MINUTES
 INTEGRITY : 104 MINUTES
 INSULATION : 104 MINUTES
 FIRE RESISTANCE : 104 MINUTES

DATA SHEET NUMBER **48b**

THERMOCOUPLE LOCATION	TEMPERATURE Deg. C AFTER VARIOUS TIMES (MINUTES)																				
	5	10	20	25	30	40	45	50	55	60	65	70	75	80	85	90	95	100	103	104	
UNEXPOSED FLANGE	F1	33	56	92	112	129	161	176	190	202	213	228	241	254	267	279	290	300	313	322	*
	F2	19	34	60	75	95	136	155	172	186	199	217	234	246	258	269	279	288	298	304	*
	F3	18	21	41	53	69	105	122	138	151	163	174	186	195	204	213	220	226	232	234	*
	F4	17	19	34	46	60	92	108	123	137	148	162	176	187	197	207	215	223	230	233	*
	F5	16	17	23	28	36	54	63	71	78	86	95	104	112	120	127	134	140	146	149	*
MEAN	21	29	50	63	78	110	125	139	151	162	175	188	199	209	219	228	235	244	248		
UNEXPOSED WEB	W1	37	66	125	150	174	220	241	259	276	292	309	326	340	354	367	379	391	403	411	*
	W2	27	54	118	154	190	250	274	295	314	332	358	378	393	409	424	437	450	464	474	*
	W3	20	33	82	115	144	190	210	228	244	259	277	295	307	320	331	341	350	358	364	*
	W4	20	32	76	106	132	178	199	220	236	251	278	300	315	330	343	355	366	376	382	*
	W5	18	23	43	61	80	112	125	137	148	158	169	181	191	200	209	218	226	234	238	*
MEAN	24	42	89	117	144	190	210	228	244	258	278	296	309	323	335	346	357	367	374		
EXPOSED WEB	W6	121	217	400	478	542	639	670	700	733	765	797	824	843	861	881	899	915	931	940	*
	W7	101	210	431	535	615	707	747	786	818	845	874	892	906	920	937	952	964	977	983	*
	W8	94	213	437	537	607	687	722	758	792	821	853	870	883	899	919	935	948	960	966	*
	W9	74	170	385	486	559	653	694	731	762	791	831	853	867	886	909	926	939	952	958	*
	W10	51	100	250	365	454	550	584	615	642	671	713	748	768	790	816	840	860	887	882	*
MEAN	88	182	381	480	555	647	683	718	749	779	814	837	853	871	892	910	925	941	946		
EXPOSED FLANGE	F6	155	283	513	604	668	743	772	804	836	862	887	902	914	928	944	957	970	983	989	*
	F7	133	284	559	665	727	801	838	866	889	908	933	944	953	966	981	993	1004	1014	1018	*
	F8	158	319	570	660	716	777	812	847	873	894	918	927	936	950	968	982	994	1005	1010	*
	F9	148	290	563	664	723	787	821	852	875	896	922	931	939	954	974	989	999	1010	1013	*
	F10	90	172	415	584	677	739	764	790	815	838	874	893	904	919	939	957	969	981	980	*
MEAN	137	270	524	635	702	769	801	832	858	880	907	919	929	943	961	976	987	999	1002		
MEAN FURNACE GAS	589	657	808	825	856	882	896	913	927	940	955	963	970	988	1001	1014	1024	1035	1033	*	
STANDARD CURVE (d)	575	677	780	814	841	884	901	917	931	944	956	967	978	987	996	1005	1013	1021	1025	1027	
DEFLECTION (mm)	6.5	16.7	21.5	8.4	-1.9	-13.9	-16.2	-17.3	-18.1	-19.7	-22.4	-25.5	-28.1	-30.9	-34.0	-37.8	-39.6	-45.0	*	*	
EXTENSION (mm)(e)	1.0	3.1	5.0	3.7	2.3	1.4	1.3	1.4	1.5	1.7	1.9	2.0	2.0	2.1	2.1	2.1	2.0	2.0	1.8	1.5	



notional period of stability is taken as the 80% of the time to failure. If collapse occurs during the reload period, the notional period to stability is taken as the 80% of the heating period. For columns, the fire resistance is determined from when the test is terminated or when the criterion of stability is no longer satisfied (the axial load can no longer be maintained).For columns built into a cavity wall, the fire resistance is determined from when the test is terminated or when the failure occurs under the criterion of stability or integrity or insulation.

In the following examples, the standard fire tests [Data sheet 40a,b and 48a,b] used for comparisons between analytical and fire tests' results are reported in the Compendium [Wainman, 1988]. They are in accordance to BS476: Part 8: 1972 [BSI, 1972] which has been superceded by BS476: Part 20,21,22 [BSI, 1987]. All the column tests reported in the above document have been conducted at FIRTO-Borehamwood.

For columns, which are the subject of the present research, the test assemblies used are given in Figures 3.1 and 3.2 [Wainman, 1988] for columns exposed on four sides and columns built into cavity walls respectively. Details of the design, construction and test procedures for the column tests are given in the relevant document [BSI, 1987].

3.2.2 Full scale tests

The standard fire tests do not accurately portray the response of structures to fire. Fire tests in actual structures with real contents are conducted in order to assess the development and severity of fires and evaluate the performance of structures in fires.

In the fourth example which follows and is concerned with assessment of a corner column's response to a fire, a full scale test conducted by BHP Melbourne Research Laboratories in 1985 is used for comparison with the

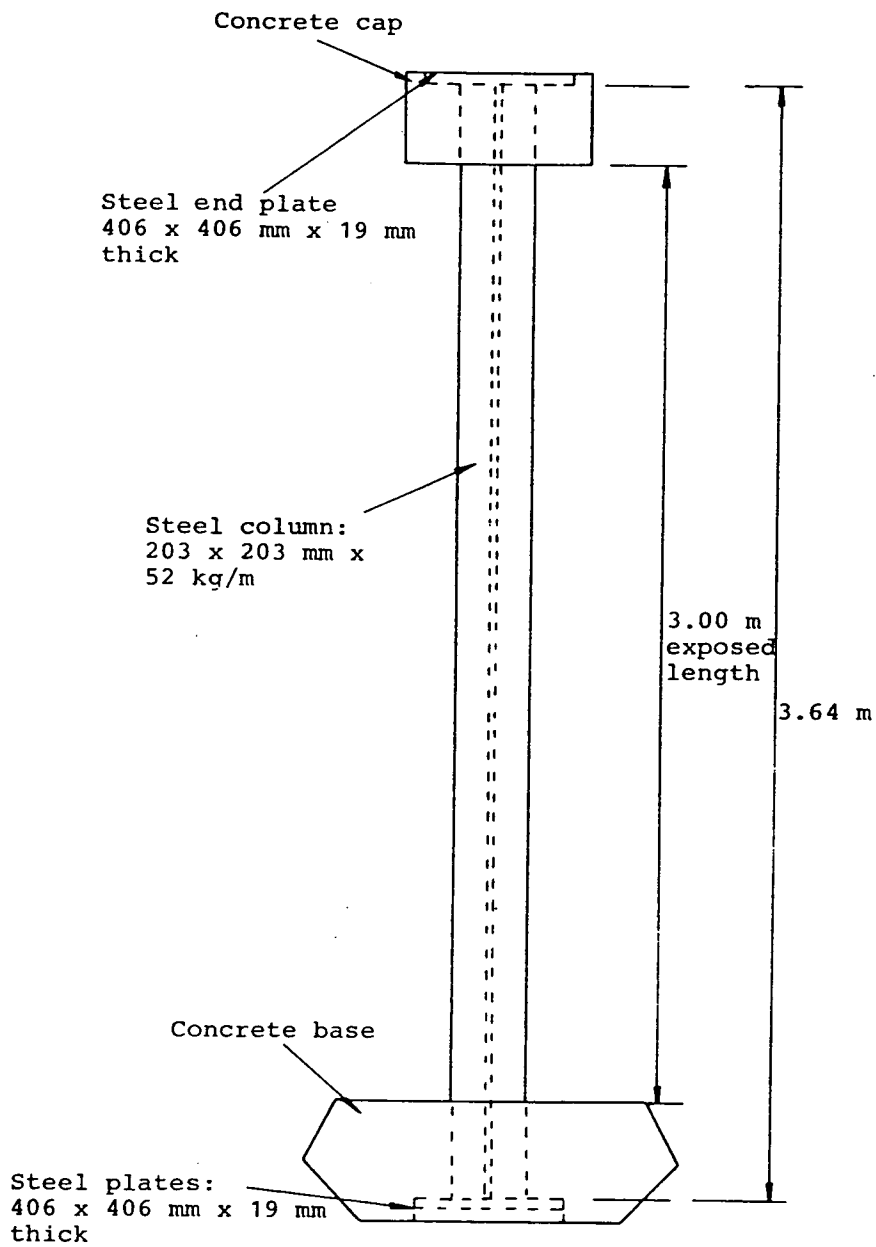


Fig. 3.1 Vertical section of an unprotected column assembly (U.K.)
[Wainman, 1988].

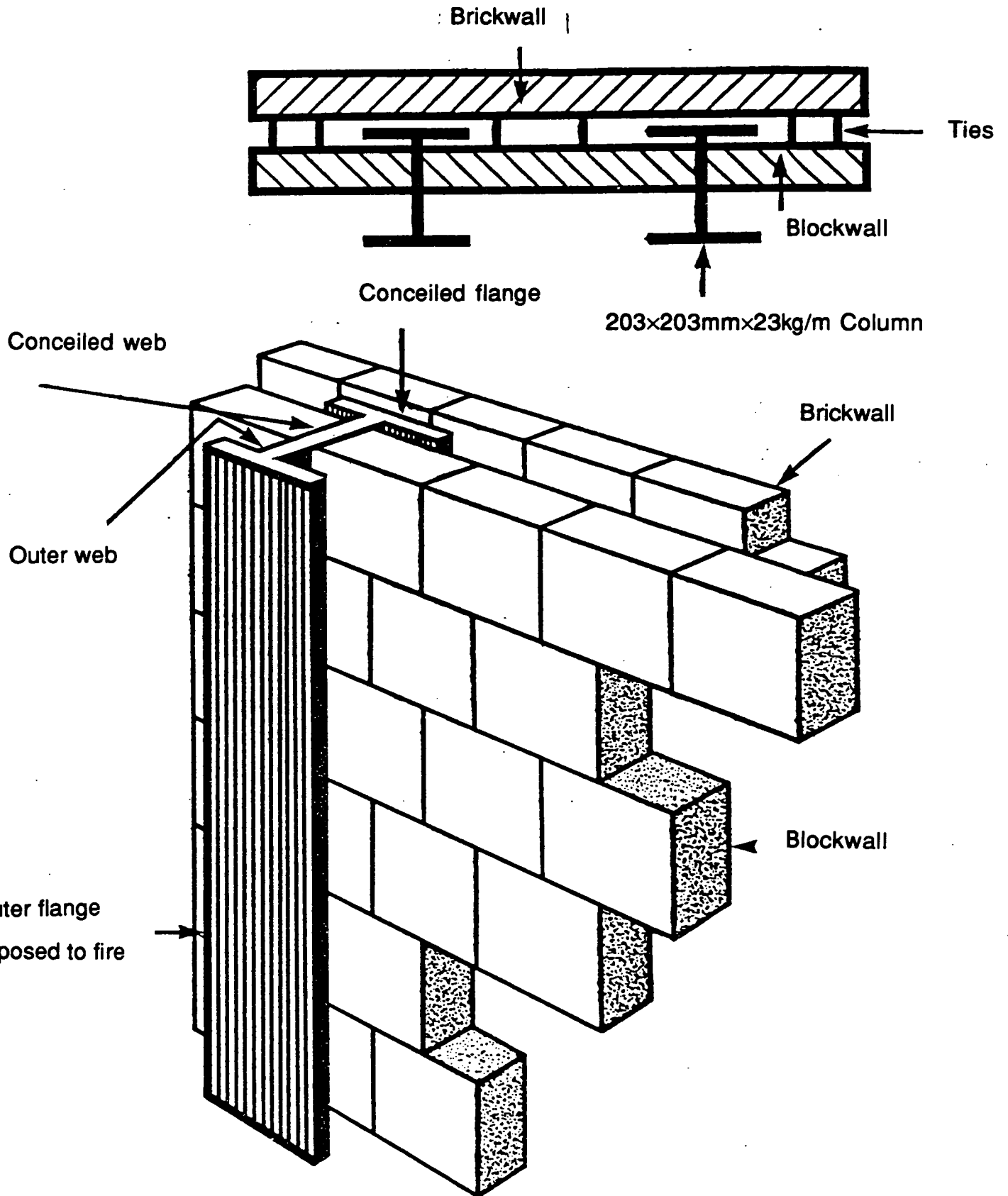
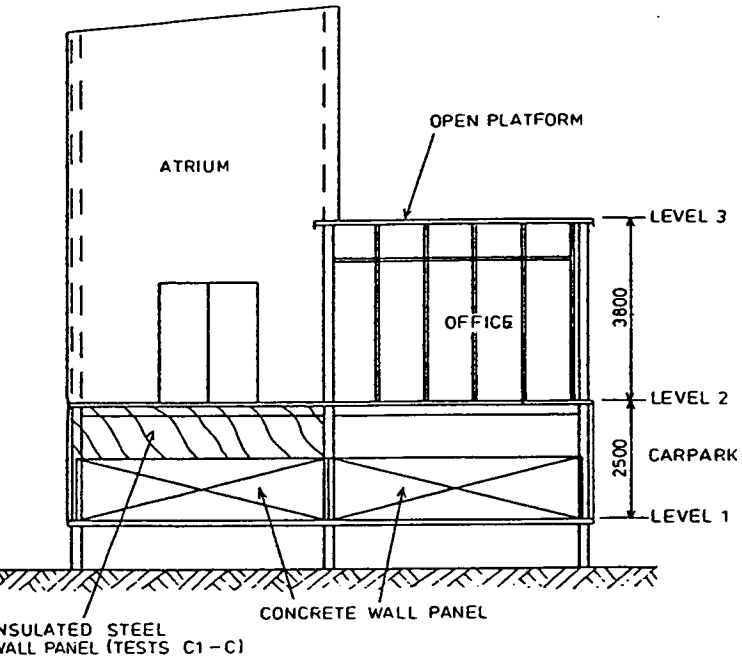
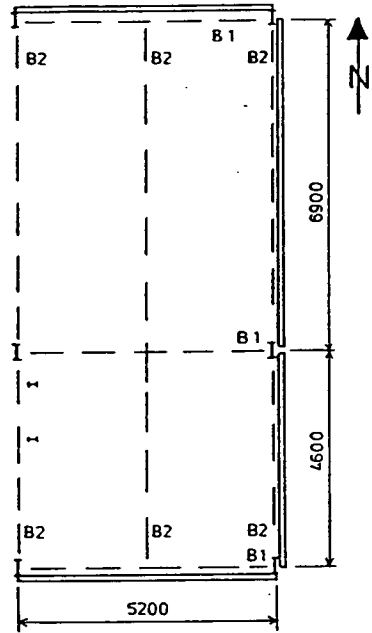


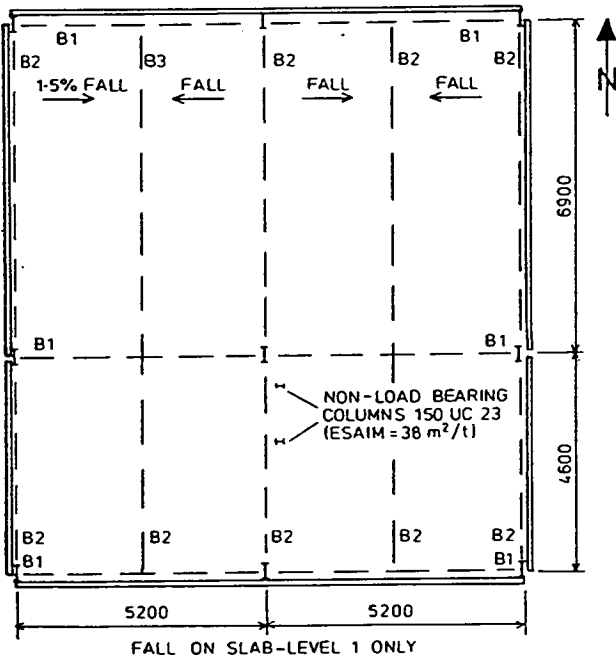
Fig 3.2 Configuration of a steel column built into a wall [Wainman, 1988]



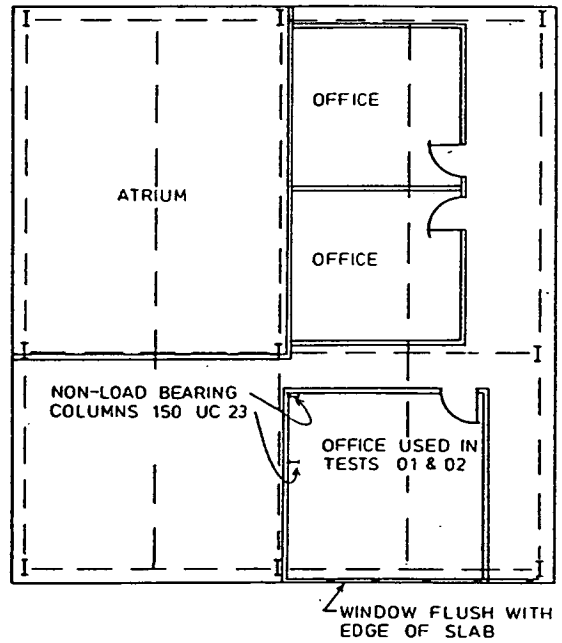
SOUTH ELEVATION



PLAN - LEVEL 3

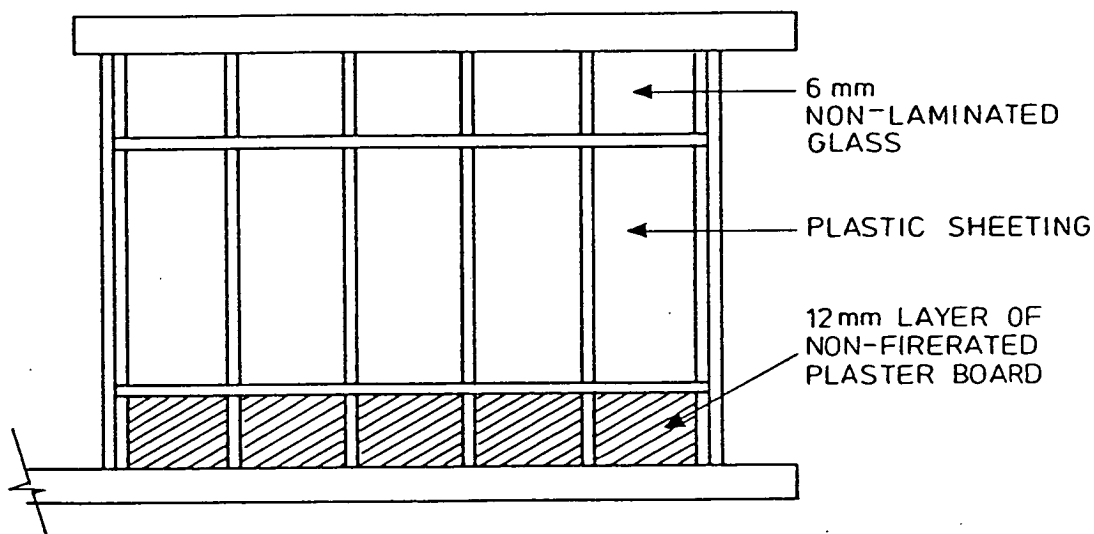


PLAN - LEVELS 1 & 2



LAYOUT OF LEVEL 3

Fig. 3.3 Building structural details - Plans and elevation
 [Almand, 1989].



(b) Facade for Test O2

Fig. 3.3 Building structural details - Office facade details
[Almand, 1989].

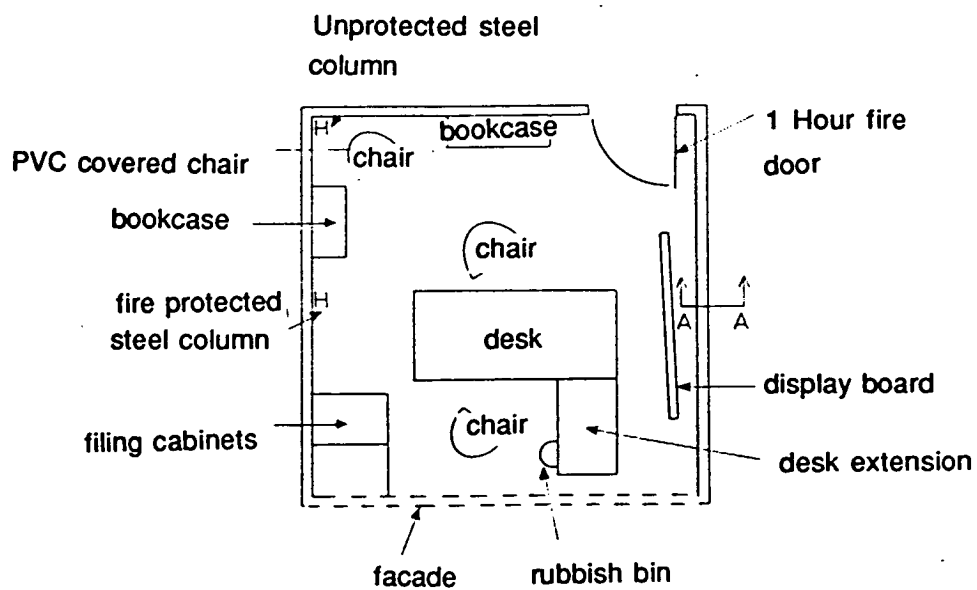


Fig. 3.4 Layout of office before test [Almand, 1989].

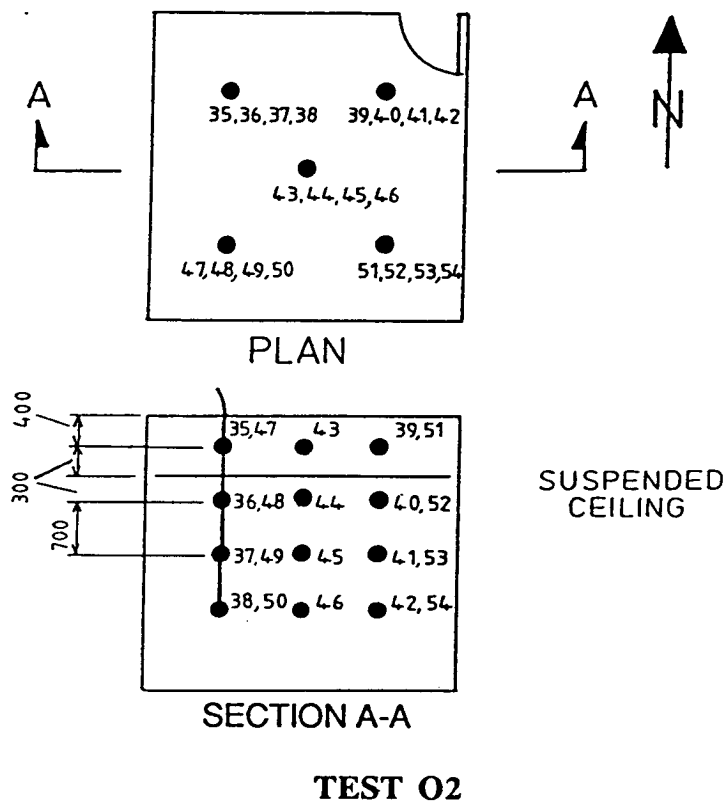
analytical results. The above mentioned full scale test was conducted in a building consisting of a carpark above which there were an atrium and several offices of which only one was used in the test. The actual office area was 4m square in plan which is typical of a personal office space. Identical protected and unprotected beam and column members were installed in the office. A more detailed description of the building structure is given in the Figure 3.3 [Almand, 1989]. The office was furnished with typical contents i.e. filing cabinets, chairs, bookcases, desks and paper (Table 3.1, Fig. 3.4). The fire load is estimated to be 45 kg/m² wood equivalent which falls in the high range of fire loads as surveyed in American office buildings [Almand, 1989]. The air temperature and steel temperature measurements in the office were taken by thermocouples, the location of which are shown in Figures 3.5 and 3.6 respectively. Readings were taken at 50 seconds intervals. Even though a sprinkler was installed in the office, it was not used at all. Observations regarding the fire growth in the office are listed in the Table 3.2. Ventilation was clearly the factor controlling the development of the fire in the test. The maximum air temperatures recorded in the office throughout the test are given in Graph 14. The maximum temperature was not always recorded in the same location during the test. Graph 14 also presents a graphical summary of the maximum cross sectional average temperatures for the bare steel 150UC23 column in the office under consideration. No smoke measurements were made during the test. However, it was observed that significant quantities of smoke and flame exited from the window opening.

3.2.3 Existing Computer Programs

Most of the existing computer programs are based on two basic numerical approaches, namely the finite difference and the finite element methods. The finite difference approach directly models the differential equations

Table 3.1 Fire load in office [Almand, 1989].

Plastics	90 kg	@ 40 MJ/kg	=	3600 MJ
Paper	320 kg	@ 17 MJ/kg	=	5440 MJ
Timber	190 kg	@ 17 MJ/kg	=	3430 MJ
TOTAL	600 kg			12470 MJ
Wood equivalent @ 17 MJ/kg = 721 kg				
Floor area = 16 m ² (4 m x 4 m)				
Fire load = 45 kg/m ² wood equivalent				

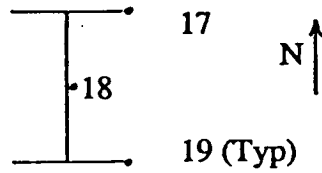


Other thermocouples: 68 and 69 on window, 300 and 700 mm below ceiling

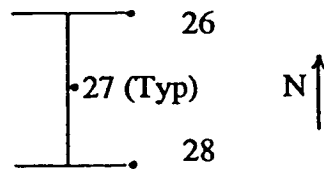
Fig. 3.5 Air temperature thermocouple layout in office [Almand, 1989].

OTHER THERMOCOUPLES:

17, 18, and 19 on north 150UC23 column in office, 1425 mm below ceiling
 20, 21, and 22 on north 150UC23 column in office, 300 mm above floor
 23, 24, and 25 on north 150UC23 column in carpark, 300 mm below office floor



26, 27 and 28 on south 150UC23 column in office, 1425 mm below ceiling
 29, 30 and 31 on south 150UC23 column in office, 300 mm above floor
 32, 33 and 34 on south 150UC23 column in carpark, 300 mm below office floor



70 and 71 on bottom flange of beam in office ceiling space

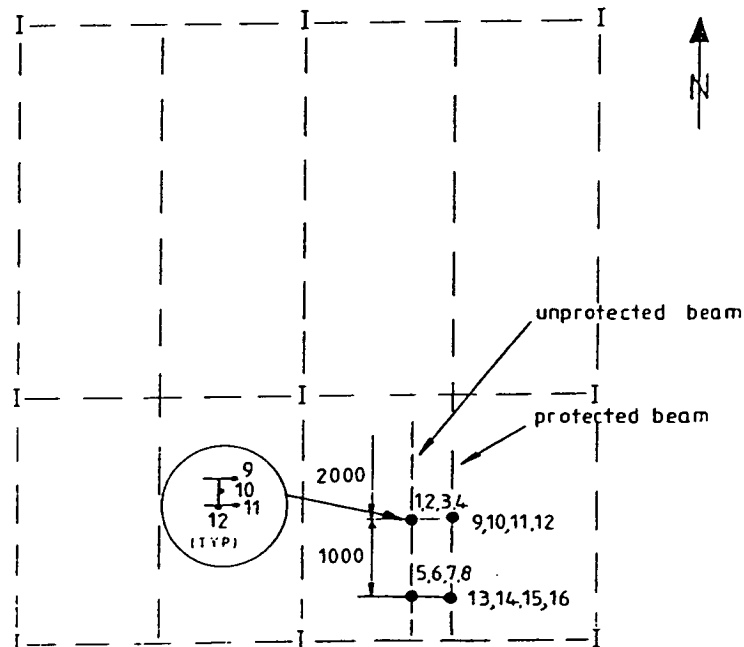


Fig. 3.6 Steel temperature thermocouple layout [Almand, 1989].

Table 3.2 Observations of the full scale test in office [Almand, 1989].

TIME (min.sec)	OBSERVATION
00.00	Ignition
00.40	Smoke rising, flames under desk
01.10	Flames above desk, no smoke visible outside office Plastic film "window" moving in and out due wind
01.30	Door closed, window movement continues due to wind effects
07.40	Small hole (approx 150 mm diameter) in plastic film, third panel from west, approx 200 mm above sill Smoke visible outside office, emitted from top of west side window
16.10	Door opened to encourage more rapid development. Some flaming on floor
20.40	Fire visible above desk again, stacked plastic trays on desk burning
22.40	Flames building up on top of desk (plastic trays, etc)
24.10	Plastic film on windows distorting and disappearing, much greater smoke emission
24.40	Plastic curtains affected
25.10	East curtain disappears
25.20	West curtain disappears
25.30	Flames reach ceiling and spill out front of office
26.00	Flames spread to room divider
26.10	Glass windows above ceiling break
26.20	Flashover
27.10	Flames above ceiling
27.40	Smoke and flames emitted from top of door (between door and frame)
27.50	Ceiling panel falls (several have by this stage)
28.20	Full room involvement ceases

29.10	Ceiling panel falls (most have by this stage). Side bookcase and corner chair continue burning
32.40	Rear bookcase also burning
44.40	Fire dies down, 2 small flaming areas
50.00	Door noted to be warped and door lock inoperative (not openable)
56.40	Door forced open (ash, etc behind door makes opening difficult)

describing the heat transfer for a particular problem. The finite element approach discretizes the continuum.

Using finite element computer programs, one has to consider the trade-off between computer size and mesh shape and size and the time increment size. The smaller the mesh size and the time increment size, the larger the required computer capacity. The choice of element size, shape and orientation is a process of trial and error. The accuracy of the results is often a function of the experience of the engineer using the model. The choice of the time increment usually depends on numerical stability requirements. Stability is a function of element boundary conditions, temperature distribution and material properties.

There are two widely available finite element computer programs for calculating heat transfer from fires to structures. These are described in the following paragraphs.

TASEF - 2

TASEF (Temperature Analysis of Structures Exposed to Fire) is a thermo-analysis computer program developed by Ulf Wickstrom in Sweden [Sterner,1990]. It may be used to calculate temperatures in structures exposed to fire. It is based on the finite element method. Although the original version is limited to a two-dimensional analysis, the author states that it could be modified to handle three dimensions. Structures comprised of one or more materials can be analysed. At the boundaries, heat transfer by convection and radiation can be modelled. Two dimensional rectangular elements are used. Input of the geometry and generation of the finite element mesh have been automated. Non-linearities due to temperature dependence of material properties and boundary conditions can be considered. Heat transfer by convection and radiation can be calculated.

TASEF-2 uses an explicit , forward difference time integration approach which leads to shorter execution times and allows better modelling of latent heat effects. It automatically calculates a critical time increment for each iteration.

Details about the finite element approximation are given in the TASEF - User's Manual [Sterner, 1990].

FIRES-T

FIRES-T3 is a three-dimensional finite element computer program, developed at the University of California, Berkeley. It is a very general computer program which uses a backward difference time integration approach. Because of that approach, latent heat effects in materials like concrete or gypsum cannot be modelled directly. Instead, they are modelled by assuming an appropriate internal heat generation. The backward difference integration approach has the advantage of numeric stability.

If a small time increment is not initially specified, the program may reach the point where it will not converge. The program must be restarted with a smaller time increment.

Some typical input requirements of the program are listed below:

- Object geometry,
- Material properties as a function of temperature,
- Boundary conditions ,
- Fixed temperature or a heat flux based on gas time-temperature data.

With both the above considered models, unnecessary small time increments can sometimes be avoided by assuming lumped temperature distributions. A lumped temperature distribution assumes the same temperature for adjoining nodes. In a similar manner, it is possible to assume a specified temperature boundary condition instead of a highly non-linear heat flux

boundary. These approximations require a thorough understanding of the physical basis of the heat transfer being modeled. Incorrect application of these approximations may lead to serious errors.

Other Computer Programs

ABAQUS

ABAQUS [Terro,1987] is a computer program which is based on the finite element method. It is developed to model solid body heat conduction with general, temperature dependent conductivity, internal energy (including latent heat effects) and quite general convection and radiation boundary conditions.

LUSAS

LUSAS [Terro,1987] is a finite element computer program. It is developed for transient field analysis, governed by the quasi - harmonic field equation. The finite difference discretisation in time employs the Crank - Nicholson rule:

$$\theta_{t+\Delta t} = 2 \theta_{t + \Delta t/2} - \theta_t \quad (3.13)$$

where: θ_t = nodal temperature at time t

PAFEC

PAFEC [Terro,1987] is a computer program based on the finite element method. It is developed for temperature analysis of complex engineering structures. There are two choices of thermal calculation in this particular program. It is the steady-state analysis and the transient-state analysis.

3.2.4 Existing Codes

ECCS [ECCS, 1983] recommends that the standard fire curve [BSI,1987] be used in calculations of the steel temperature. It is illustrated in Fig. 3.7 and is given by the equation:

$$\theta_s - \theta_o = 345 \log_{10} (8t + 1). \quad (3.14)$$

where: t = time;

θ_s = furnace temperature at time t $^{\circ}\text{C}$,

θ_o = furnace temperature at time $t=0$ $^{\circ}\text{C}$.

The heat flow transmitted from the fire compartment to unit length of the steel member is calculated using the following equation:

$$Q = K F (\theta_t - \theta_s) \quad [\text{W} / \text{m}] \quad (3.15)$$

where:

Q = heat flow $[\text{W} / \text{m}]$;

K = coefficient of total heat transfer $[\text{W} / \text{m}^2 \text{ } ^{\circ}\text{C}]$,

F = surface area of the member per unit length exposed to heating $[\text{m}^2/\text{m}]$,

θ_t = ambient gas temperature at time t $^{\circ}\text{C}$,

θ_s = temperature of the steel member $^{\circ}\text{C}$.

The coefficient of total heat transfer has three components and is given by the following equation:

$$K = \frac{1}{\frac{1}{\alpha_c + \alpha_r} + \frac{d_i}{\lambda_i}} \quad [\text{W} / \text{m}^2 \text{ } ^{\circ}\text{C}] \quad (3.16)$$

where:

α_c = coefficient of heat transfer due to convection from the fire to the exposed surface of the member $[\text{W}/\text{m}^2 \text{ } ^{\circ}\text{C}]$,

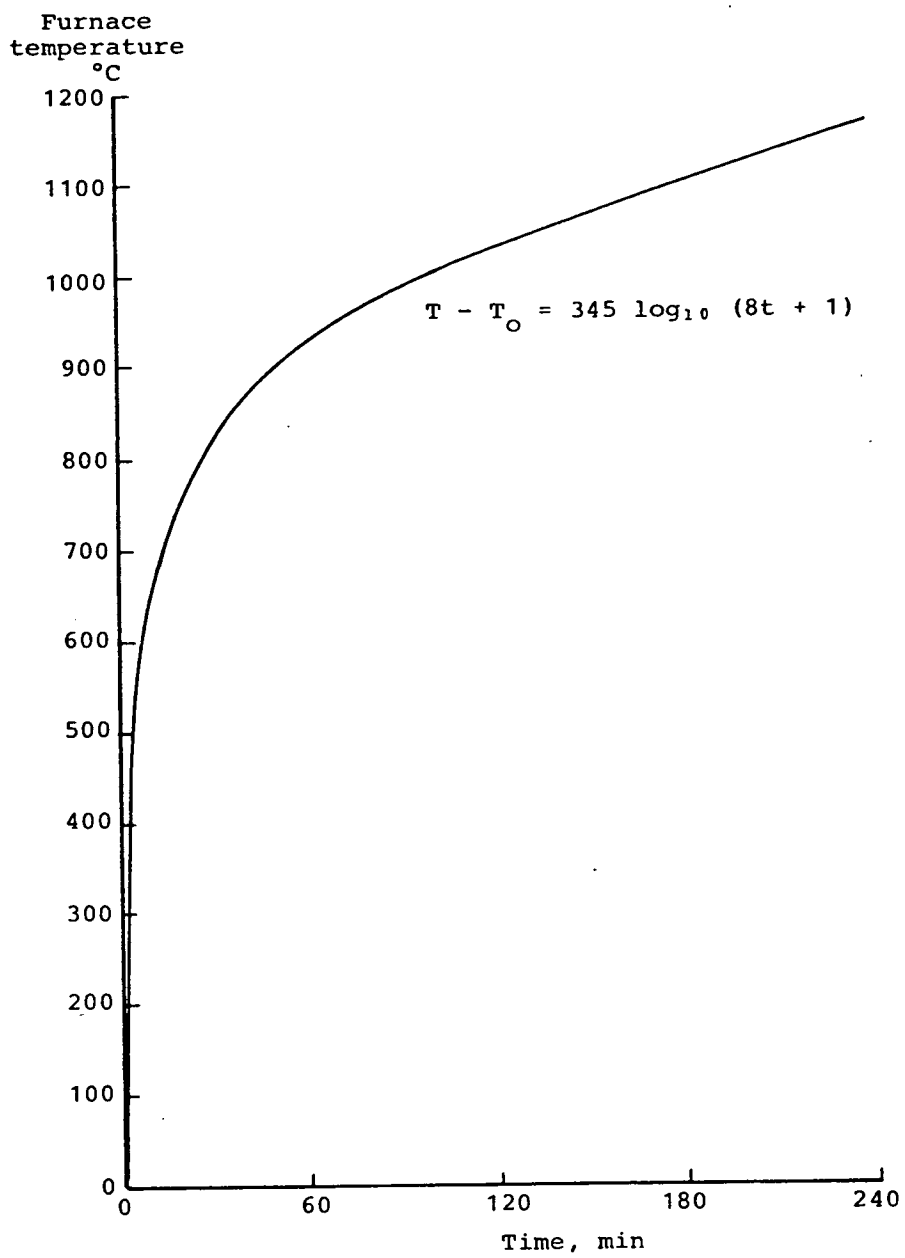


Fig. 3.7 Furnace heating curve [BSI, 1972].

- α_r =coefficient of heat transfer due to radiation from the fire to the exposed surface of the member [W /m² °C],
- λ_i =thermal conductivity of the insulation material [W /m °C],
- d_i =thickness of the insulation [m].

For bare steel elements, the equation of heat flow is given as follows:

$$Q = (\alpha_c + \alpha_r) F (\theta_t - \theta_s) \quad (3.17)$$

where:

$$\alpha_c = 25$$

$$\alpha_r = \frac{5.77 \epsilon_r}{\theta_t - \theta_s} \left[\left(\frac{\theta_t + 273}{100} \right)^4 - \left(\frac{\theta_s + 273}{100} \right)^4 \right] \quad [\text{W/m}^2 \text{ } ^\circ\text{C}]$$

where:

ϵ_r = resultant emissivity of the flames, combustion gases and exposed surface.

The value of α_c , given according to the equation, is based on experimental investigations of standard fire exposure as well as natural fire exposure.

The value of α_r is based on the Stefan - Boltzmann law.

The resultant emissivity ϵ_r depends on the type of the fire and the position of the exposed member. ECCS recommends the value of 0.5 for the resultant emissivity ($\epsilon_r=0.5$) which gives a conservative solution.

In Chapter Five (5) of this thesis, a more accurate evaluation of the resultant emissivity is given according to the Swedish Fire Engineering Design of Steel Structures [Pettersson, 1976].

The calculation of the temperature increase Δq_s of a non- insulated member exposed to fire is based on the assumption of quasi- stationary, one dimensional heat transfer. The steel is considered as a heat sink, in which

the heat supplied is instantly distributed to give a uniform temperature. The equation which describes the temperature increase of a member during a time interval Δt is given as follows:

$$\Delta\theta_s = \frac{\alpha}{c_s \rho_s} \frac{F}{V} (\theta_t - \theta_s) \Delta t \quad [^{\circ}\text{C}] \quad (3.18)$$

where:

$\alpha = \alpha_c + \alpha_r$ = coefficient of heat transfer $[\text{W}/\text{m}^2 \text{ } ^{\circ}\text{C}]$,

c_s = specific heat of steel $[\text{J}/\text{kg } ^{\circ}\text{C}]$,

ρ_s = density of steel $[\text{kg}/\text{m}^3]$,

F = surface area of the member per unit length exposed to fire $[\text{m}^2/\text{m}]$,

V = volume of the member per unit length $[\text{m}^3/\text{m}]$

θ_t = ambient gas temperature during the time interval Δt $[^{\circ}\text{C}]$,

θ_s = steel temperature during the time interval Δt $[^{\circ}\text{C}]$,

Δt = time interval $[\text{sec}]$.

The temperature increase in the member depends on the geometry, represented by "the section factor (F / V)".

The value of the time interval (Δt), for convergence, has an upper limit given below:

$$\Delta t < \frac{2.5 \times 10^4}{F/V} \quad [\text{sec}]. \quad (3.19)$$

3.3 Examples

Four cases (Fig. 3.8) were analysed in order to assess the accuracy and efficiency of TASEF-2. The solutions of the first case (column exposed from

four sides) and the third case (column built into a wall) are compared to temperatures measured during laboratory tests (Data sheet 40a,b and 48a,b [Wainman, 1988]). The solution of the second case (column built next to a wall) is compared relatively to the other two solutions because of the lack of fire test results concerned with this particular problem. The solution of the fourth case (corner column) is compared to temperature measured during a full scale fire test in an office building [Almand, 1989].

The coefficient of convection heat transfer and the resultant emissivity of the radiative mode of heat transfer have to be chosen. These factors depend on the relative situation of the burners to the test specimens, the furnace size, the type of fuel, the furnace wall characteristics. They are rather difficult to be defined precisely. Values for the various factors have been chosen for the four cases studied as listed below:

- Heat transfer by radiation

$$\epsilon_{\text{steel}}=0.6$$

$$\epsilon_{\text{concrete}}=0.8$$

where: ϵ = resultant emissivity.

-Convective heat transfer

For the fire exposed surfaces,

$$\beta=25.00 \text{ W/m}^2\text{K}$$

$$\gamma=1.00$$

For the non fire exposed surfaces,

$$\beta=2.25 \text{ W/m}^2\text{K}$$

$$\gamma=1.00$$

where: β = convective heat transfer coefficient ($\text{W/m}^2\text{K}$),

γ = convective heat transfer power



The gas temperature obtained in the furnace should be identical to the standard fire:

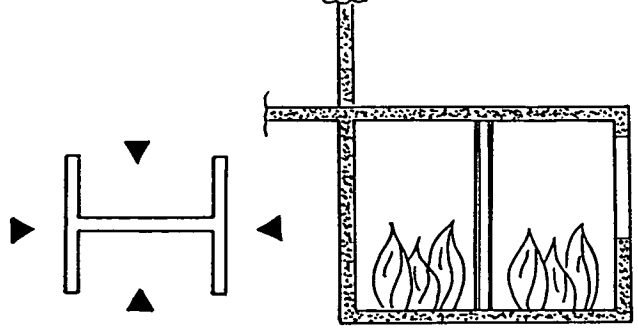
$$T = 20 + 345 \log_{10} (8t + 1) \quad (3.20)$$

where: t =fire time in minutes.

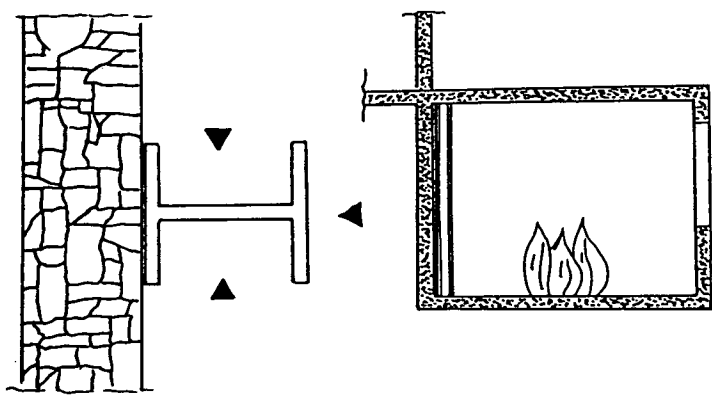
The temperatures realised in the fire tests [Wainmann, 1988] are very close to the ideal temperature curve so that all the tests can be classified as standard fire tests. For the first and third cases, the temperature input for TASEF-2 was taken to be the gas temperature as measured during the actual fire test, used to validate the analytical results. For the second case, the temperature input was taken to be the standard curve. The temperature input for the fourth case was taken as the actual gas temperature measured during the full scale test. Like most of the computer programs, TASEF-2 uses as temperature input the gas temperature instead of the temperature of the furnace walls. This gives a good approximation because of the low thermal conductivity of the walls of the furnace.

For each column studied, the cross section finite element mesh (Figs 3.9 - 3.12) used for analysis with TASEF-2 is illustrated. The results are presented in the form of steel temperature - time plots (Graphs 1-14). The measured and calculated results are in good concordance for the cases A (Graphs 1-3) and C (Graphs 7 -12). Case B (Graphs 4 -6) compared relatively to cases A and C, shows good results. For Case D (Graph 12-14), the measured and calculated results are in good agreement considering the uncertainties involved in the simulation of a real fire environment.

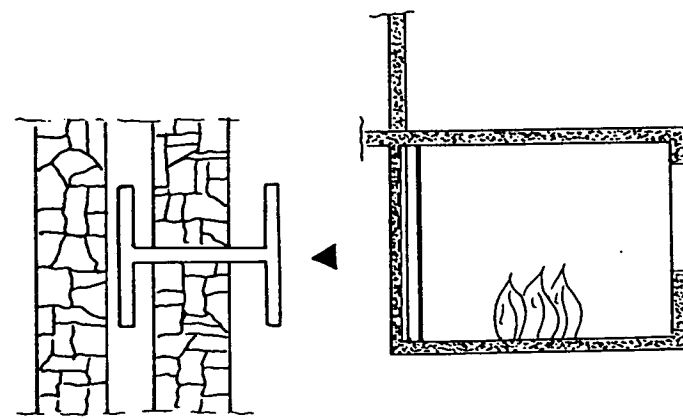
The variation of the material properties of concrete and steel with temperature was taken to be as given in the reference [Sterner, 1990].



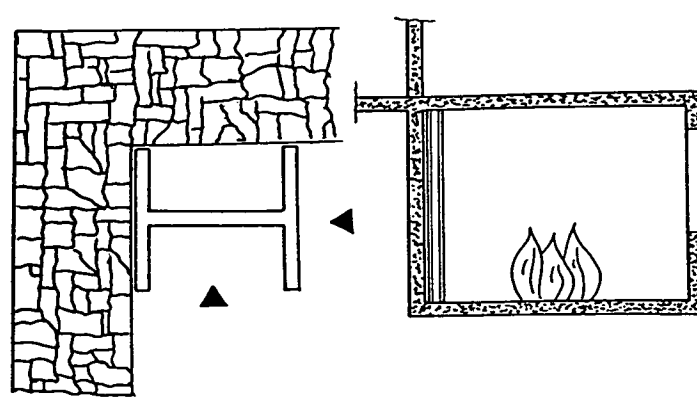
Case A



Case B



Case C



Case D

Fig. 3.8 Cases studied

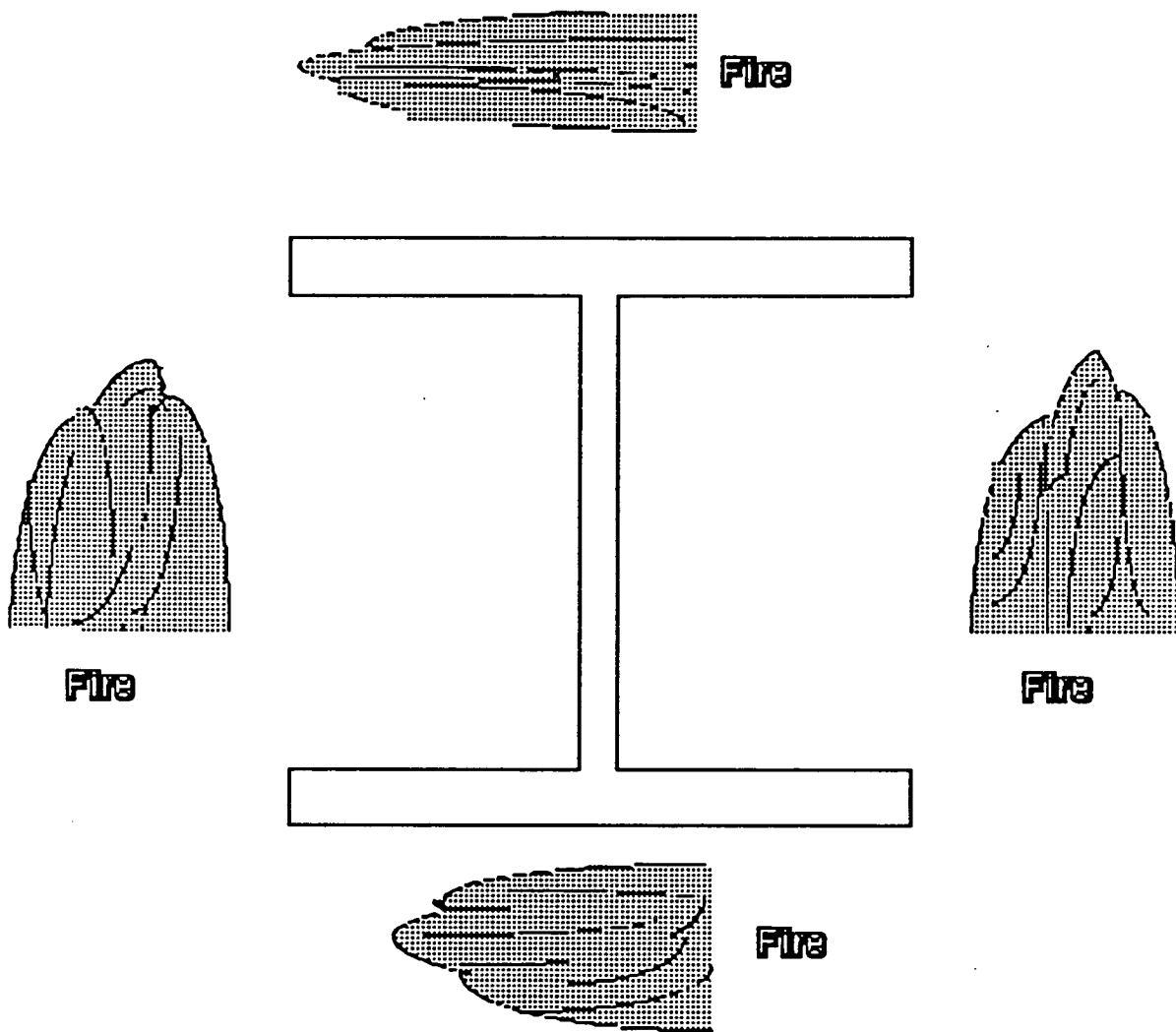


Fig. 3.9a Case A - Cross section profile

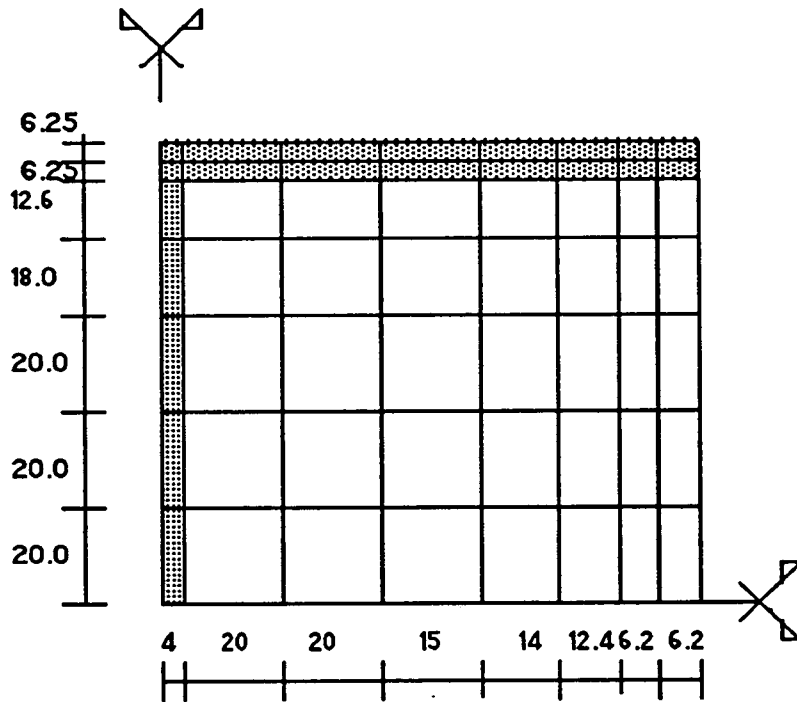
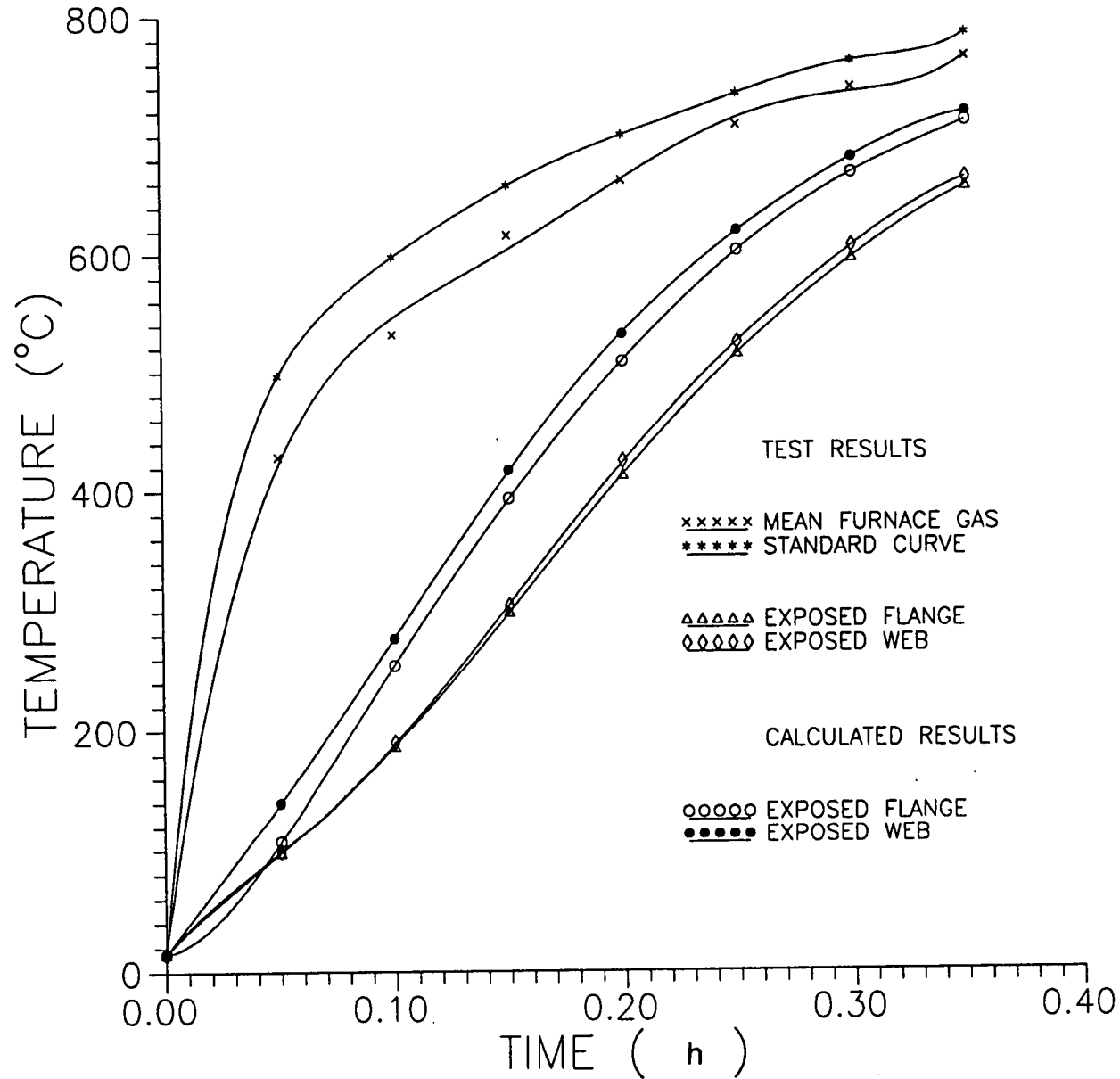
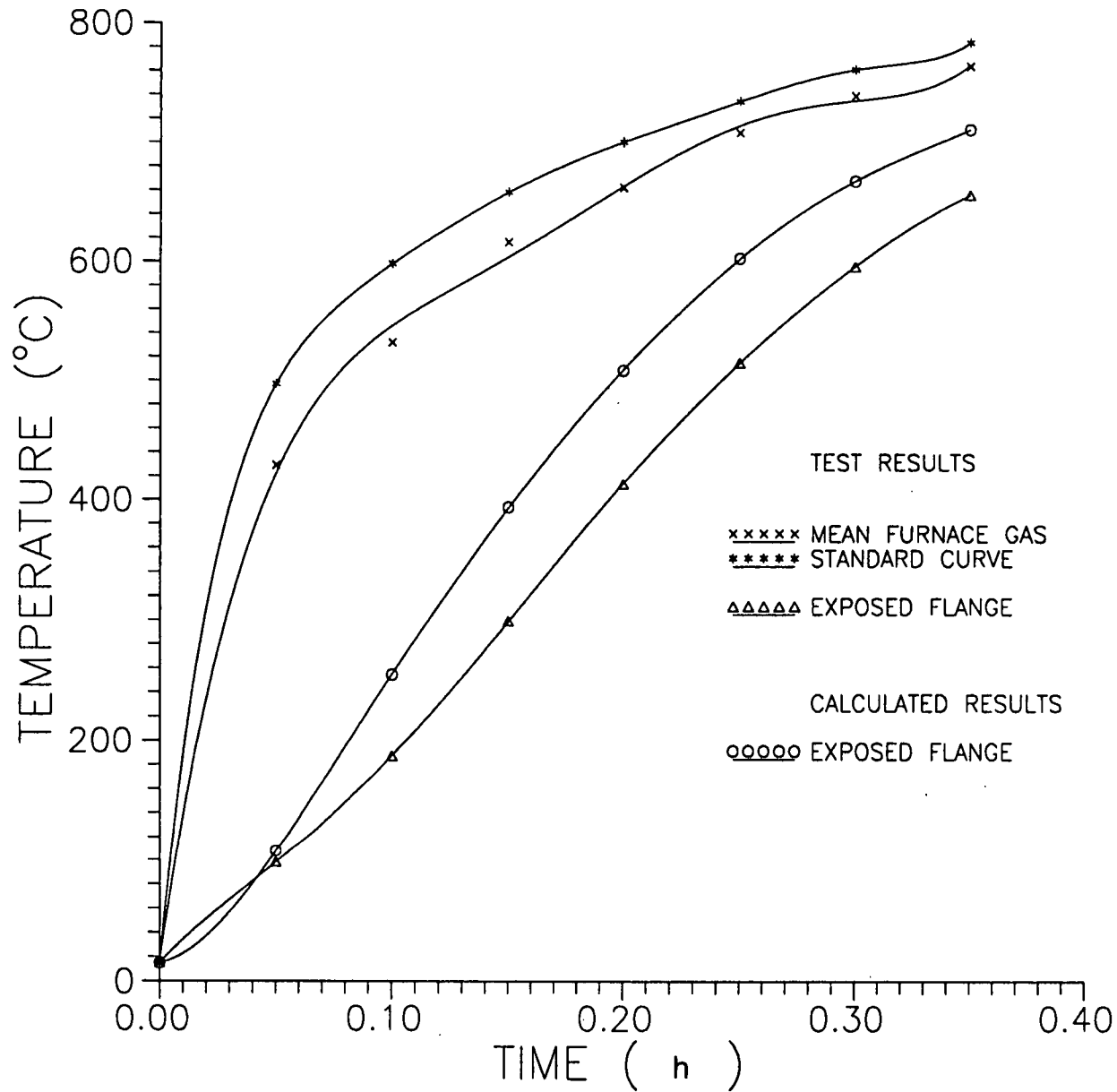


Fig. 3.9b Case A - Cross section finite element mesh

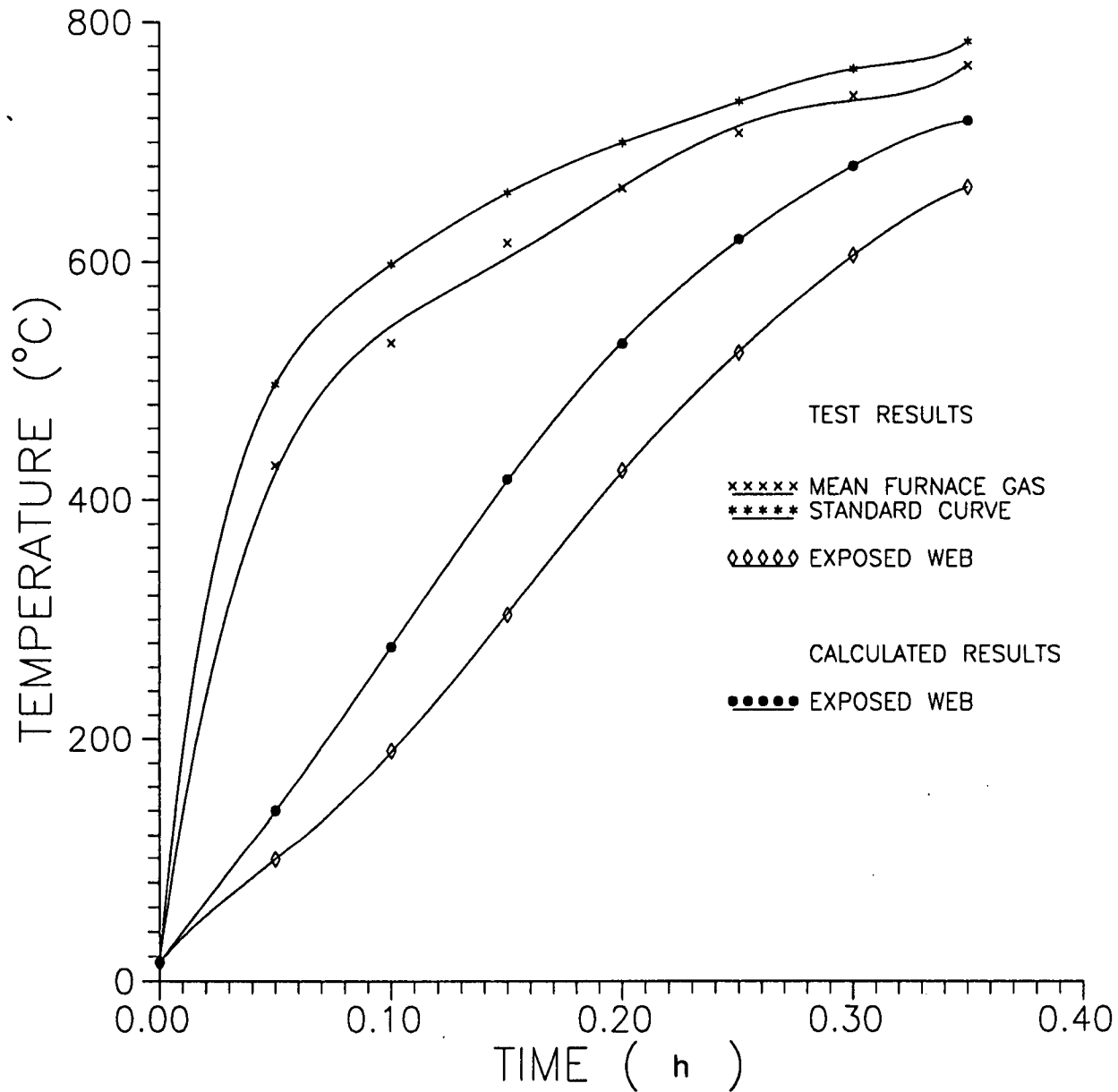
GRAPH 1. TIME-TEMPERATURE PLOT - CASE A



GRAPH 2. TIME-TEMPERATURE PLOT - CASE A



GRAPH 3. TIME-TEMPERATURE PLOT - CASE A



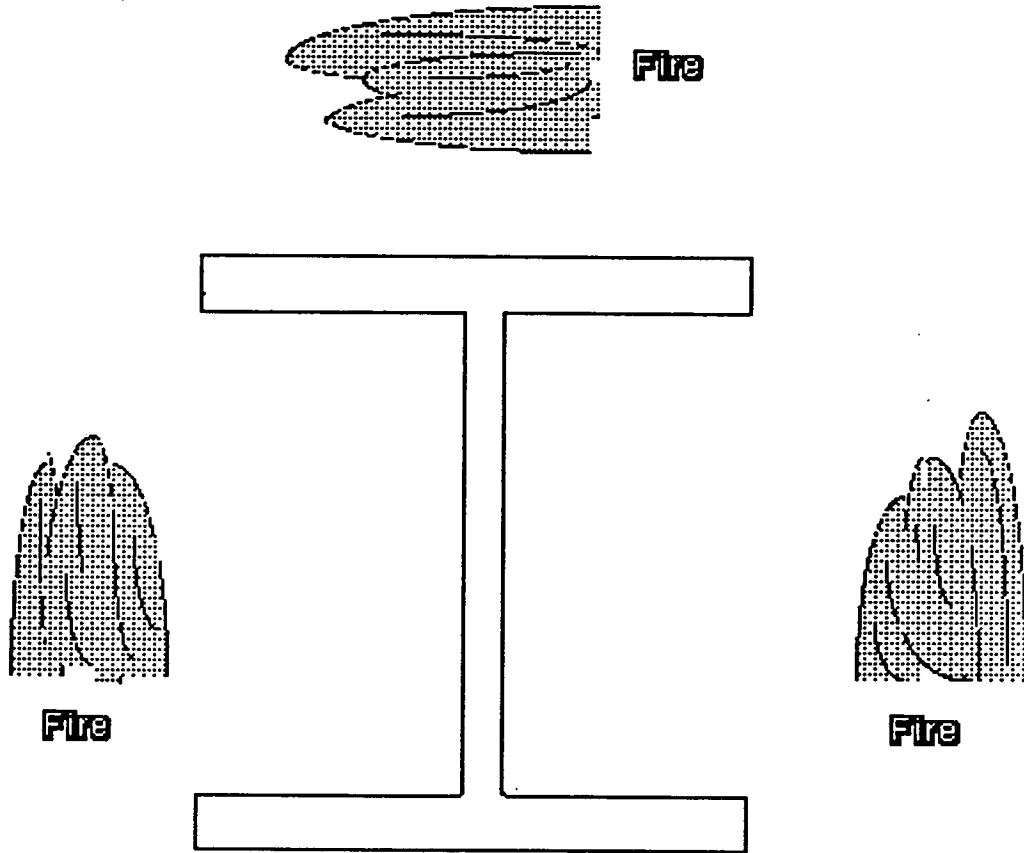


Fig. 3.10a Case B - Cross section profile

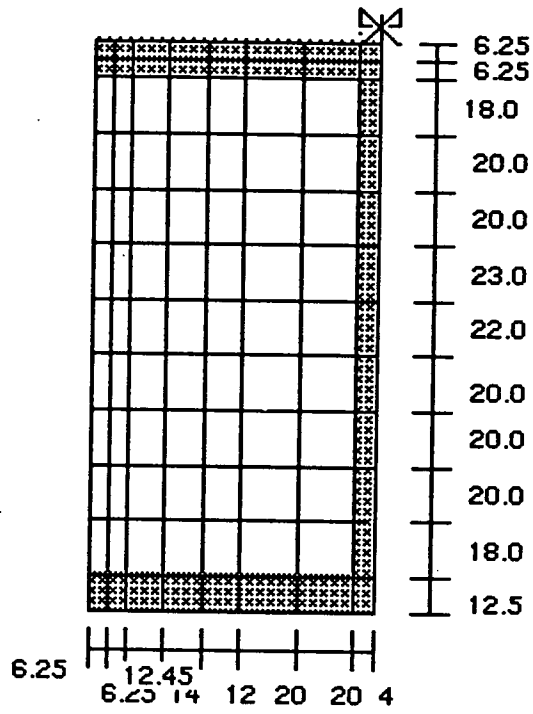
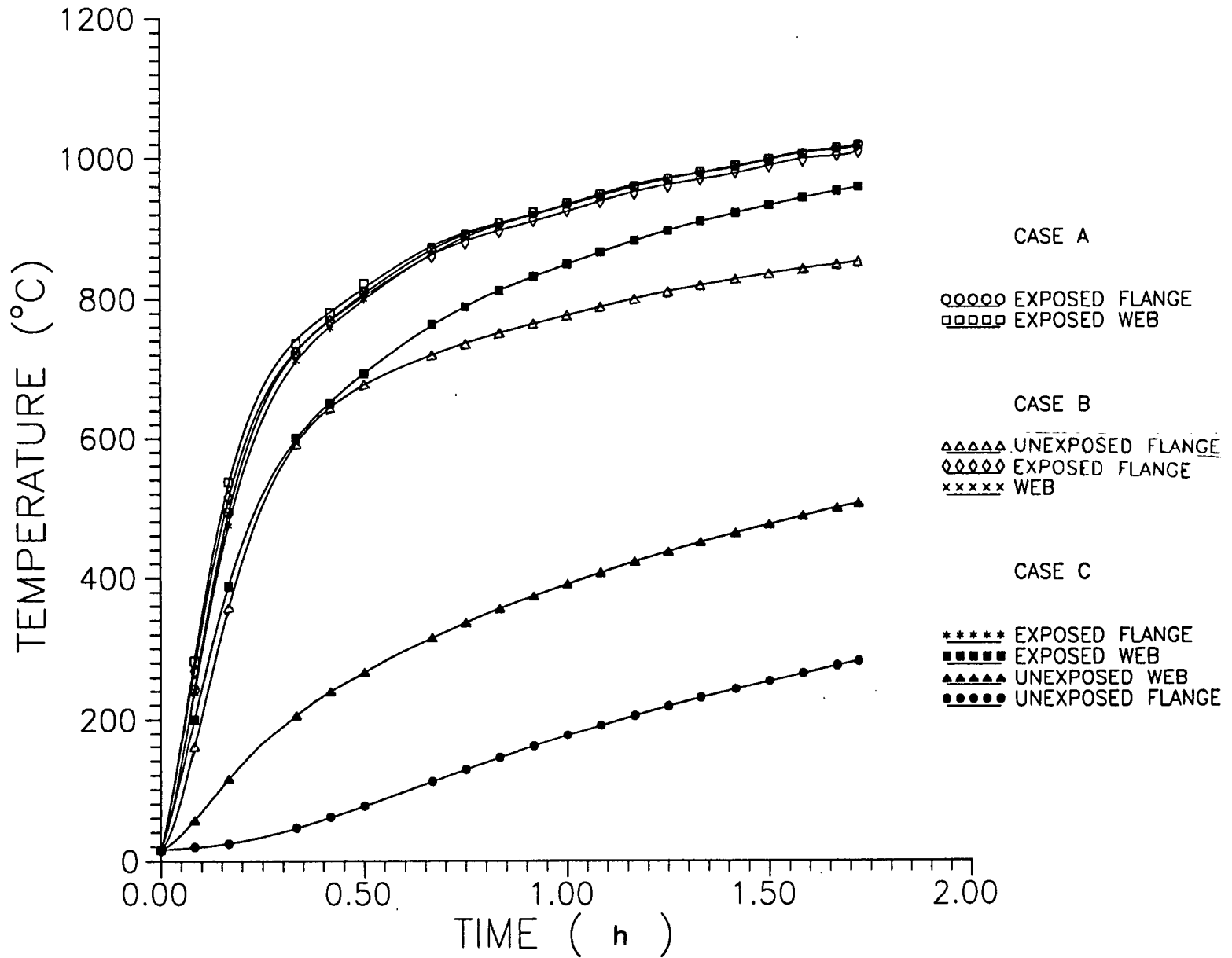
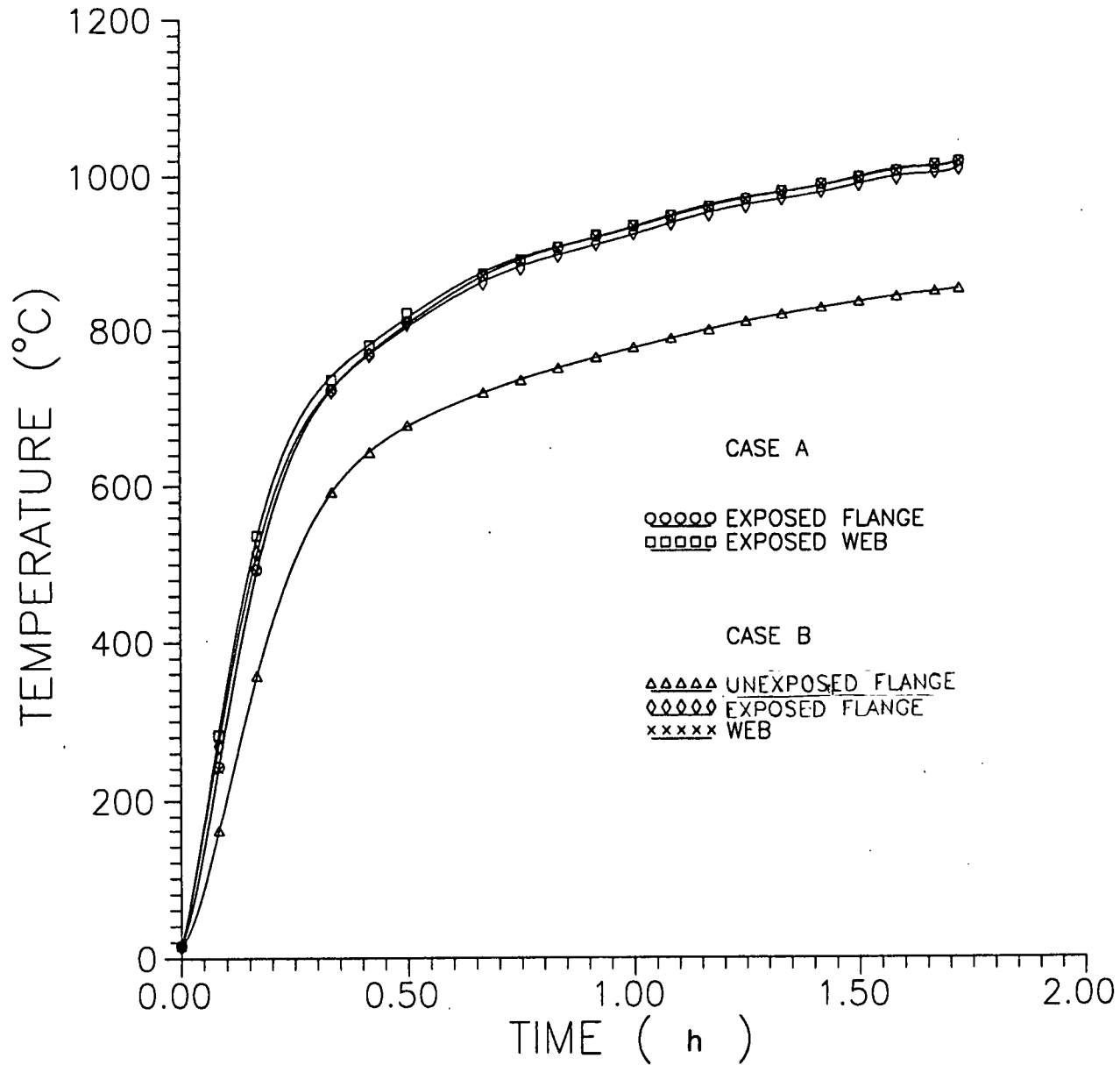


Fig. 3.10b Case B - Cross section finite element mesh

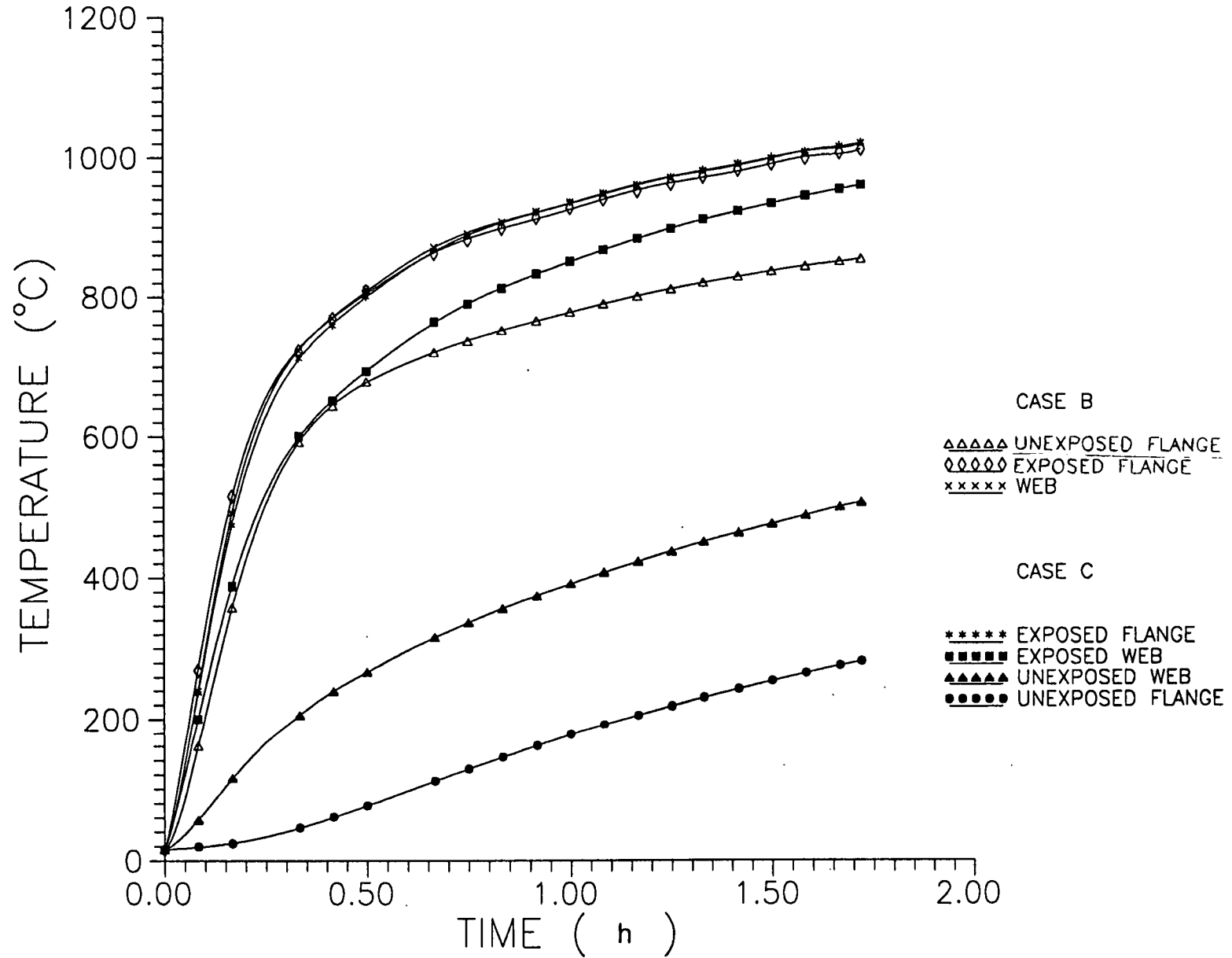
GRAPH 4. TIME-TEMPERATURE PLOT - CASE B



GRAPH 5. TIME-TEMPERATURE PLOT - CASE B



GRAPH 6. TIME-TEMPERATURE PLOT - CASE B



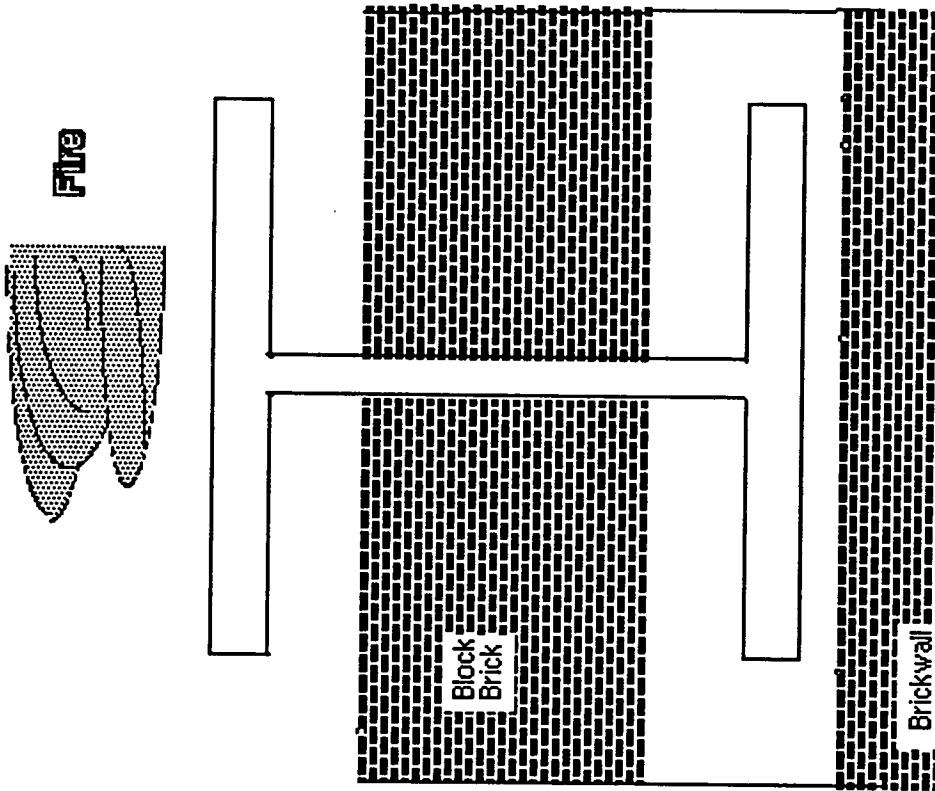


Fig. 3.11a Case C - Cross section profile

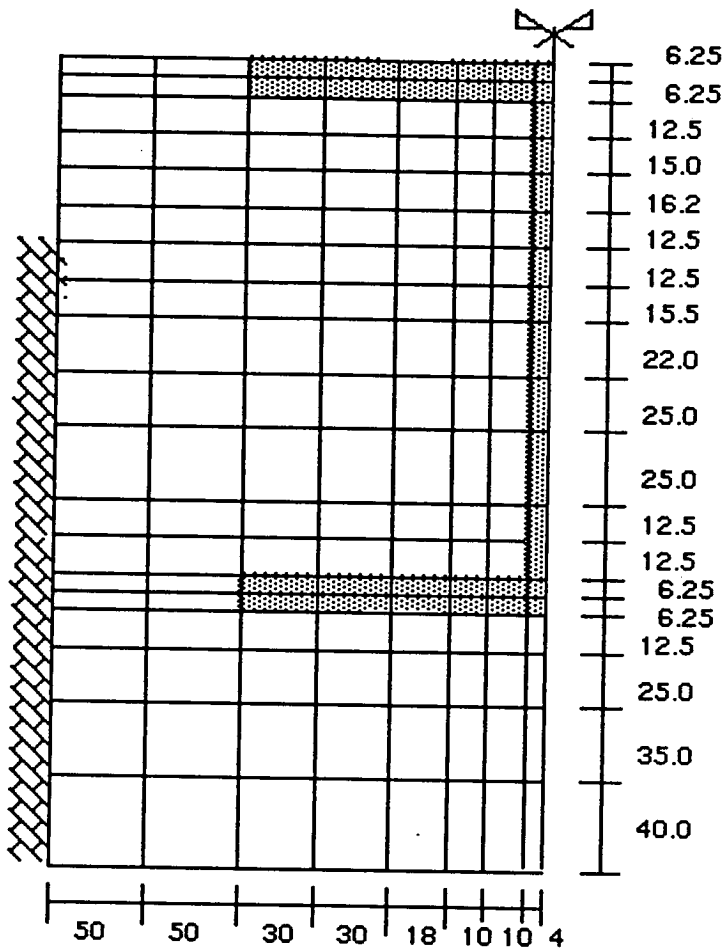
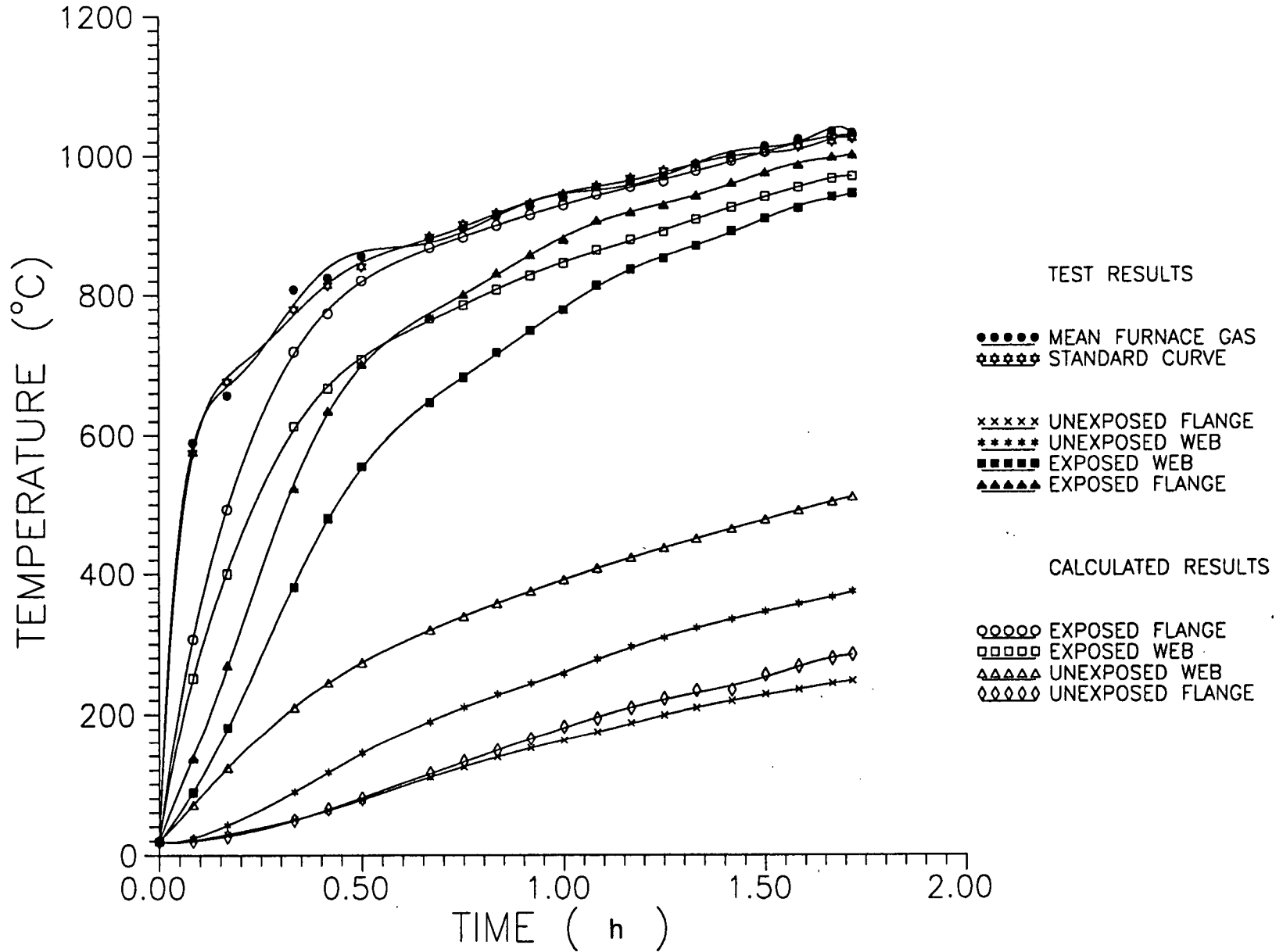
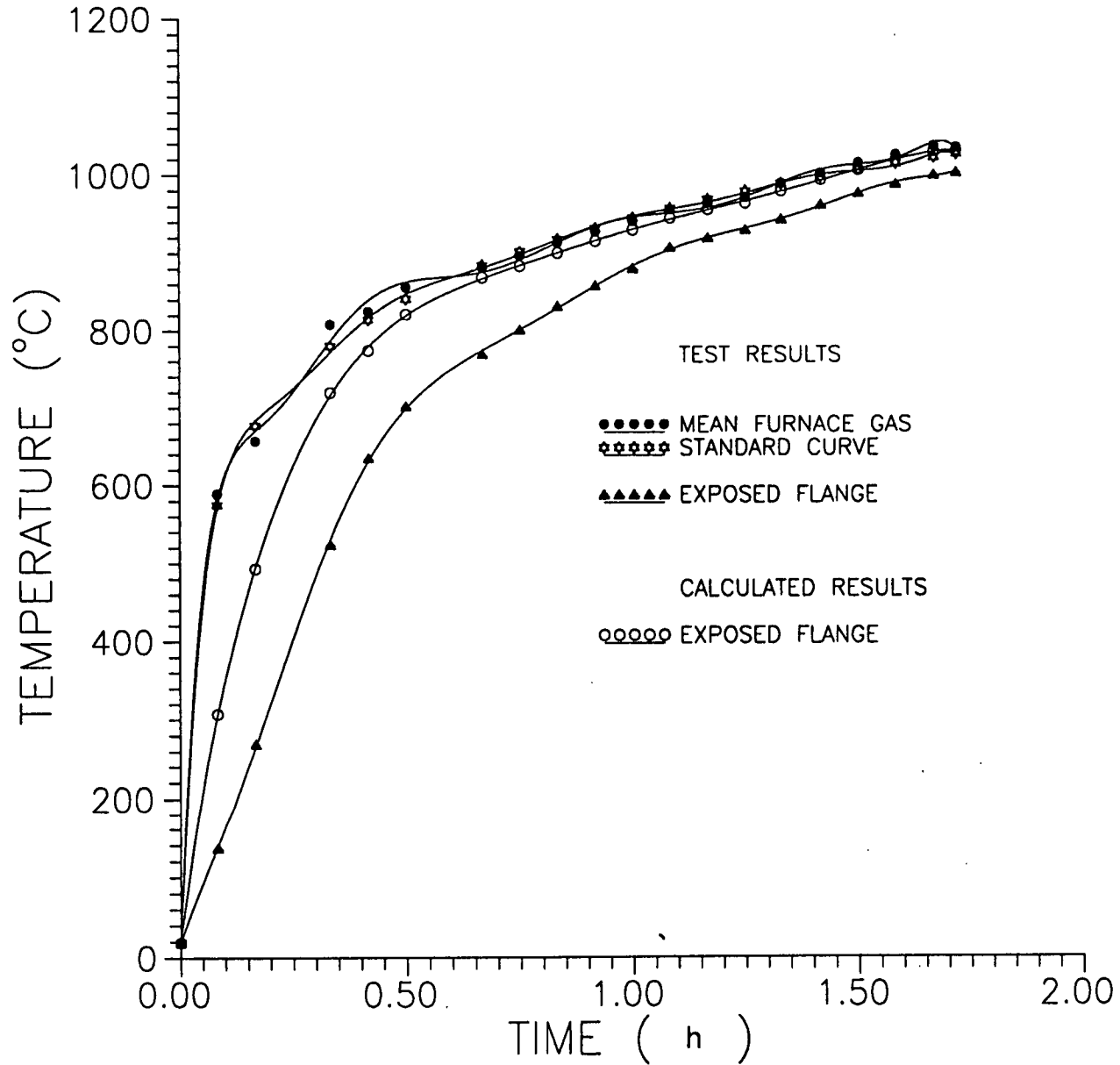


Fig. 3.11b Case C - Cross section finite element mesh

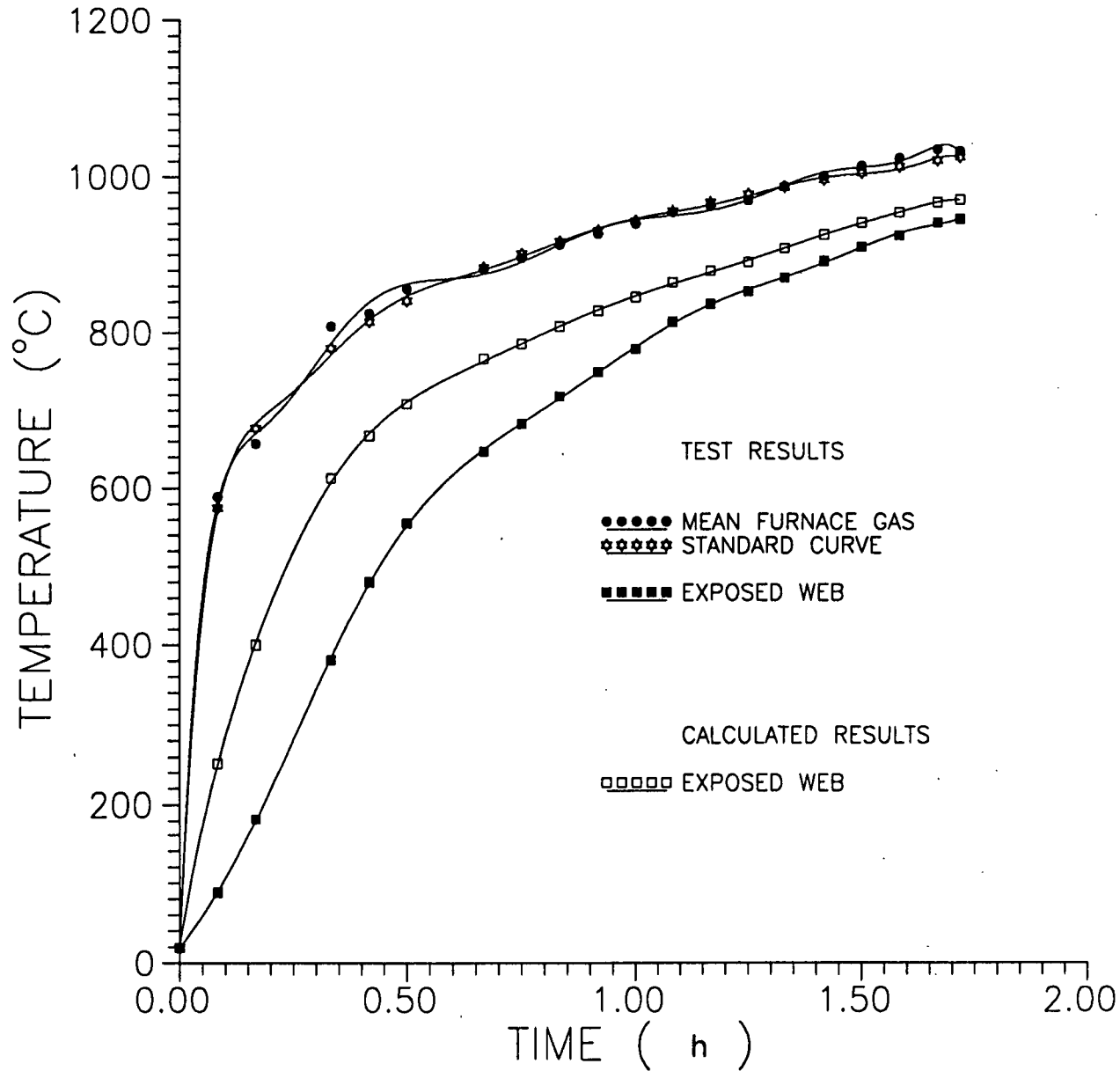
GRAPH 7. TIME-TEMPERATURE PLOT - CASE C



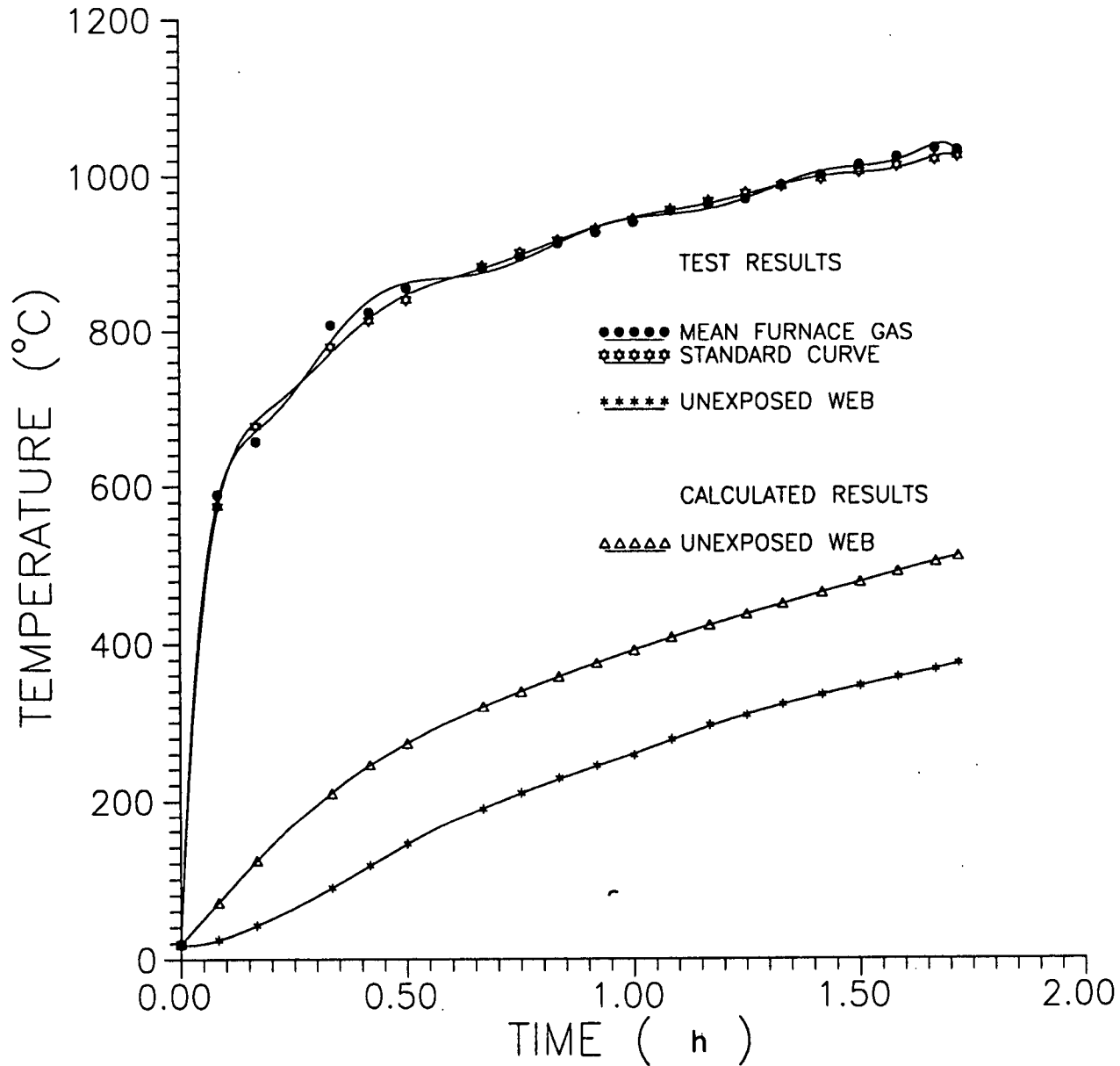
GRAPH 8. TIME-TEMPERATURE PLOT - CASE C



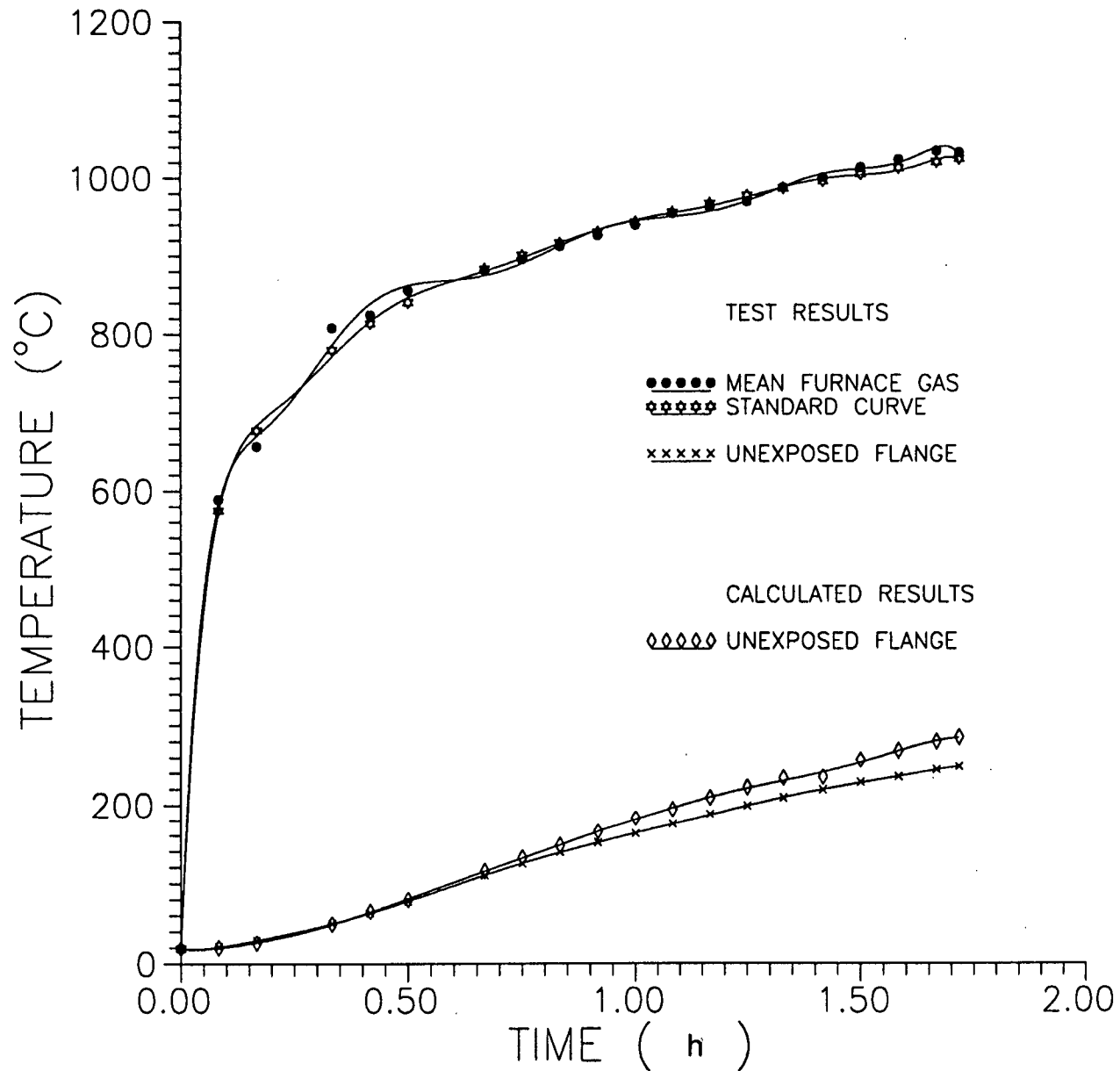
GRAPH 9. TIME-TEMPERATURE PLOT - CASE C



GRAPH 10. TIME-TEMPERATURE PLOT - CASE C



GRAPH 11. TIME-TEMPERATURE PLOT - CASE C



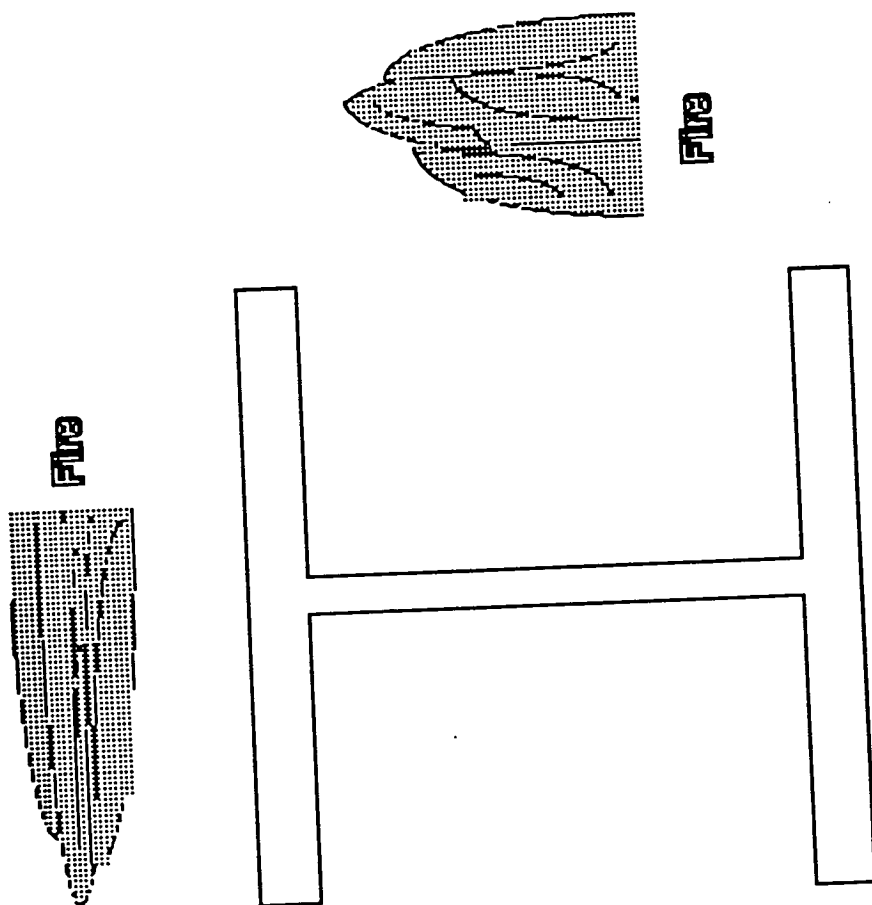


Fig. 3.12a Case D - Cross section profile

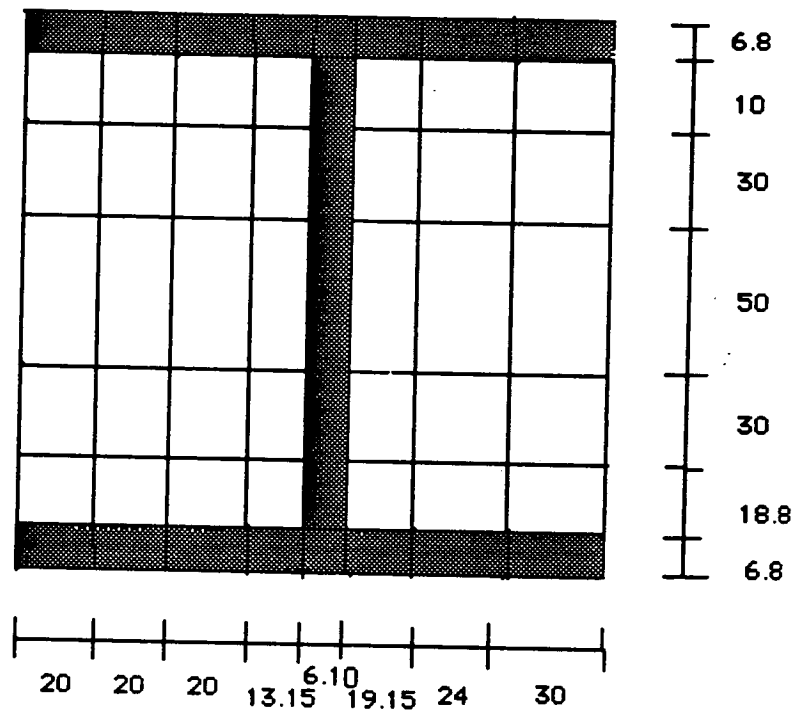
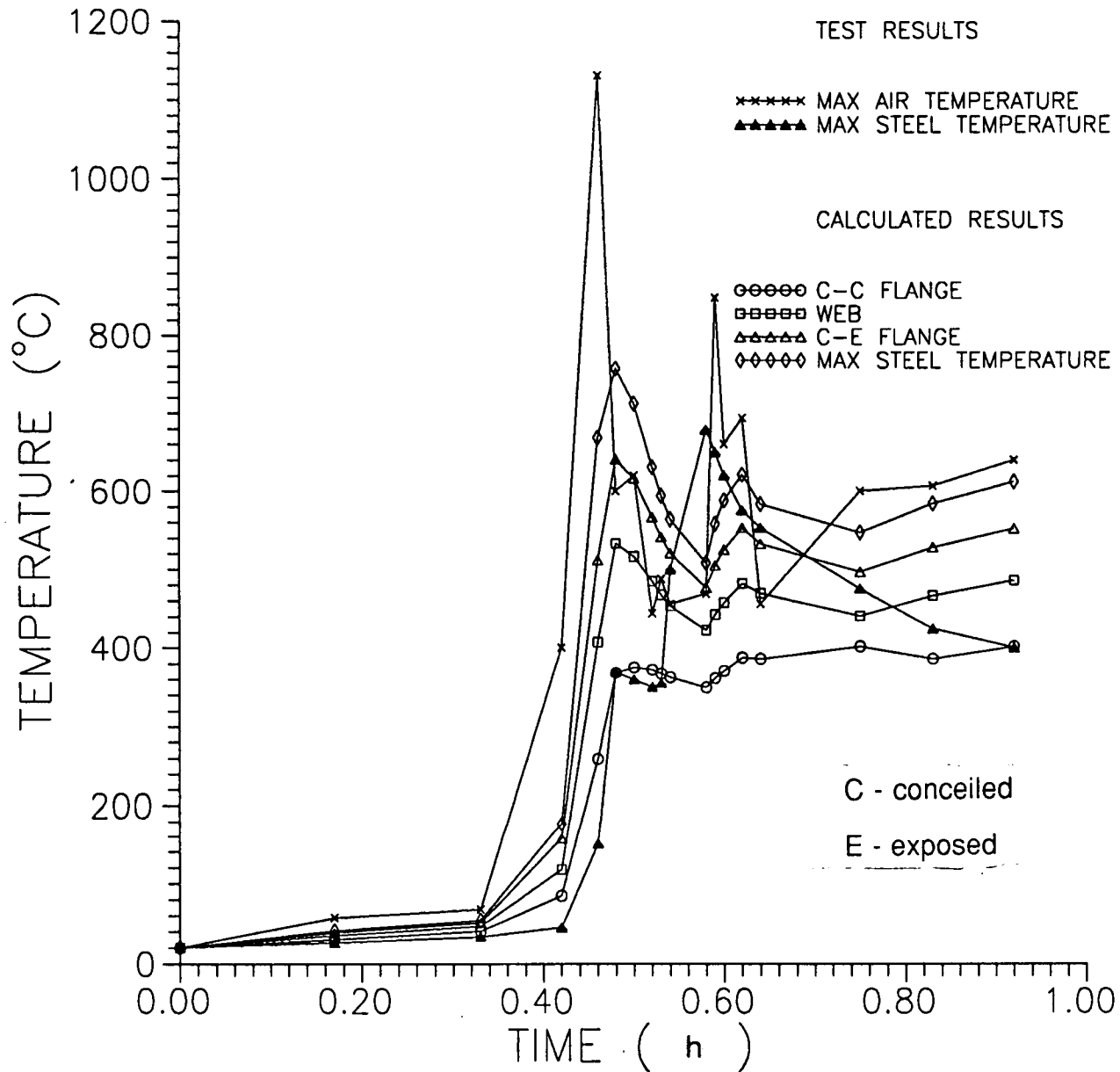
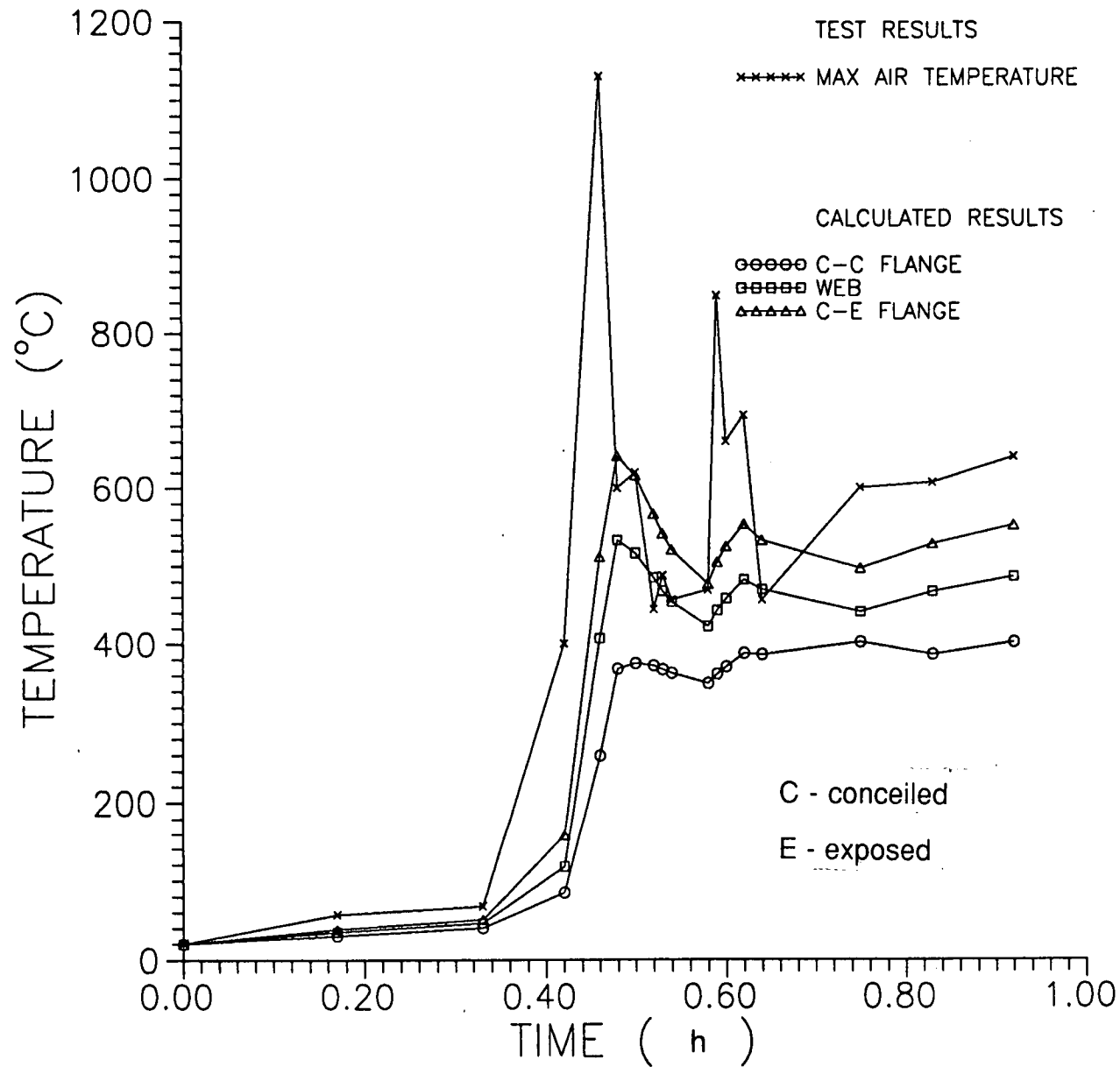


Fig. 3.12b Case D - Cross section finite element mesh

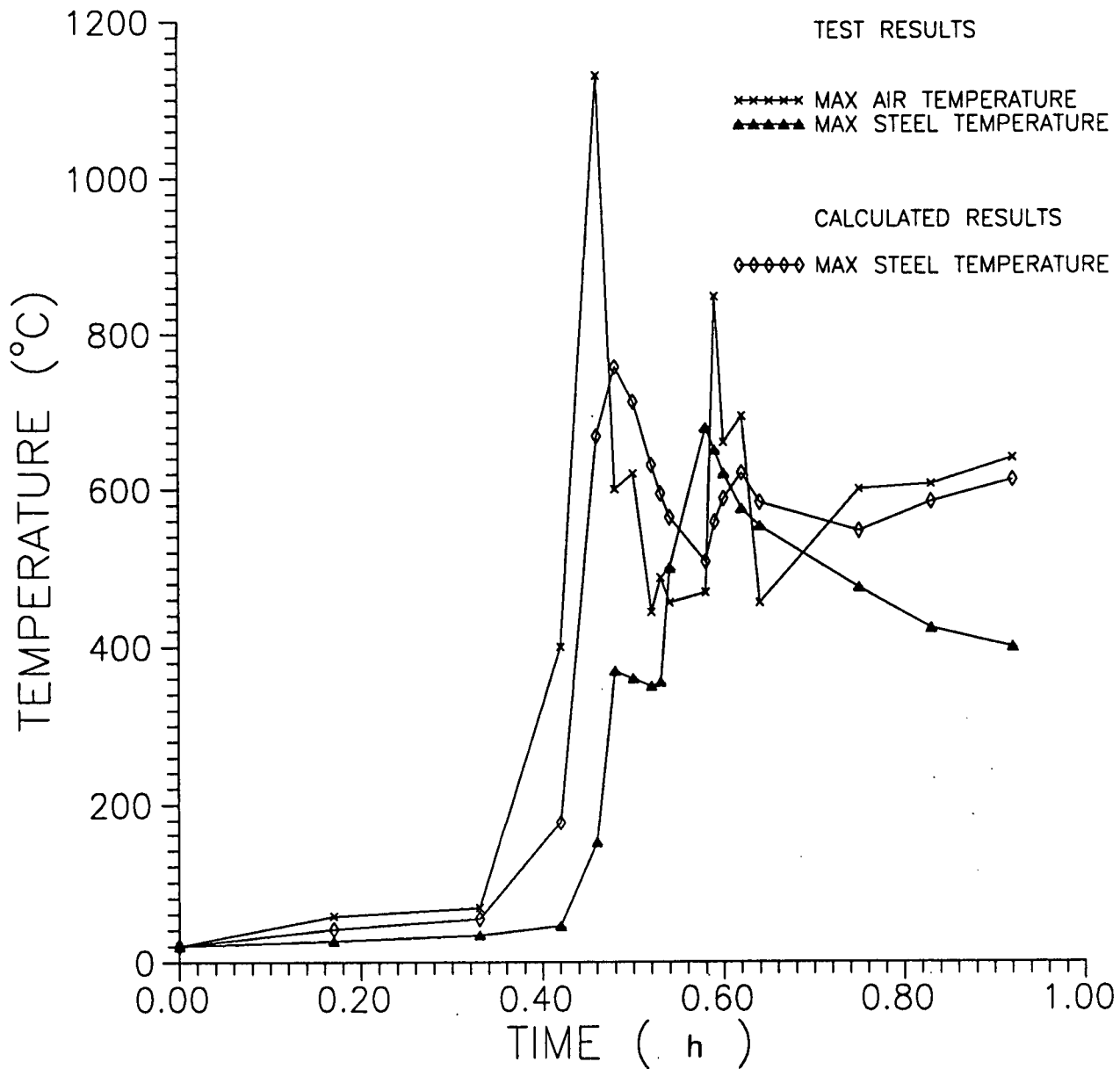
GRAPH 12. TIME-TEMPERATURE PLOT - CASE D



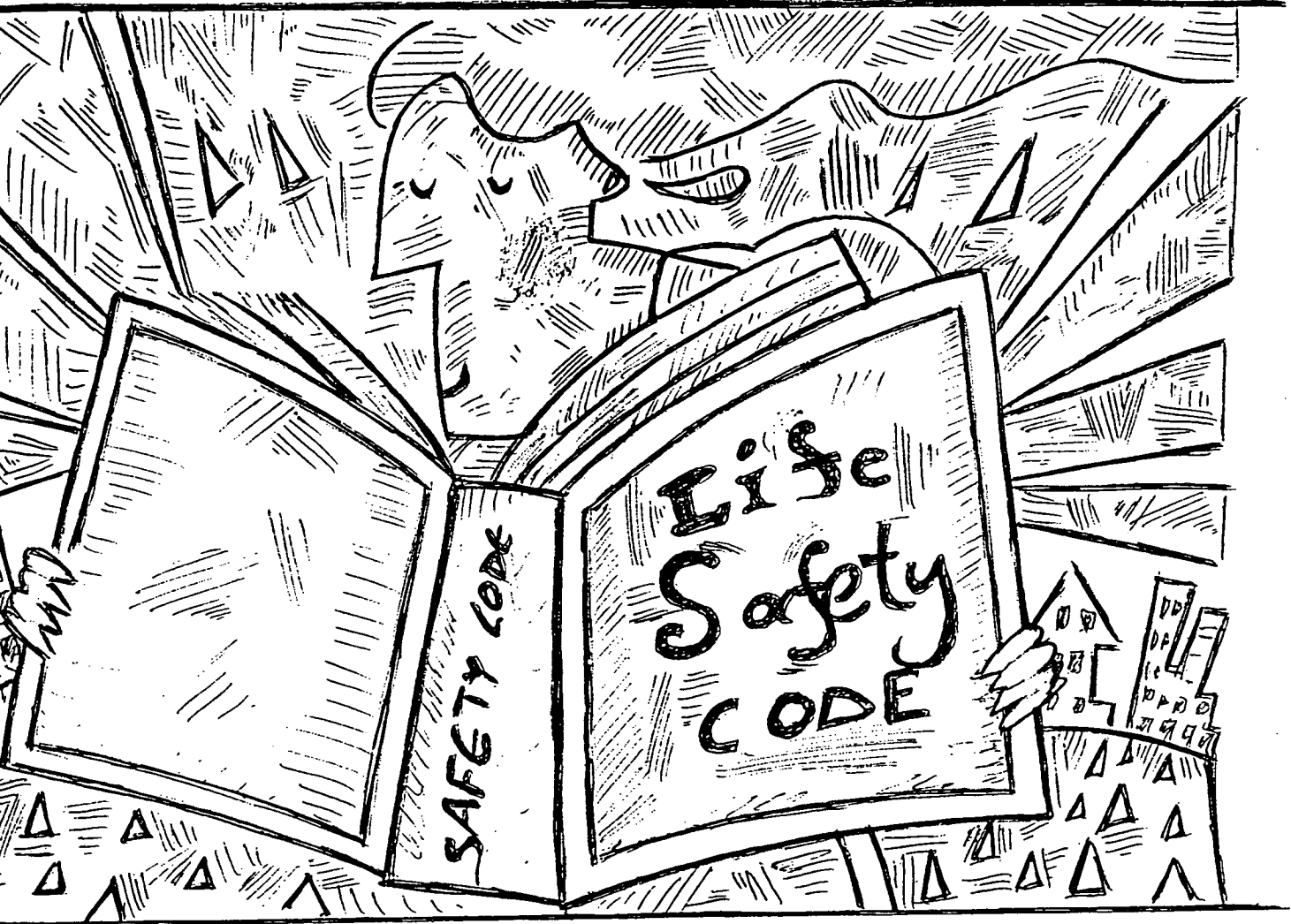
GRAPH 13. TIME-TEMPERATURE PLOT - CASE D



GRAPH 14. TIME-TEMPERATURE PLOT - CASE D



PART II: Structural Problem



CHAPTER FOUR Mechanical properties

The mechanical properties of interest are yield strength, modulus of elasticity, tensile strength, creep. All the properties are strongly influenced by temperature.

4.1 Stress and Strain - Axial loading

4.1.1 Stress-Strain Diagram at room temperature.

To obtain a stress-strain diagram for a material, one usually has to conduct a tensile test on a specimen of the material. One type of specimen commonly used is the one given in Fig. 4.1. The cross-sectional area of the cylindrical central portion of the specimen must be accurately determined and two gauge marks must be inscribed on that portion at a distance L_0 from each other. The testing machine in which the test specimen is then placed, applies a concentric load P . As the load P increases, the distance L between the two gauge marks also increases (Fig. 4.2). The distance L is measured with a dial gauge and the elongation is recorded for each value of P . Simultaneously a second dial gauge is often used to measure the change in diameter of the specimen. For each pair of readings of the load P and the elongation d , one can compute the stress by dividing the load by the original cross sectional area and the strain by dividing the elongation by the original length L_0 . The stress strain diagram is then obtained by plotting the strain as the abscissa and the stress as an ordinate. One can divide the materials on the basis of their common characteristics from the stress-strain diagrams into two broad categories; ductile materials and brittle materials.

Ductile materials are characterized by their ability to yield under normal temperature.

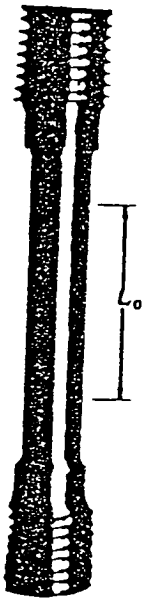


Fig. 4.1

Typical tensile - test specimen
(Beer, 1979).



Fig. 4.2

Test specimen subjected to
tensile load (Beer, 1979).

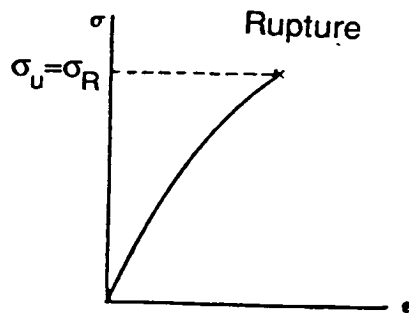


Fig. 4.3

Stress - strain diagram for a typical
brittle material (Beer, 1979).

Brittle materials are characterized by the fact that rupture occurs without any noticeable prior change in the rate of elongation (Fig.4.3).

Structural steel belongs to the first category which has characteristics as follows. As the specimen is subjected to an increasing load, its length increases linearly and at a very slow rate - the initial portion of the stress- strain diagram is a straight line with a steep slope (Fig.4.4). After the value of stress has been reached, the specimen undergoes a large deformation with relatively small increase in applied load. This material property is called strain-hardening and does not characterize the whole range of ductile materials. For example, aluminium behaves differently. When a certain maximum load has been reached, the diameter of the portion begins to decrease due to local instability (Fig.4.5a). This phenomenon is called necking. After necking, the specimen keeps elongating further under lower loads until it finally ruptures (Fig.4.5b). The failure of ductile materials is due to shear. Under an axial load, the shearing stresses are largest on surfaces forming an angle of 45° with the axial load.

The material behaves elastically when the strains caused by the application of the load disappear when the load is removed. This happens when the actual stress is less than the elastic limit of the material. For the case of structural steel the elastic limit coincides with the proportional limit and the yield point. The behaviour of a material is plastic when the strains do not disappear when the load is removed. Plastic deformation depends on the maximum value reached by stress and the time elapsed before the load is removed. The stress dependent part of the plastic deformation is referred to as slip and the time dependent-which is also influenced by the temperature-as creep.

In the elastic range, Hooke's Law applies and the deformation is given by the equation $\epsilon = \sigma/E$. For multiaxial loading, the corresponding components of strain are as follows:

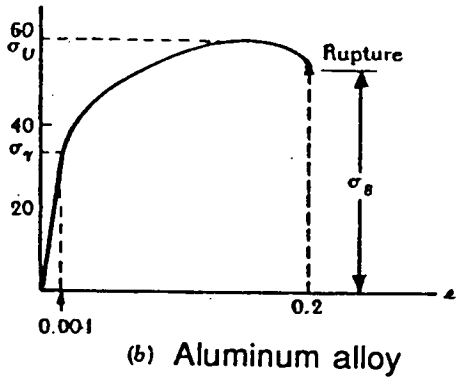
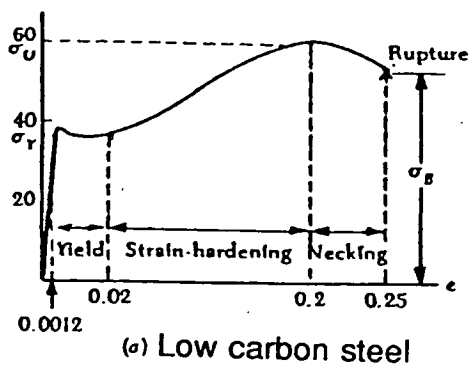


Fig. 4.4

Stress - strain diagrams of two typical ductile materials (Beer, 1979).

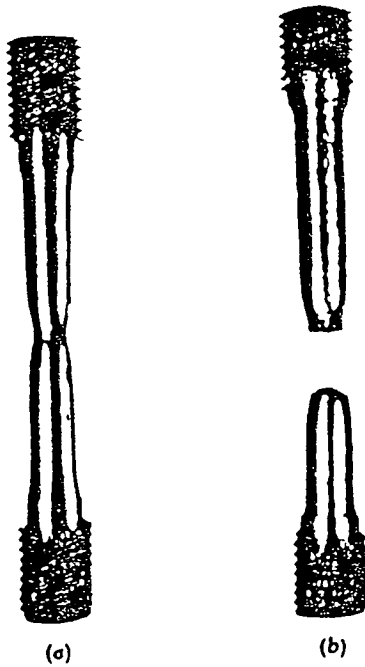


Fig. 4.5 Test specimen (necking - rupture) (Beer, 1979).

$$\begin{aligned}
 \epsilon_x &= \frac{\sigma_x}{E} - \frac{\nu\sigma_y}{E} \\
 \epsilon_y &= \frac{\sigma_y}{E} - \frac{\nu\sigma_x}{E} \\
 \epsilon_z &= \frac{-\nu\sigma_x}{E} - \frac{\nu\sigma_y}{E}
 \end{aligned}
 \tag{4.1}$$

where: E = the modulus of elasticity of the material
 ν = the Poisson's ratio

The modulus of elasticity represents the ability of a material to resist deformation, in other words its stiffness. It is expressed in the same units as stress. BS 5950: Part 1: 1985 recommends the value of $E=205 \text{ kN/mm}^2$ for the design of structural steel at room temperature.

The Poisson's ratio is defined as the absolute value of the ratio of the lateral strain over the axial strain and it quantifies the phenomenon that when a body is pulled, it becomes longer and thinner or when compressed, shorter and thicker. In the literature, most of the given values are derived from the relationship:

$$\nu = \frac{E}{2G-1}
 \tag{4.2}$$

where E = modulus of elasticity,
 G = modulus of rigidity.

[Cooke, 1988]

According to Woolman and Mottram, the values of Poisson's ratio derived from the above equation are subject to considerable errors. The same authors suggest that the best estimate for the family of low alloy steels - structural steel is one of its members - are all between 0.27 and 0.30. BS 5950: Part 1: 1985 - Structural Use of Steelwork in Building - recommends the value of $\nu=0.30$ for room temperature design.

4.1.2 Stress-Strain Diagram at high temperatures Modulus of Elasticity

4.1.2.1 Effect of temperature on flow stress

Stresses above the ^{elastic} limit appearing due to the effect of a time variable temperature field on the metal, cause residual deformations and strain hardening, which subsequently can be combined by softening. The temperature at which load is applied can also cause phase transformations and other changes in the metal which can differ from those taking place in the metal under constant conditions of heating (without stresses). These transformations under the simultaneous effect of high temperatures and long-acting stresses may cause unpredictable changes in the properties of the metal and impair its value as a structural material. Temperature directly affects the mechanism of resistance to plastic deformation and fracture. As the temperature increases, the existing obstacles to dislocation movement become less effective and dislocations can move in a crystal under appreciably lower forces, assuming that the plastic deformation of the crystals occurred by slip - twinning deformation is observed at relatively low temperatures.

Slip is the displacement of a portion of a crystal relative to another portion while the crystal structure of both portions remains unchanged. The boundary between the portion where slip has occurred and another portion where it has not is called the dislocation line and is responsible for distortions in the geometric regularity of arrangement of atoms in the original material. A dislocation disappears after slip has been completed and a unit shear has occurred in a crystal (Fig.4.6). The great obstacles to dislocation movement are the internal distortions in crystals which can be obtained by mechanisms of strengthening because the dislocation locking increases the strain resistance of a metal. The effectiveness of the strengthening methods depend on

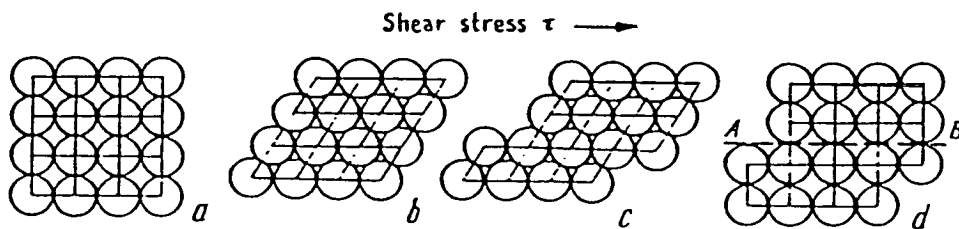


Fig. 4.6 Plastic deformation by slip (Bernstein, 1979).

a-undeformed state of a crystal;

b-elastically deformed;

c-elastically and plastically deformed;

d-plastically deformed in which slip has taken place;

AB-slip plane.

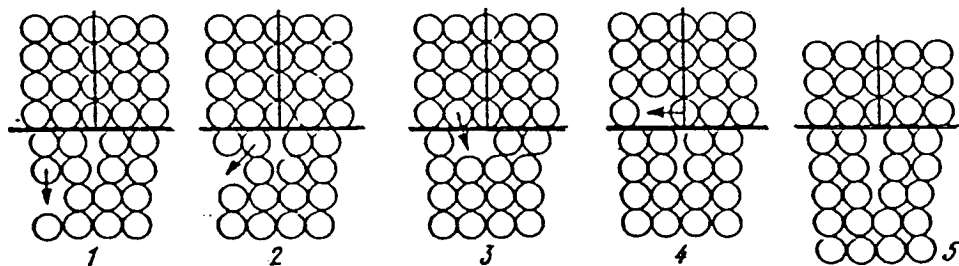


Fig. 4.7 Climb of dislocations from one slip plane to another as a result of self - diffusion and elevated mobility of vacancies at high temperatures; the mobility of atoms in the lattice determines the possibility of dislocation movement in a new slip plane (new position) (Bernstein, 1979).

1 - 5 stages of climb.

temperature. At an increase in temperature, movement of point defects is activated appreciably and dislocation configurations can be changed by the climb mechanism (Fig.4.7). Finally, an increase in temperature can cause recrystallization which diminishes sharply the dislocation density and produces softening.

4.1.2.2 Testing Methods

There are test methods to determine the ability of a material to withstand external loads at elevated temperatures. It is possible to distinguish between two main methods of testing:

- Static methods
- Dynamic methods

4.1.2.2.1 Static methods

The determination of the mechanical properties of metals in tests under static loading is the most popular method. For determination of the stress - strain diagram at elevated temperature, one is interested in the variation of flow stress of a steel sample as a function of temperature. Regarding this, one may distinguish between tests under transient heating conditions and steady-state heating conditions. The three main test parameters are the heating process, application and control of the load and control of the strain.

- Tests under steady-state heating conditions

Steady state is characterized by the steady state temperature curve as shown in Figure 4.8.

The strain measured before the load is applied corresponds to the thermal expansion. Four practical regimes can be used to determine the mechanical properties.

- stress-strain relationship (stress rate controlled)

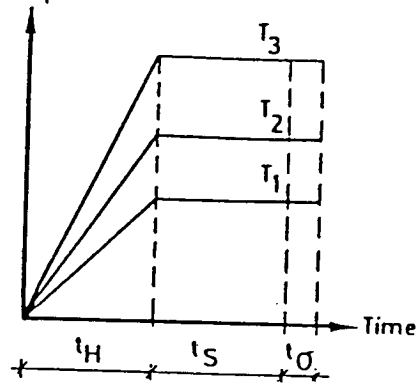


Fig. 4.8 Typical steady state temperature curve (Anderberg, 1983).

t_H =heating period

t_S =stabilizing period

t_σ =loading period (1-3 minutes).

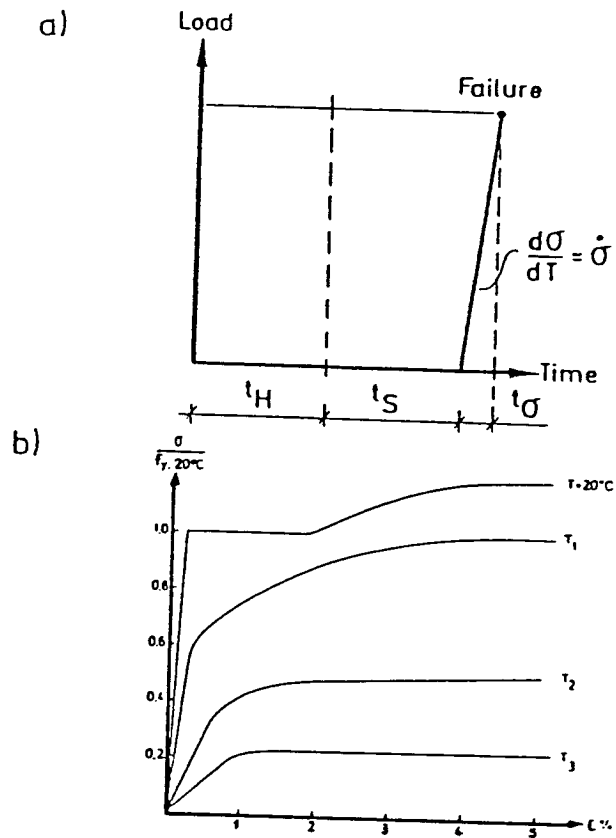


Fig. 4.9 Stress rate controlled σ - ϵ tests (Anderberg, 1983).

a. Loading procedure

b. Typical results

-stress-strain relationship (strain rate controlled)

-creep

-relaxation

a/ Stress rate controlled " σ - ϵ " tests are characterized by the loading procedure as shown in Figure 4.9a.

The test may be used to provide the following data :

-Compressive-tensile strength

-Modulus of elasticity

-Ultimate strain at collapse

Typical results of the above tests are given in Fig. 4.9b.

The stress-strain relationship is obtained in a high rate of loading -loading time 1-2 minutes- in order to avoid influence of creep which is significant for temperatures above 400°C for ordinary steel. Under the influence of creep the " σ - ϵ " curve is displaced in a way that we can notice a lower ultimate strength.

b/ Strain rate controlled " σ - ϵ " tests are characterized by the deformation procedure as shown in Figure 4.10a.

The test can be used to provide the following data:

-Compressive-tensile strength

-Modulus of elasticity

-Ultimate strain at collapse

-Mechanical dissipation energy

The ultimate strain is related to the maximum stress level and not to the failure state. For a given strain, a maximum stress is reached. For higher strains, the stress decreases somewhat but failure occurs at much higher strains than in the corresponding stress controlled " σ - ϵ " tests.

Typical results are of similar nature to those of the stress rate controlled " σ - ϵ " tests and are given in the Figure 4.10b.

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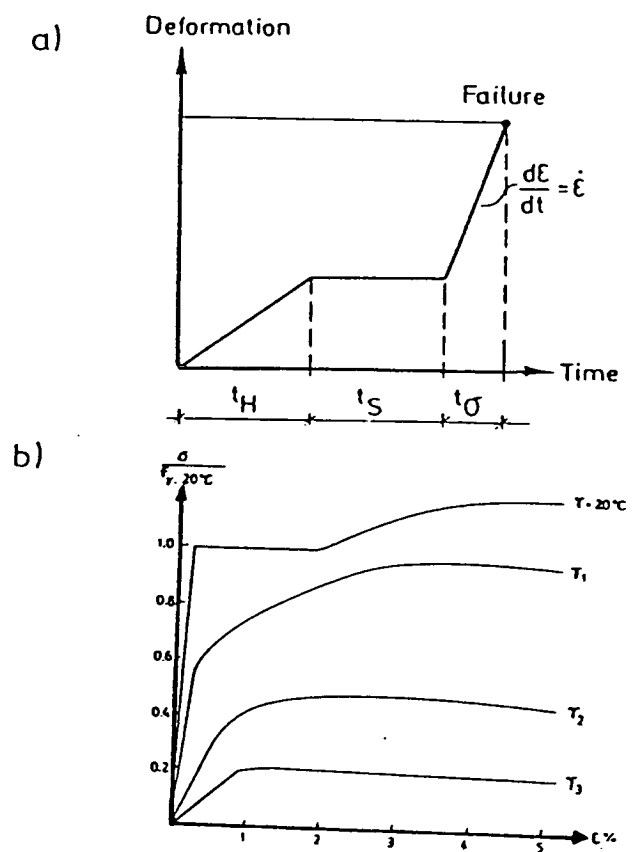


Fig. 4.10 Strain-rate controlled σ - ϵ tests (Anderberg, 1983).

- a. Deformation procedure
- b. Typical results

The test may be repeated at different temperatures in order to develop a family of load-extension curves from which one can extract the required strength-temperature data, examples of which are shown in Figures 4.17, 4.18.

c/ Creep tests are characterized by the following loading procedure as shown in Figure 4.11a.

When the specimen is loaded, the load remains constant during the whole test period

Typical results are given in Figure 4.11b.

The test period of interest is from 2 to 4 hours.

A typical creep curve is given in Figure 4.41, where three phases can be indicated in the creep process which are primary, secondary and tertiary. Usually, the primary and secondary phases are studied.

Tests for creep are usually carried out on unnotched test pieces, cylindrical or rectangular (Fig.4.12). BS18 recommends a gauge length of 40mm to be adopted. The shape of the end portions of the test pieces should match the design of the extensometer for measuring deformations. Testing machines are classified according to the type of loading mechanism as: a) machine with direct loading by weights, b) lever-type machines with upper lever, c) lever-type machines with lower lever, and d) double-lever machines. Figure 4.14 shows a scheme of a lower-lever machine. Heaters used in creep testing should meet the following requirements:

- the temperature of the test piece must be uniform along the whole gauge length,
- the temperature must be constant during the test,
- the device must ensure reliable and continuous heating for a sufficiently long time.

The allowable temperature difference along the gauge length of a test piece is ± 2 to $\pm 6^{\circ}\text{C}$ depending on the gauge length and testing temperature.

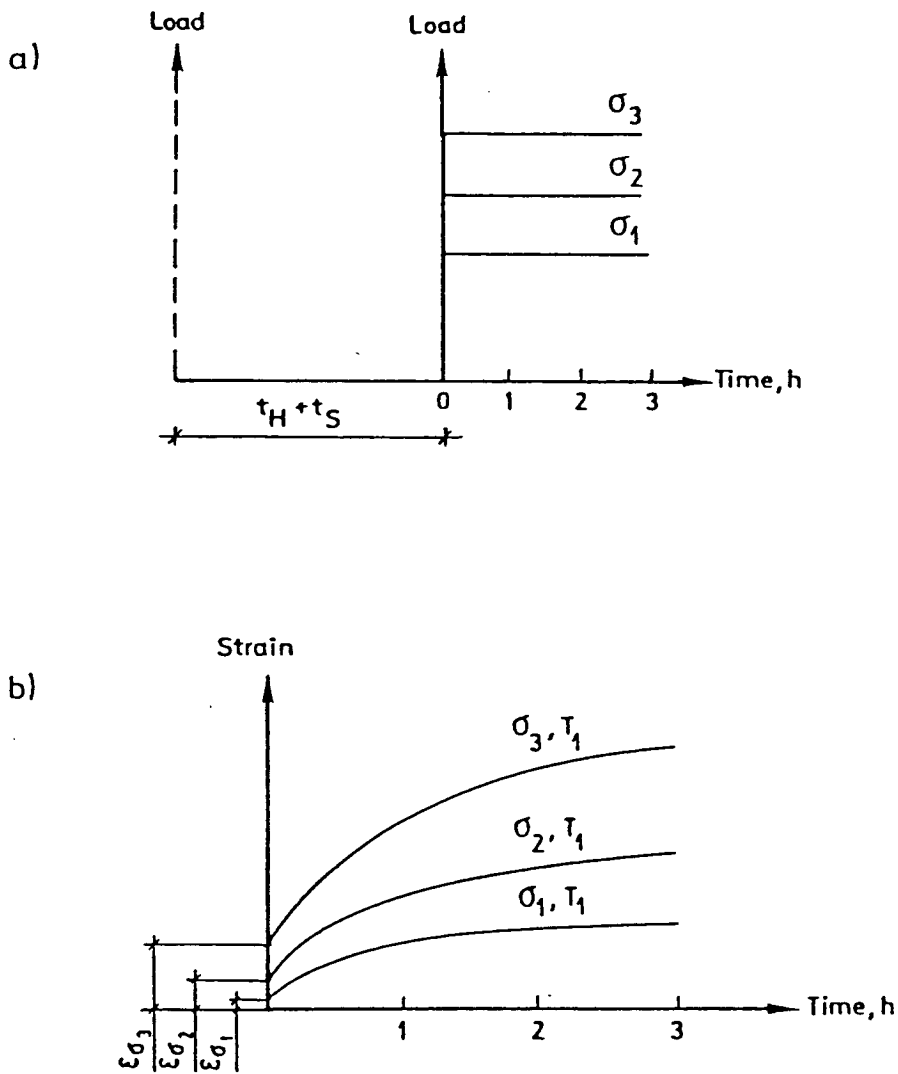


Fig. 4.11 Creep tests (Anderberg, 1983).

a. Loading procedure

b. Typical results

ϵ_{σ} = instantaneous stress-related strain

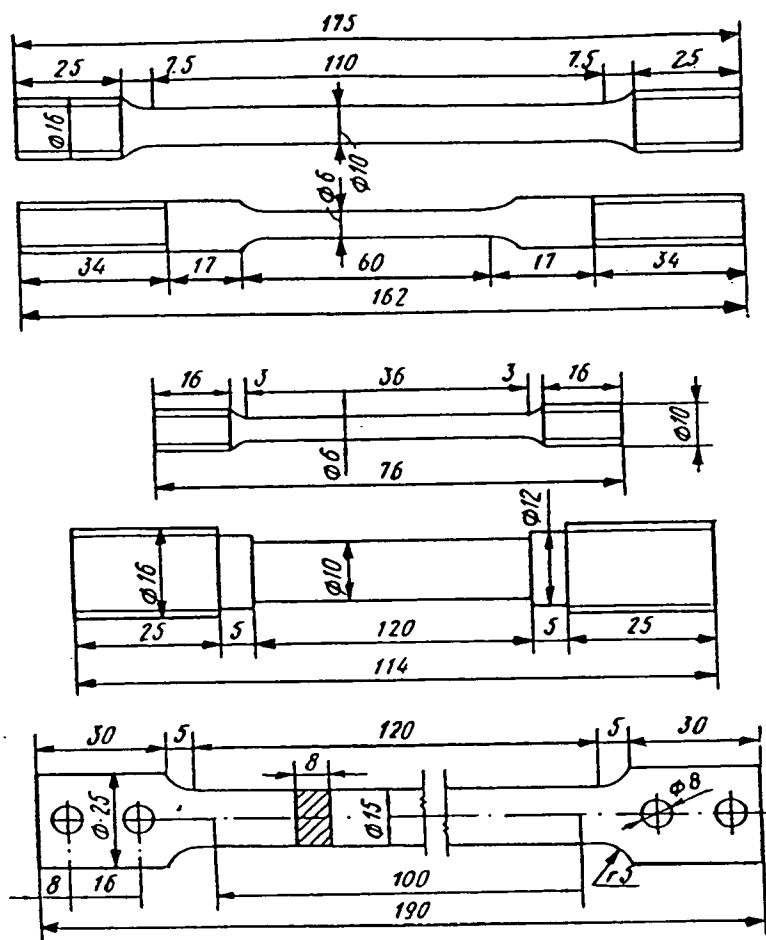


Fig. 4.12 Test pieces for hot tensile tests (creep or long-term strength) (Bernstein, 1979).

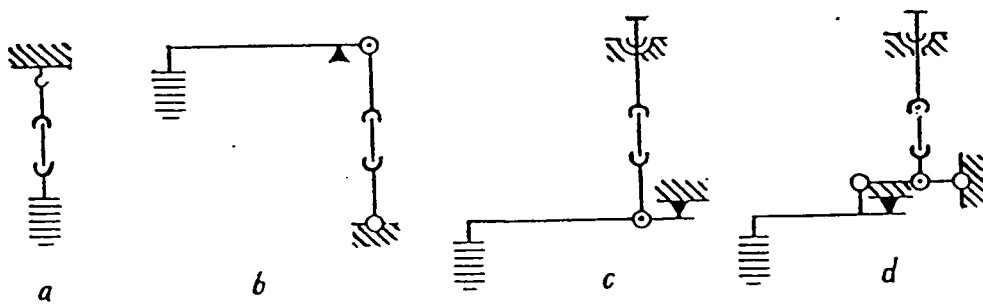


Fig. 4.13 Typical loading diagrams in machines for creep and long-term strength tests (Bernstein, 1979).

The temperature is regulated by means of dilatometric temperature controllers whose action is based on either variation of the length of a reference rod placed into the heater or variations of the dimensions of the metallic muffle of the heater proper.

The creep deformations can be determined by means of indicators. A sample is shown in Figure 4.15 as well as high temperature extensometers. A diagram of an automatic optical recording of creep deformations is given in Figure 4.16 where a light beam from a light source 1 passing through the diaphragm 2 and collecting lens 3 falls onto a mirror 4 attached to the test piece. The deflected beam is directed by mirror 5 on to a drum 6 rotating at constant speed and carrying photographic paper. The light beam records the time-elongation curve on the drum.

d/ Relaxation tests are characterized by the strain history as shown in Figure 4.19a.

When the specimen is loaded, the initial strain is kept constant and the decrease in stress is measured during the whole strain history.

Typical results are given in Figure 4.19b.

The test period of interest is from 2 to 4 hours.

- Tests under transient heating conditions

In tensile tests under transient temperatures, the load of the specimen is maintained constant while its temperature is increased at a given rate (Fig.4.20). The load can be applied before heating or developed during heating by restraint against thermal expansion. These two types of transient tests are carried out with load and strain control respectively. In this way, we can distinguish two testing regimes:

- total deformation, failure temperature (stress control)
- total forces, restraint forces (strain control)

a/ Stress control

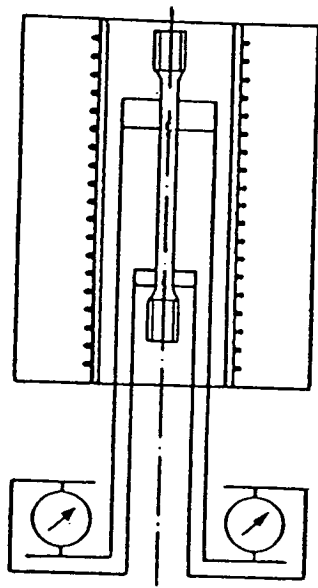


Fig. 4.14 Extenders of an indicator extensometer (Bernstein, 1979).

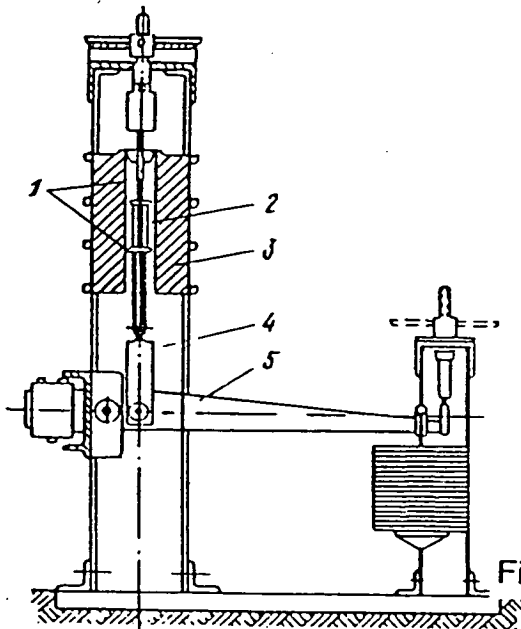
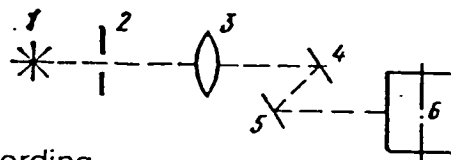


Fig. 4.15 Schematic of a creep-testing machine with lower level (Bernstein, 1979).



4.16 Diagram of automatic optical recording of creep deformations (Bernstein, 1979).

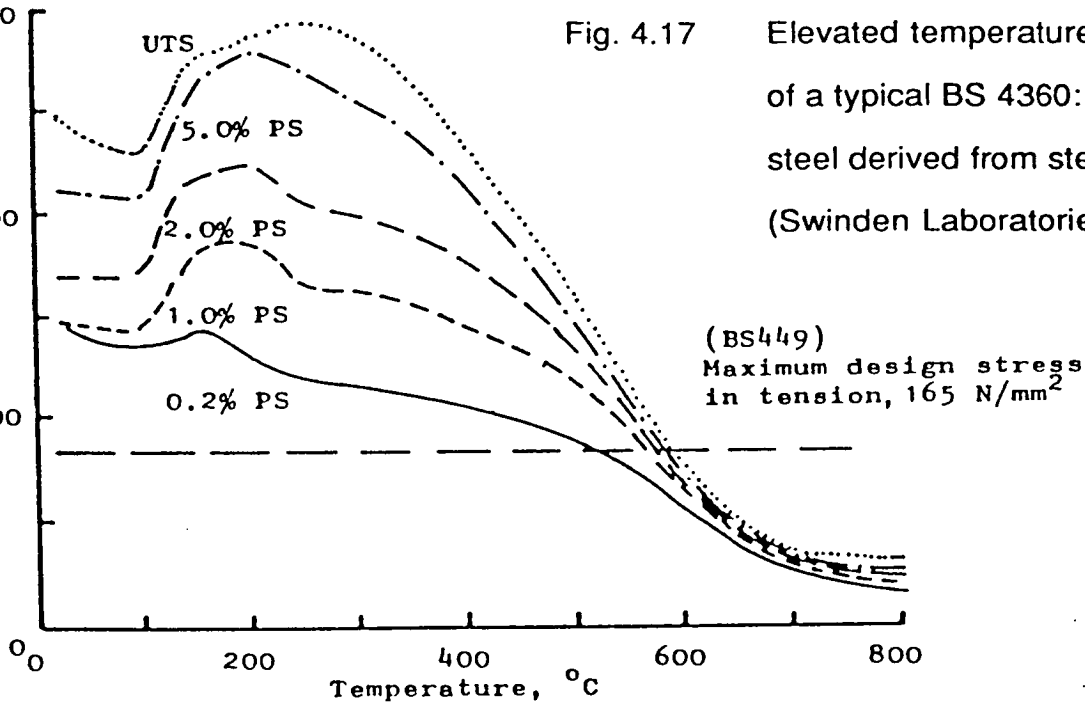
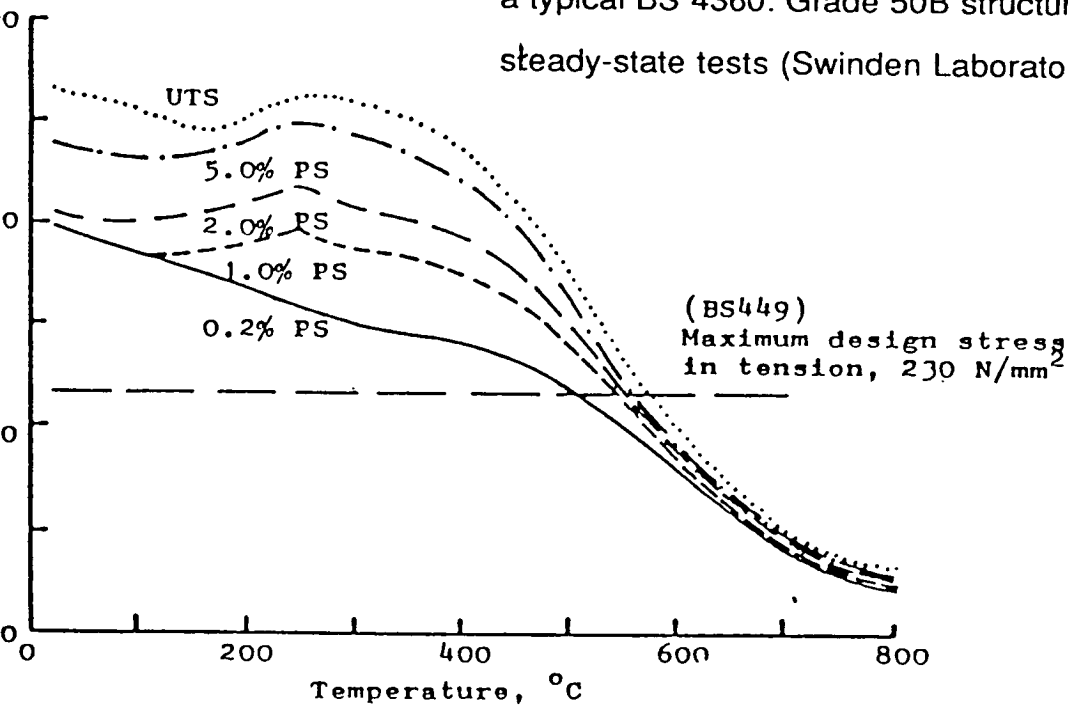


Fig. 4.18 Elevated temperature strength properties of a typical BS 4360: Grade 50B structural steel derived from steady-state tests (Swinden Laboratories) (Kirby, 1988).



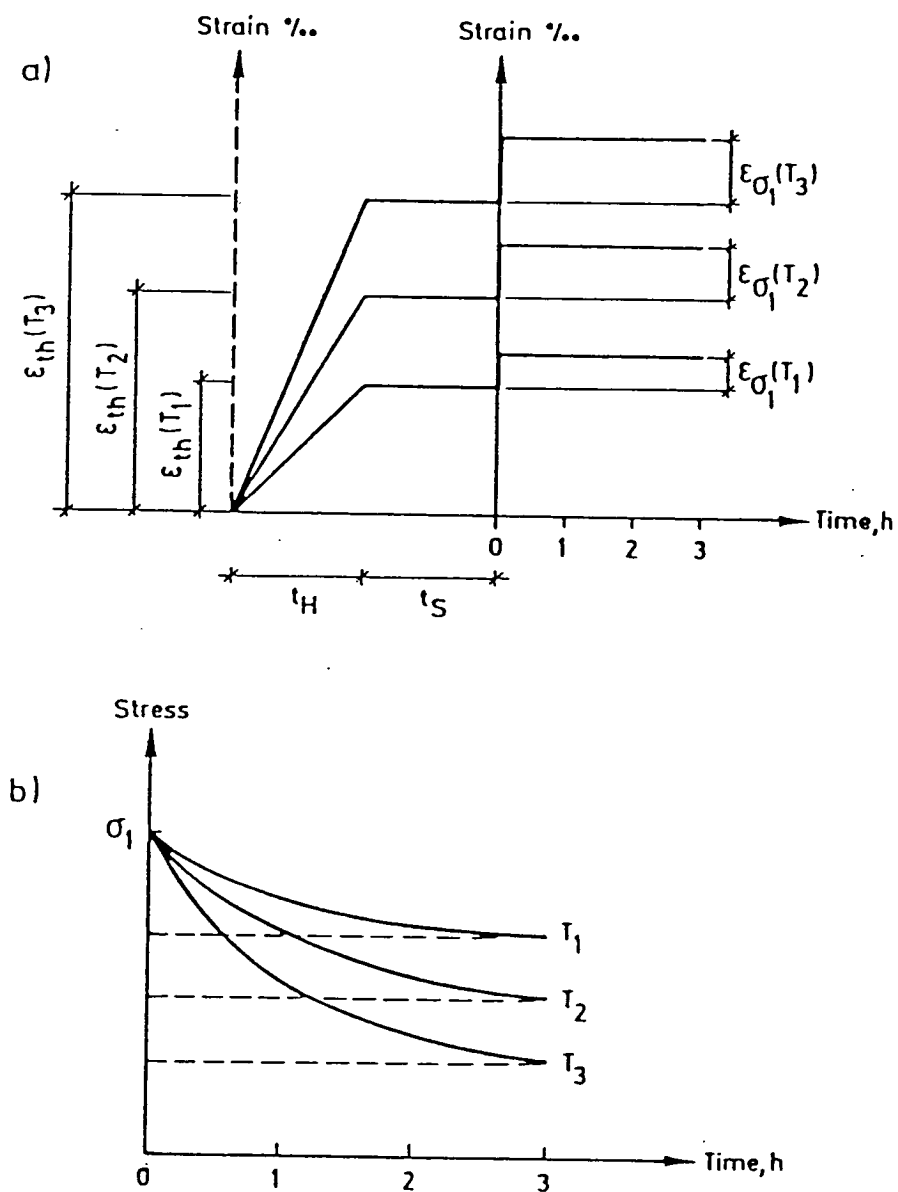


Fig. 4.19 Relaxation tests (Anderberg, 1983).

- a. Strain history
- b. Typical results

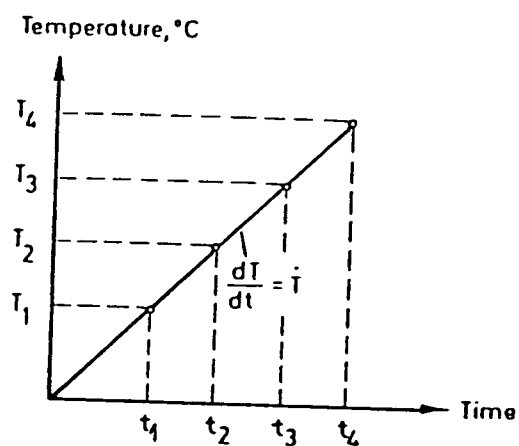


Fig. 4.20 Typical transient state temperature curve (Anderberg, 1983).

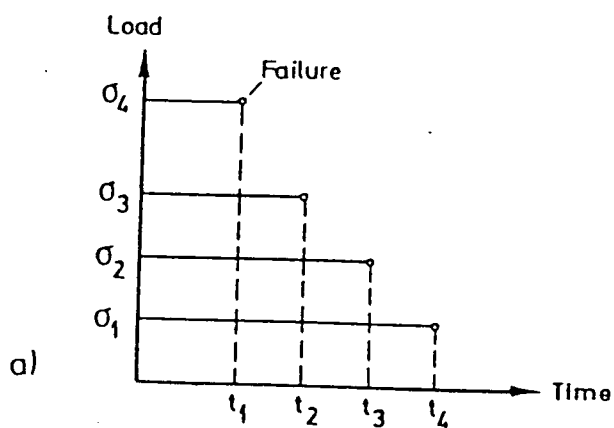


Fig. 4.21 Transient tests with load control (Anderberg, 1983).

a. Load history

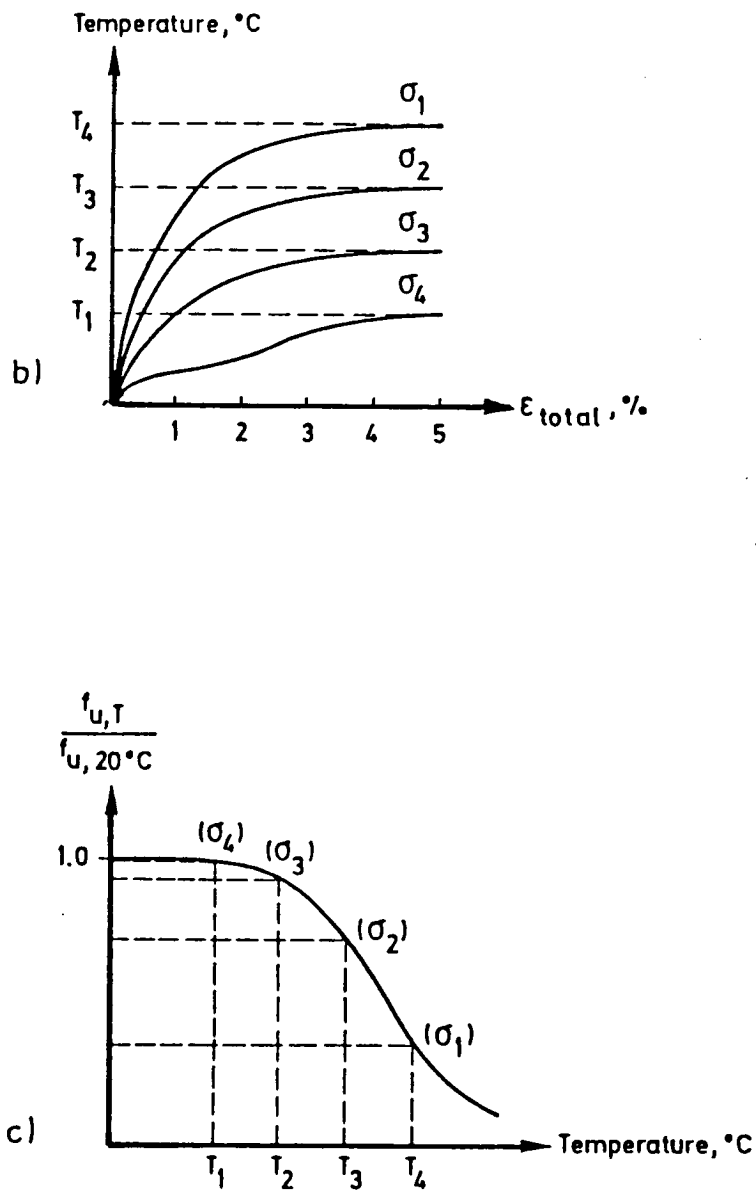


Fig. 4.21 Transient tests with load control (Anderberg, 1983).

b. Typical deformation curves

c. Typical strength-temperature relationships

The load history of this test is given in Figure 4.21a.

A load is applied to the specimen before heating and then it is usually kept constant throughout the whole test. The heating proceeds at a specified rate within the range 5-50°C/min until failure occurs.

Typical deformation curves are given in Figure 4.21b.

The total deformation is recorded until the failure point where total strain approaches infinity. The temperature measured at the critical points is called the critical (failure) temperature. The results are very much influenced by the rate of heating as creep cannot be avoided. With the aid of these curves, strength-temperature relationships can be provided.

Typical strength-temperature relationships are given in Figure 4.21c.

If we extract thermal strain from the total deformation, one obtains " ϵ -T" curves. (Fig.4.22a,b). From these curves, one can construct " σ - ϵ " curves with creep included. The heating rate has a great influence on " σ - ϵ " curves.

The specimen shape and the testing machine used do not differ from the ones used under steady-state testing conditions described above, except that the temperature provided by the furnace varies with time. An example of such a machine is given in Figure 4.23. A similar one was used to provide elevated temperature data on the behaviour of hot-rolled, structural steel for use in fire engineering studies by British Steel Corporation at Swinden Laboratories.

The heater used is a three-zone, split furnace that was programmed to provide constant specimen heating rates of 2.5, 5.0, 10, 20°C/min which simulated the average heating rates of fully loaded, steel sections surviving approximately 4, 2, 1 and 1/2 hours of fire exposure in the ISO 834 standard fire resistance test. The load was transmitted to the specimens by means of Nimonic pull rods passing through the ends of the furnace and hydraulically gripped in the machine. Specimen strain was monitored by Nimonic extensometers fixed by pressure screws at each end of the gauge length and measuring the elastic, plastic strains, as a result of the applied loads and thermal strain, due to thermal

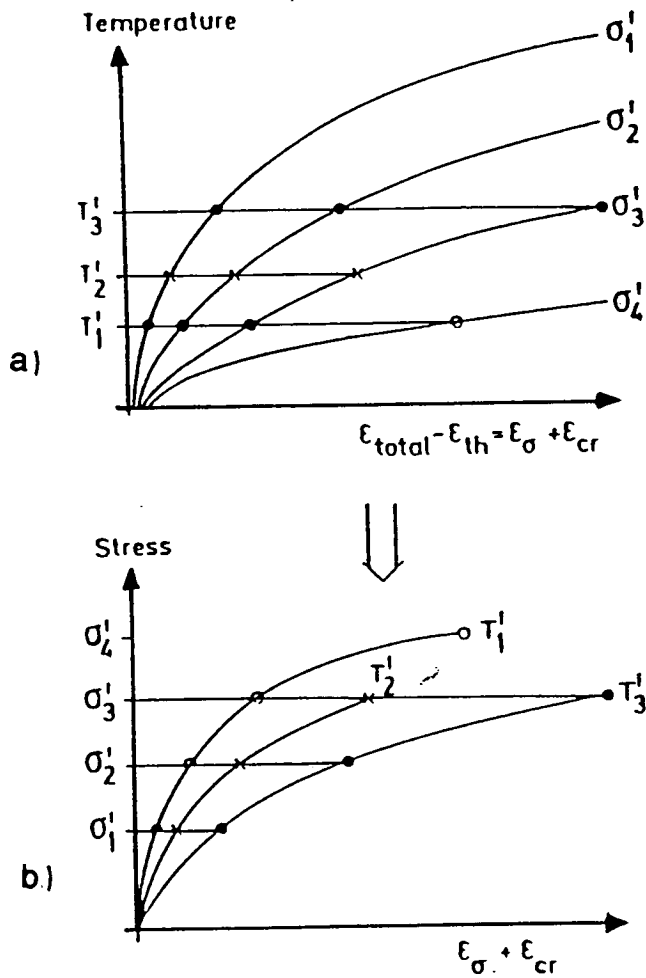
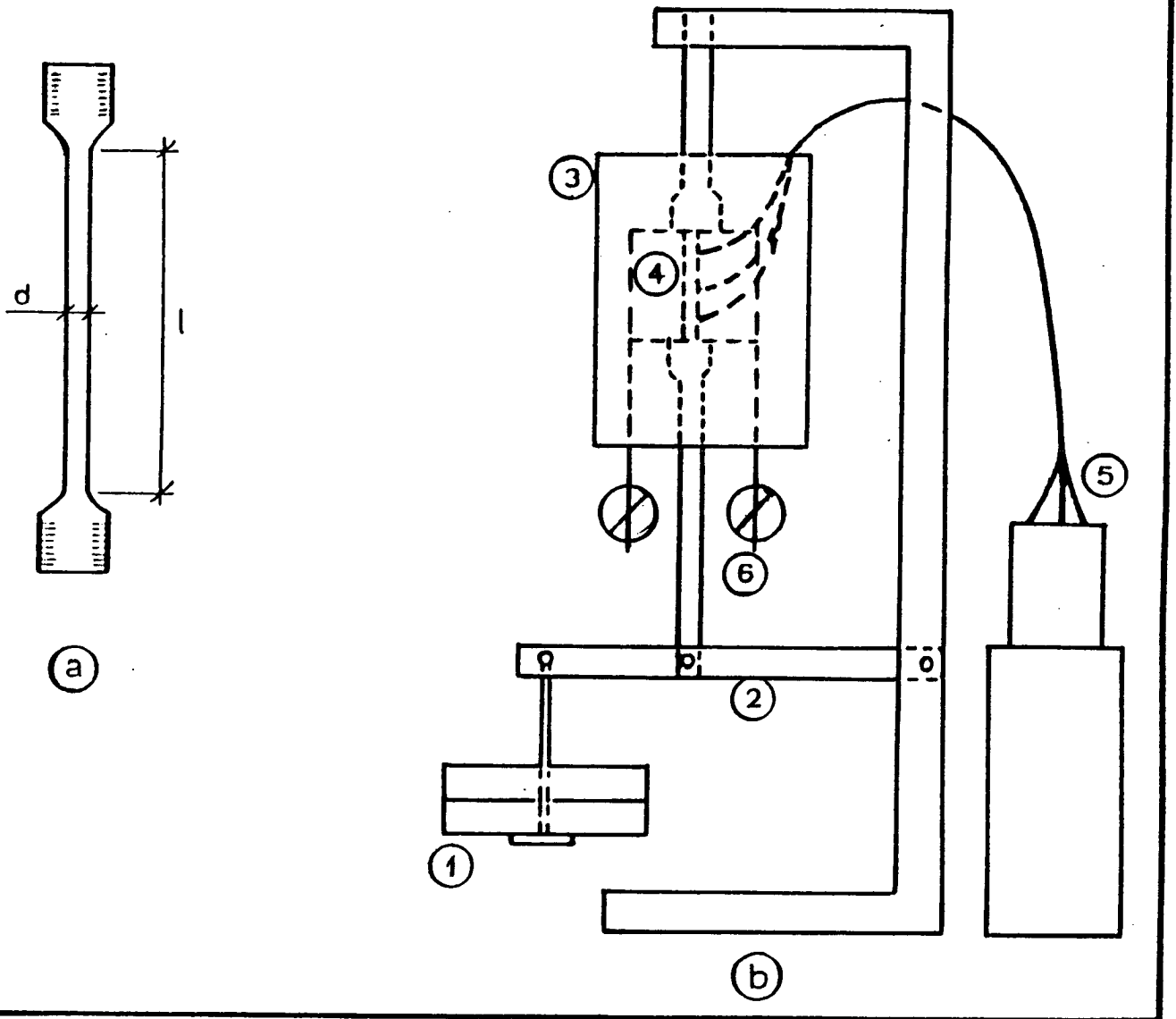


Fig. 4.22 Transient tests with load control (Anderberg, 1983).

a. Typical ϵ - T curves at four different stress levels

$$\epsilon = \epsilon_{\text{tot}} - \epsilon_{\text{th}}$$

b. Calculated σ - ϵ curves (including creep) at different temperatures



- Fig. 4.23 a. Test piece, gauge length $l = 75$ mm, diameter $d = 4.5$ mm.
- b. General arrangement of test rig; 1. weights, 2. system of levers; 3. furnace; 4. test piece with thermocouples, 5. temperature recorder; 6. dial gauges.

(Thor, 1983)

expansion. Separate tests were carried out under zero applied load to measure the thermal strain separately and eliminate it from the results. When a strain of 5% - regarded as the upper limit for practical use of flow stress data - is the one when attained most tests were terminated.

For each steel and heating rate, a series of strain-temperature curves as well as elevated temperature proof stress/room temperature yield stress-temperature curves may be obtained (Fig.4.24).

b/ Strain control

In a strain control transient test, restraint forces arise. In Figure 4.25, the strain history as well as typical curves of restraint forces are given. At temperatures above about 550-600°C, the total force is independent of the initial load which can be zero. If the total load minus the initial load is considered, one will obtain differing curves depending on the varying influence of creep.

This kind of test can only be carried out on steel members in compression with no buckling influence (very short columns).

- Discussion

In international literature, there is little agreement amongst the elevated temperature elastic modulus data for steel grades within the family of structural steels (Fig.4.26). The reasons vary and are related to testing machines, test specimens, testing procedure, results interpretation.

Different tensile test machines have different accuracies measuring very small strains and achieving a uniform and accurately known temperature over the entire gauge length. This inaccuracy of strain measurements affects the stress-strain curve to be affected and consequently the slope of the tangent at its origin which is the modulus of elasticity.

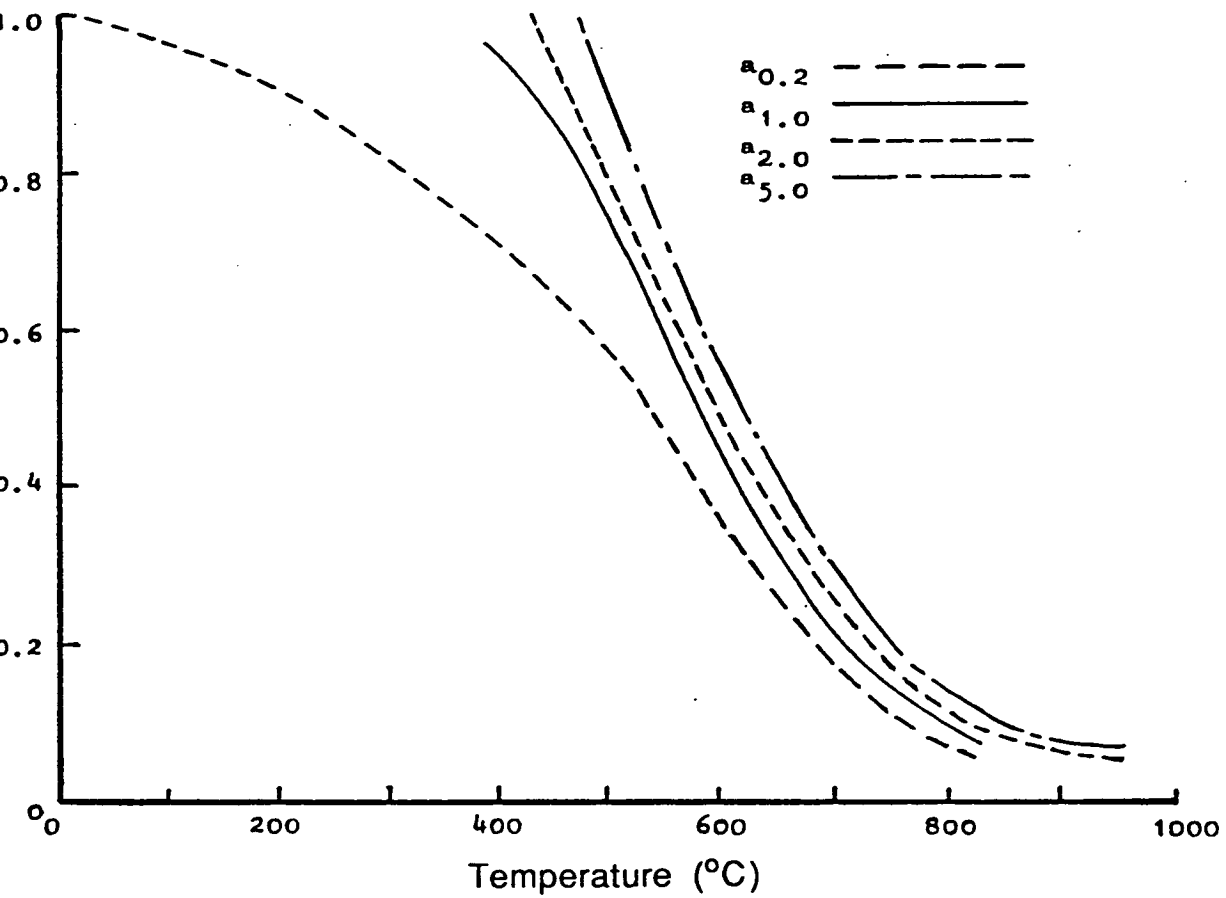


Fig. 4.24 $a_{0.2}$, $a_{1.0}$, $a_{2.0}$, $a_{5.0}$ versus temperature for BS 4360: Grades 43A and 50B structural steels determined from tensile tests conducted under transient heating conditions (Swinden Laboratories) (Kirby, 1988).

The chemical composition of the test specimens can affect the strain measurements. For instance, larger plastic strains are measured with the addition of small amounts of aluminium to improve notch toughness.

For anisothermal testing, one must be careful with the interpretation of the test results. Deducting the thermal strain from the measured strain before plotting the stress-strain curve results in higher derived elastic modulus values.

For isothermal testing, at high temperatures, the soaking period prior to application of stress can relieve residual stresses causing grains elongated in the hot-rolling process to return to their original shape. This could result in shrinkage, and as a result the measured strain in a tensile test is reduced. In isothermal testing, different rates of strain cause different values of measured stress. A fast rate of strain results in higher measured stresses than a slow rate. In Britain, according to BS 3688: Part 1: 1963, which deals with tensile testing, the rate of strain must be within the range of 0.001-0.003 per minute near the elastic limit.

For structures in which the temperature is constant and the stress fluctuates, isothermal data must be used. In building structures subjected to fire, anisothermal data can be used assuming that the applied load is constant, as on a floor above the fire, while the temperature varies. For structures where the load and temperature vary, it is not clear which data should be used.

ECCS recommendation states that for stresses $\sigma \rightarrow 0$, the modulus of elasticity decreases with temperature and may - for all grades of steel and for the temperature range $0 \leq \theta_s \leq 600^\circ\text{C}$.- be approximated by the following equation :

$$E_{\theta} = E(-17.2 \times 10^{-12} \theta_s^4 + 11.8 \times 10^{-9} \theta_s^3 - 34.5 \times 10^{-7} \theta_s^2 + 15.9 \times 10^5 \theta_s + 1) [\text{N/mm}^2]$$

For temperatures over 600°C, the modulus of elasticity is not defined due to the effect of creep which becomes more important and must be analysed explicitly for such temperatures (Fig.4.27).

Arbed - the Luxembourg steel company - reported another relation between elastic modulus versus temperature (Fig.4.25) which is given by the following equations:

$$\begin{aligned} \text{for } \theta \leq 100^\circ; & \quad \frac{E_{\circ \theta}}{E_{\circ 20^\circ}} = 1 & (4.3) \\ \text{for } 100^\circ < \theta \leq 600^\circ\text{C}; & \quad \frac{E_{\circ \theta}}{E_{\circ 20^\circ}} = -0.018\left(\frac{\theta}{100}\right)^2 + 0.036\frac{\theta}{100} + 0.982 \\ \text{for } 600^\circ < \theta \leq 1200^\circ\text{C}; & \quad \frac{E_{\circ \theta}}{E_{\circ 20^\circ}} = 9.25926 \times 10^{-5} \left(\frac{\theta}{100}\right)^3 + 0.0125\left(\frac{\theta}{100}\right)^2 - 0.34\left(\frac{\theta}{100}\right) + 2.12 \\ 1200^\circ\text{C} < \theta; & \quad \frac{E_{\circ \theta}}{E_{\circ 20^\circ}} = 0 \end{aligned}$$

The variation of the plastic modulus E^* with temperature reported by the same company is given below:

$$\begin{aligned} \text{for } \theta \leq 300^\circ\text{C}; & \quad \frac{E^*_{\circ \theta}}{E^*_{\circ 20^\circ}} = 5 \times 10^{-5} \theta + 10^{-2} \\ \text{for } 300^\circ < \theta \leq 600^\circ\text{C}; & \quad \frac{E^*_{\circ \theta}}{E^*_{\circ 20^\circ}} = -7 \times 10^{-5} \theta + 4.6 \times 10^{-2} & (4.4) \\ \text{for } 600^\circ\text{C} < \theta; & \quad \frac{E^*_{\circ \theta}}{E^*_{\circ 20^\circ}} = 0.4 \times 10^{-2} \end{aligned}$$

The above relationships are used in a finite element computer program called CEFICOSS (Computer Engineering of the Fire resistance for COmposite and Steel Structures). Creep is not taken into account in an explicit way but implicitly

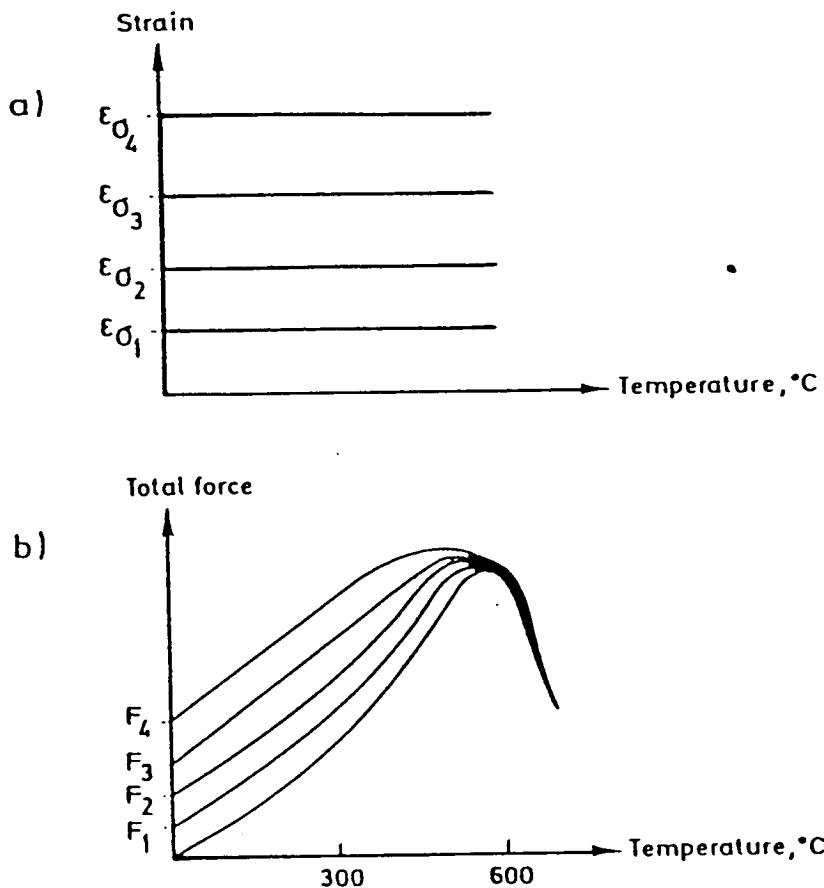


Fig. 4.25 Transient tests with strain control (Anderberg, 1983).

- a. Strain history
- b. Typical curves of restraint forces

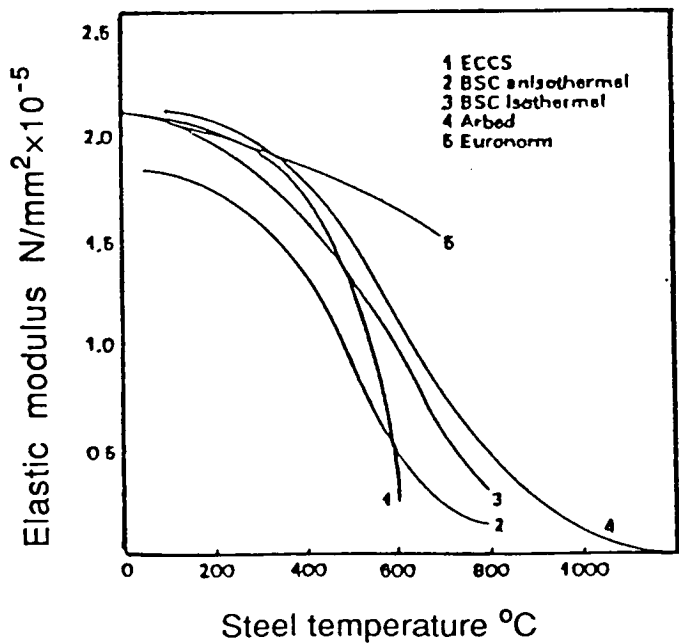


Fig. 4.26
Variation of elastic modulus with temperature (Cooke, 1987).

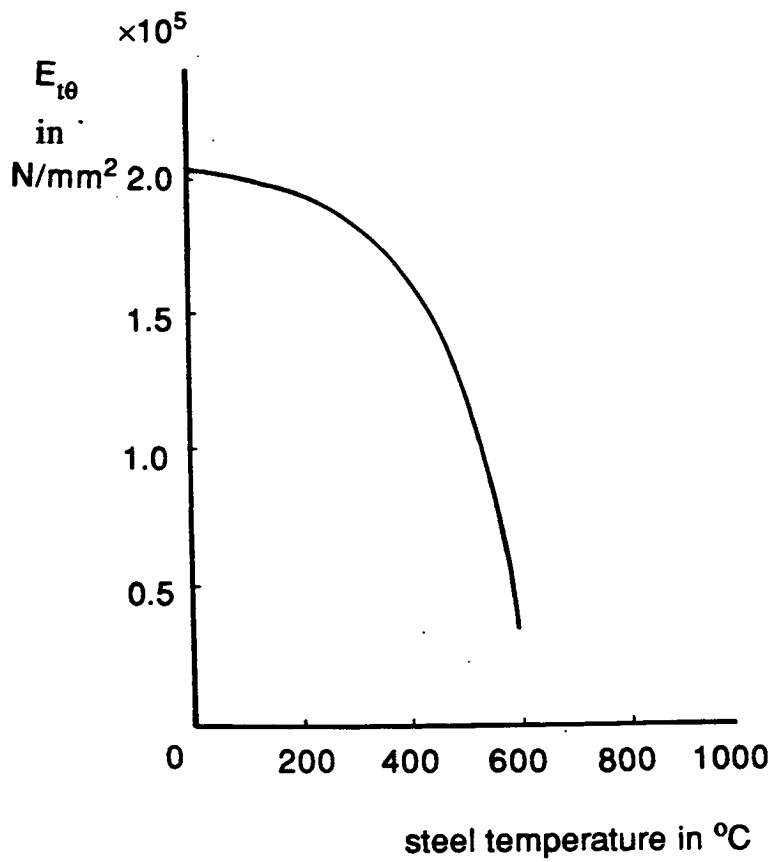


Fig. 4.27 Modulus of elasticity E_{10} as function of the steel temperature (ECCS, 1983).

by means of softened stress related strain. This happens due to lack of data and because it is considered as a phenomenon of non-importance in real structures.

ECCS recommends an elastic modulus-temperature relationship given in Figure 4.28.

The curve in Figure 4.26 has been developed from G.M.E. Cooke who averaged the results that have been taken from an international survey made in 1980 from Stirland C., British Steel Corporation Teesside Laboratories which included dynamically and statically derived data for different grades of Euronorm 25-1972 structural steel. Kirby, SCI Sheffield Laboratories, developed isothermal and anisothermal elastic modulus data. In the anisothermal testing, steel grades 43A, 50B (BS 4360:1979) were tested, the heating rates were 20,10,5,2.5°C/min, the applied stress-constant in each test/ varied from 15-250 N/mm² for grade 43A and from 15-400 N/mm² for grade 50B and the test terminated at 5% strain.

The anisothermal curves, developed by Kirby (1988), are given in Figures 4.28, 4.29.

4.1.2.2.2 Dynamic methods

The dynamic methods are applicable at lower deformations where they possess higher sensitivity than the static methods which present the risk that the test will pass beyond the elastic region. In a dynamic test, the plastic and creep strains are prevented from occurring due to the rapid stress reversal. This produces a greater value for the modulus of elasticity than the statically derived one.

The specimen is usually a wire caused to vibrate. One end of the specimen is usually held in a fixed clamp and the other oscillates freely across the path of a light beam from a lamp to a photocell. The modulus of elasticity is determined from the measured frequency of vibration at each temperature.

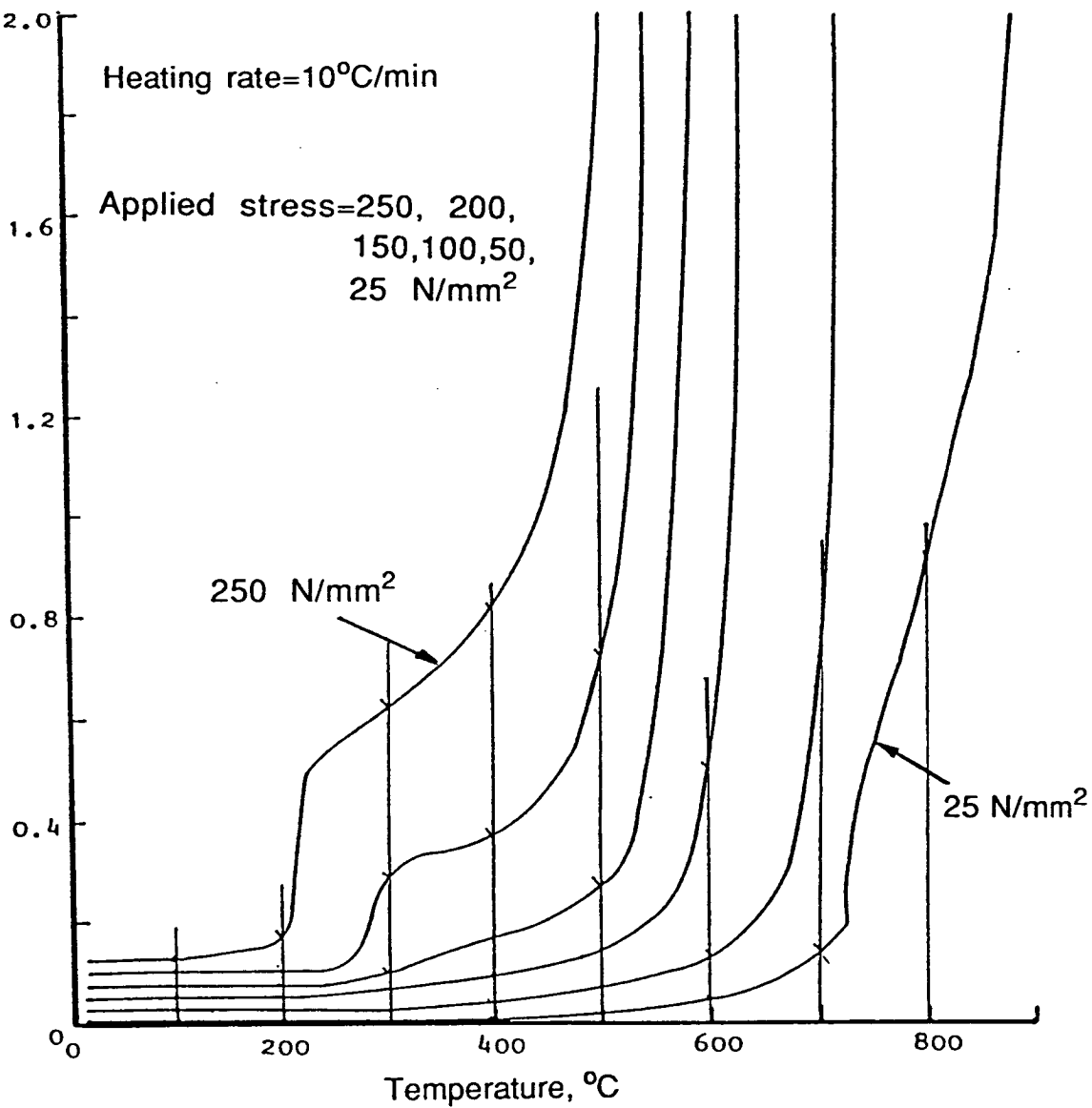


Fig. 4.28 Tensile curves for a Grade 43A steel derived from transient tests (Kirby, 1988).

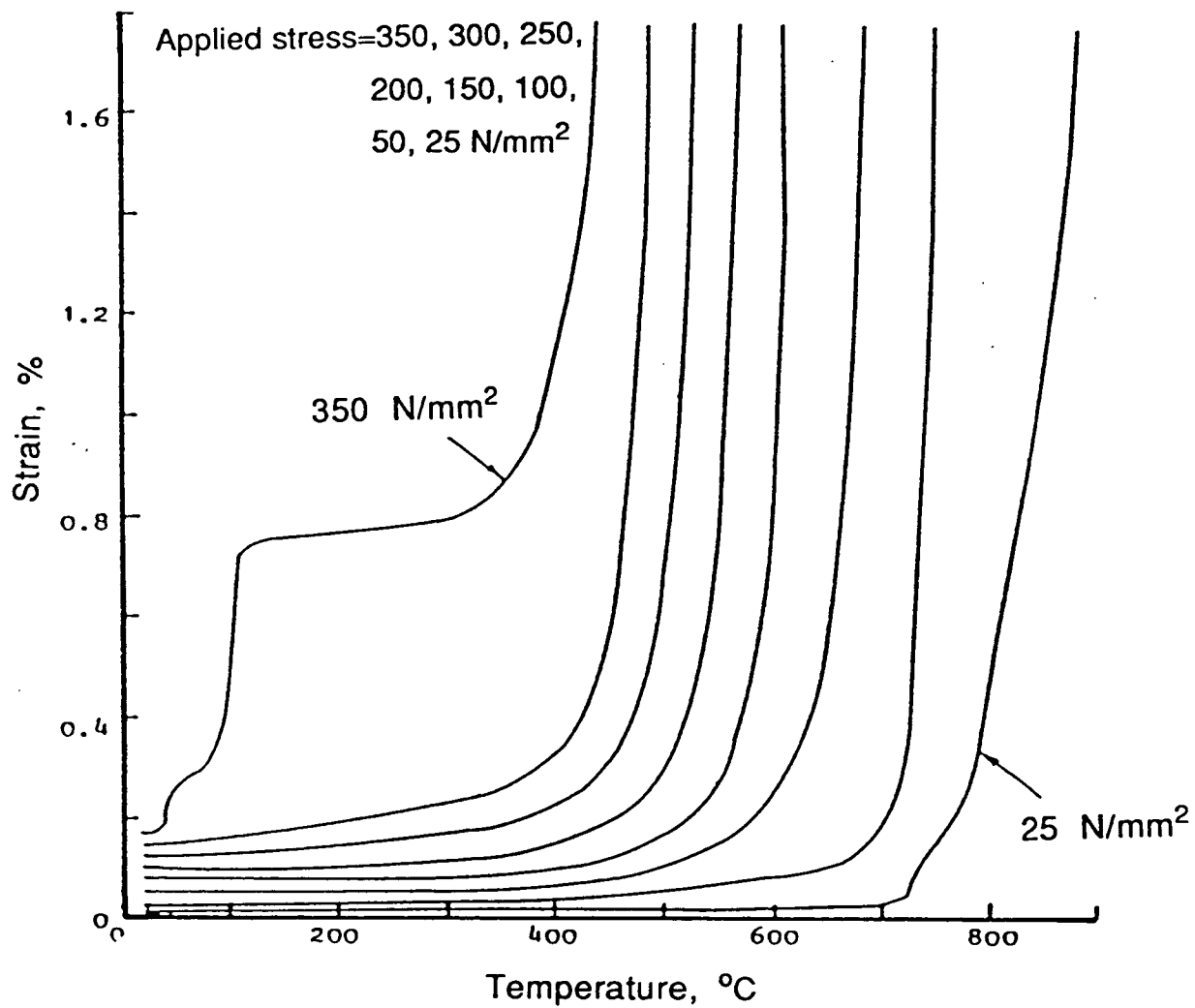


Fig. 4.29 Tensile curves for a Grade 50B steel derived from transient tests (Kirby, 1988).

4.1.2.2.3 Suggestions

Generally, because the elastic modulus data can be produced with different test methods, one must specify the test method used.

It is better for researchers concerned with structural analyses to adopt an E versus temperature relationship which, though falling within the scatterband of experimentally determined elastic modulus values, is chosen so that it best correlates with bench-mark data.

4.2 Tensile Strength

The tensile strength can be defined as the ultimate tensile strength (f_u) or the tensile strength related to a specified residual stress induced strain in the " σ - ϵ " curve. It can be obtained either from steady state or transient state tests. Parameters that influence significantly the analysis of tensile strength are given below:

- in steady state tests
 - rate of stress, σ , at stress rate control
 - rate of strain, ϵ , at strain rate control
- in load controlled transient tests
 - rate of temperature increase, $T(^{\circ}\text{C}/\text{min})$,
 - strain rate criteria

The influence of creep is different for each testing procedure.

It is shown that the relative strength decrease is almost the same for all hot-rolled steels. The 0.2% proof stress and the ultimate strength of such steels obtained in steady state and transient state tests have at 500-550⁰C, 50% of its original value left and at 700⁰C about 20%.

4.3 Residual Stress

In hot-rolled steel sections important residual stresses may occur due to the manufacturing process. These residual stresses may affect the structural

Fig. 4.30 Residual properties (Anderberg, 1983).

- a) Structural steel : ① St 60/90, $f_{u,20^{\circ}\text{C}} = 590 \text{ MPa}$
- c) Prestressing steel : ② St 145/165, $f_{u,20^{\circ}\text{C}} = 1650 \text{ MPa}$
 ③ St 160...180, $f_{u,20^{\circ}\text{C}} = 1740 \text{ MPa}$
- Test conditions : No loading during heating or cooling
- Specimen : ① Hot-rolled, $\phi = 26 \text{ mm}$
 ② Cold-drawn, $\phi = 5.2 \text{ mm}$
 ③ Cold-drawn, $\phi = 5 \text{ mm}$
- Chem. composition :
- Remarks : Residual ultimate tensile strength
- Reference : Dannenberg et al (1959)

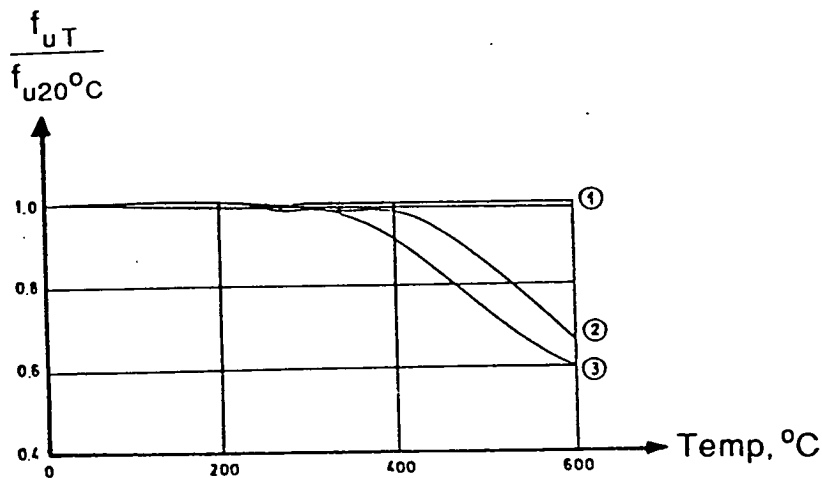


Fig. 4.31 Residual properties (Anderberg, 1983).

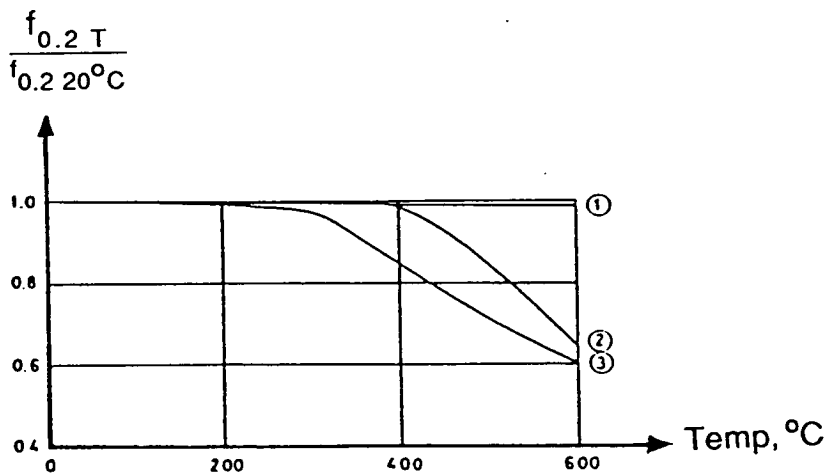
- a) Structural steel : ① St 60/90, $f_{0.2, 20^{\circ}\text{C}} = 590 \text{ MPa}$
 c) Prestressing steel : ② St 145/165, $f_{0.2, 20^{\circ}\text{C}} = 1520 \text{ MPa}$
 ③ St 160...180, $f_{0.2, 20^{\circ}\text{C}} = 1570 \text{ MPa}$
 Test conditions : No loading during heating or cooling

Specimen : ① Hot-rolled, $\phi = 26 \text{ mm}$
 ② Cold-drawn, $\phi = 5.2 \text{ mm}$
 ③ Cold-drawn, $\phi = 5 \text{ mm}$

Chem. composition :

Remarks : Residual 0.2% proof tensile stress

Reference : Dannenberg et al (1959)



behaviour of fire-exposed columns. It is not yet known if the residual stresses play an important role on the buckling load at elevated temperatures.

In some theoretical models, it is assumed by some workers that the residual stresses vary as a function of the yield stress or that the residual stresses vanish at elevated temperatures.

Experimental results presented in Figures 4.30, 4.31 show that the hot-rolled steel is not influenced by temperature as it concerns neither the 0.2% proof stress nor the tensile strength.

In the European Recommendations, it is assumed that the buckling curves at elevated temperatures can be described by equations similar to those derived for ambient temperature conditions.

4.4 Modelling steel behaviour

The deformation process of steel at transient high temperatures can be described by three strain components:

$$\varepsilon = \varepsilon_{th}(T) + \varepsilon_{\sigma}(\sigma, T) + \varepsilon_{cr}(\sigma, T, t) \quad (4.5)$$

where: $\varepsilon_{th}(T)$ = thermal strain,
 $\varepsilon_{\sigma}^{\sigma}$ = instantaneous, stress related strain based on " σ - ε " relationship,
 $\varepsilon_{cr}(\sigma, T, t)$ = creep strain or time dependent strain.

The deformation components are the same at steady state as well as at transient conditions. The strains are found separately in different steady state results.

4.4.1 Thermal strain

Thermal strain or thermal expansion is measured on unstressed specimens under variable temperature and is given approximately by the following equation, assuming linear relationship:

$$\epsilon_{th}^s = \int_0^T \alpha(T) dT \quad (4.6)$$

where: α =coefficient of thermal expansion,
 T =temperature ($^{\circ}C$).

The linear relationship is most often used. The steel and strength characteristics seem to have no significant influence. In a previous chapter (Chapter Two), a discussion has already been given on the variation of thermal expansion with temperature.

4.4.2 Instantaneous stress related strain

Many finite element programs require input of the stress-strain curves. In this way, there is a real need to idealize these curves.

The experimental curve can be approximated in different ways.

The most straight forward way to model a stress-strain curve is to define the yield strength by the offset method. The yield strength at 0.2% offset for example is obtained by drawing through the point of the horizontal axis of abscissa $\epsilon=0.2\%$, a line parallel to the initial straight line portion of the stress-strain diagram (Fig.4.32). The stress σ_y corresponding to the point Y obtained is defined as the yield strength at 0.2% offset.

Some other more sophisticated approximations of the stress-strain curve are given in Figures 4.33,4.34.

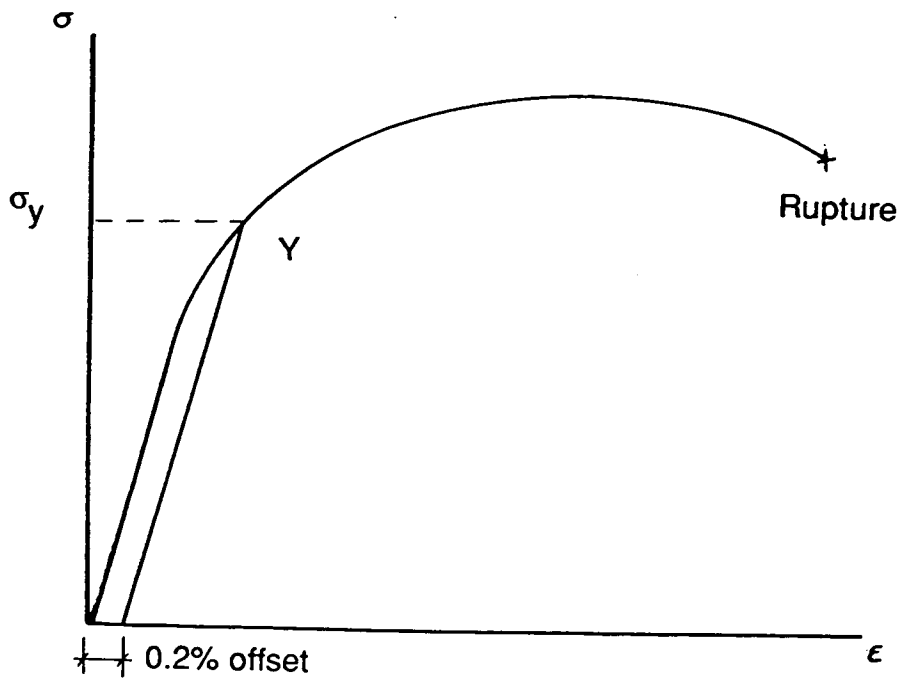


Fig. 4.32 Determination of yield strength by offset method (Beer, 1979).

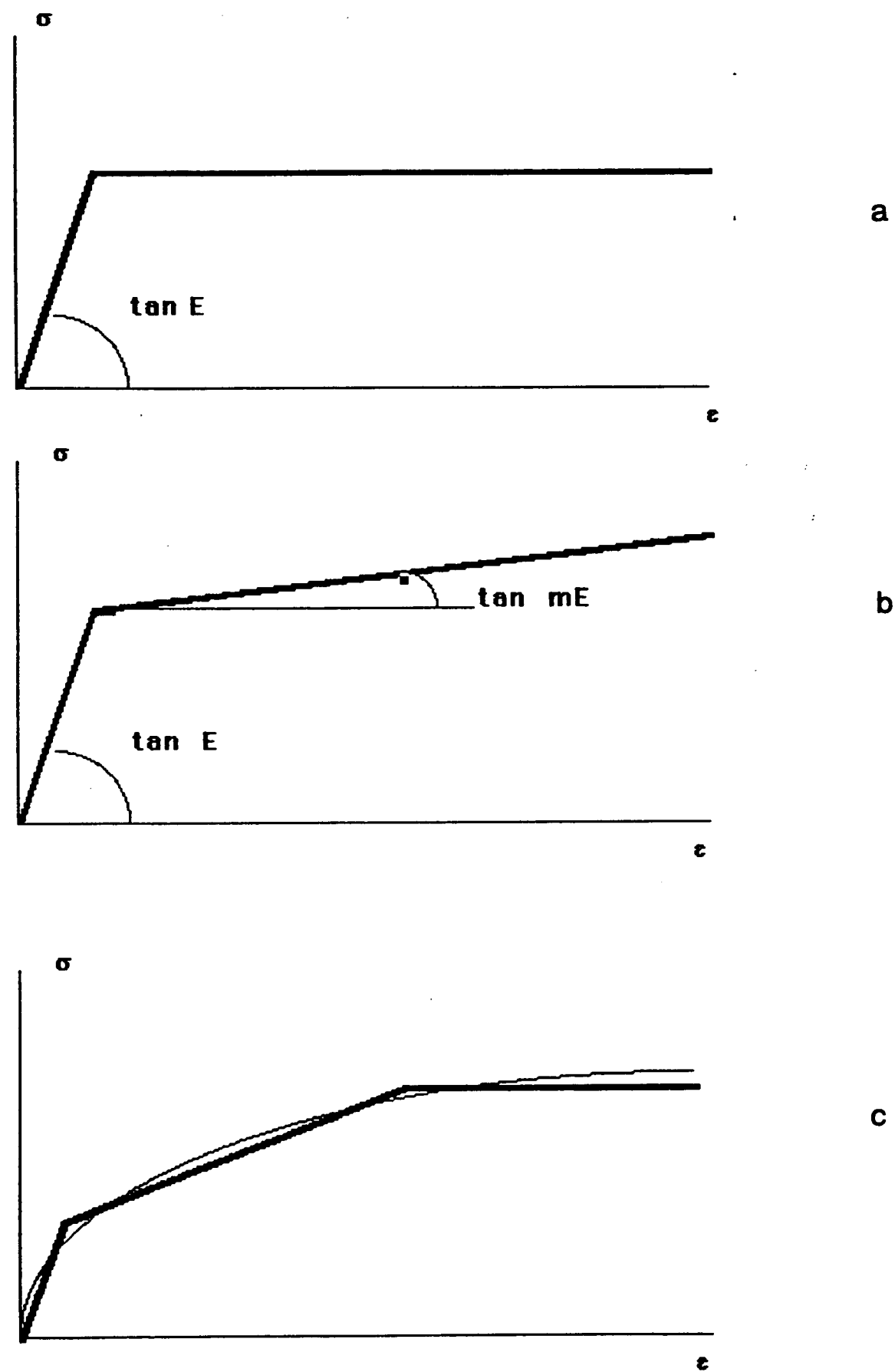


Fig. 4.33 Linear stress-strain curve models

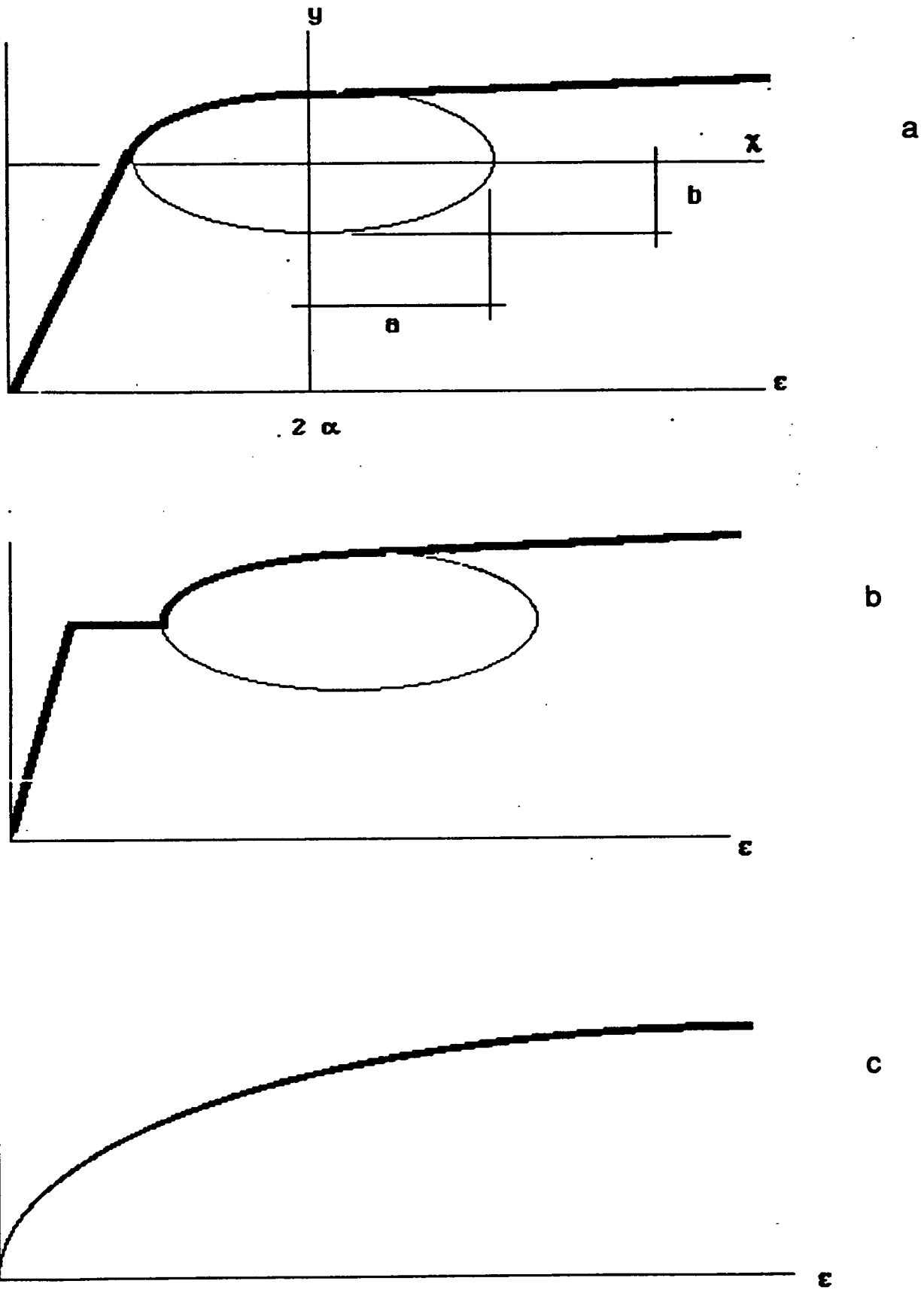


Fig. 4.34 Non-linear stress-strain curve models

The stress-strain model for steel using **Ramberg-Osgood** equation (1943) is given in the following table (Table 4.1):

Table 4.1

Temperature range in °C	Yield stress $\sigma_{y\theta}$	Elastic Modulus E_θ	n_θ
$80^\circ\text{C} < \theta \leq 200^\circ\text{C}$	$\sigma_{y20} \times (0.978 - 0.034 \frac{\theta}{350})$	$E_{20} \times [1 - 2.8 (\frac{\theta - 20}{1485})^2]$	$\frac{4600}{\theta} + \alpha$
$200^\circ\text{C} < \theta \leq 400^\circ\text{C}$	$\sigma_{y20} \times (0.978 - 0.034 \frac{\theta}{350})$	$E_{20} \times [1 - 2.8 (\frac{\theta - 20}{1485})^2]$	$\frac{2650}{\theta} + \alpha$
$400^\circ\text{C} < \theta \leq 550^\circ\text{C}$	$\sigma_{y20} \times (1.553 - 0.155 \frac{\theta}{100})$	$E_{20} \times [1 - 2.8 (\frac{\theta - 20}{1485})^2]$	$\frac{2400}{\theta} + \alpha$
$550^\circ\text{C} < \theta \leq 600^\circ\text{C}$	$\sigma_{y20} \times (2.340 - 0.220 \frac{\theta}{70})$	$E_{20} \times [1 - 3.0 (\frac{\theta - 20}{1463})^2]$	$\frac{3900}{\theta} + \alpha$
$600^\circ\text{C} < \theta \leq 690^\circ\text{C}$	$\sigma_{y20} \times (1.374 - 0.078 \frac{\theta}{50})$	$E_{20} \times [1 - 3.0 (\frac{\theta - 20}{1463})^2]$	$\frac{3600}{\theta} + \alpha$
$690^\circ\text{C} < \theta \leq 800^\circ\text{C}$	$\sigma_{y20} \times (1.120 - 0.128 \frac{\theta}{100})$	$E_{20} \times [1 - 3.0 (\frac{\theta - 20}{1463})^2]$	$\frac{4600}{\theta} + \alpha$

$$\text{where: } \alpha = \frac{\theta}{500 L_n(\frac{\theta}{1750})} \quad \theta = \text{Temperature in } ^\circ\text{C} \quad (4.7)$$

$$\epsilon = \frac{\sigma}{E_{20}} + \frac{3}{7} \left[\frac{\sigma_{y20}}{E_{20}} \right] \left[\frac{\sigma}{\sigma_{y20}} \right]^{50} \quad \text{for } 20^\circ\text{C} \leq \theta \leq 80^\circ\text{C} \quad (4.8)$$

$$\epsilon = \frac{\sigma}{E_\theta} + 0.01 \left[\frac{\sigma}{\sigma_{y\theta}} \right]^{n_\theta} \quad \text{for } 80^\circ\text{C} < \theta \leq 800^\circ\text{C}$$

[Anderberg, 1986]

Magnusson (1974) modified Ramberg and Osgood expression as follows:

$$\epsilon_\sigma = \frac{\sigma}{E(T)} + \frac{3}{7} \left[\frac{f_{0.2T}}{E(T)} \right] \left[\frac{\sigma}{f_{0.2T}} \right]^{mT} \quad (4.9)$$

where: $6 \leq m(T) \leq 50$, $m(T)$ = temperature dependent factor,

$E(T)$ = modulus of elasticity,

$f_{0.2T}$ = 0.2% proof stress at temperature T.

[Anderberg, 1986]

Dounas and Golrang (1982) developed the stress-strain relationship which is described in Figure 4.33. The analytical expression for structural and reinforcing steel is given below:

$$\begin{aligned} \sigma &= \varepsilon_{\sigma} E(T) & 0 \leq \varepsilon_{\sigma} \leq \varepsilon_1 & \quad (4.10) \\ \sigma &= 2\beta + b \sqrt{1 - (2\alpha - \varepsilon_{\sigma} / \alpha)^2} & \varepsilon_1 \leq \varepsilon_{\sigma} \leq 2\alpha \\ \sigma &= b + \frac{2\beta(\varepsilon_{\sigma} - 0.03)}{0.0123 - 0.00085T} & \varepsilon_{\sigma} \leq 2\alpha \end{aligned}$$

The parameters $\alpha, \beta, \varepsilon_1$ are dependent on temperature level and type of steel.

The analytical expression for reinforcing steel is different from the above .

Comments

Most of the existing finite element programs use bi-linear idealization. Making idealizations is not an easy task especially if the elastic and plastic domains are to be idealized bi-linearly. A bi-linear model is one that uses a straight line to represent the elastic modulus and another to represent the plastic modulus. Figure 4.33 shows two different bi-linear idealizations. The " σ - ε " model in Figure 4.33a is appropriate for strains less than $\varepsilon\%$ and will overestimate the stress at higher strains. The " σ - ε " model in Figure 4.33b overestimates the stress at the knee and it will require iterations if the load steps are not small. The choice of a bi-linear curve is easier if the strain range is limited. Multilinear idealizations (Fig. 4.33c) are easier to be made.

Existing Computer Programs

FRANSEEN [1987], author of CEFICOSS, uses a bi-linear model (Fig.4.33b) In this particular model one can distinguish two linear parts, the slope of which can be defined by the modulus of elasticity $E_0(T)$ and the plastic modulus $E^*(T)$ respectively. Knowledge of the variation of the parameters E_0, E^*, σ_y provides

the definition of the relationship between σ and ϵ_{σ} for every temperature. The same model had already been used in the program " Fires R-C" developed by Bizri. During the early stages of his research, FRANSEEN had used the perfect elastic-plastic model (broadly used at ambient temperatures), which did not satisfied him and he comments in his research thesis that it is a very gross model for the idealization of the " σ - ϵ " curve at elevated temperatures. He also used a non-linear idealization in his program which he finally abandoned for the following reasons:

- using the non-linear model, introduces difficulty in comparing the theoretical curves with these resulting from testing and calibrating the parameters which define the law.

- the numerical error using the bi-linear rather than non-linear is not so important.

- the results from the analytical modelling as a whole do not present a significant amelioration when the non-linear model is used.

FORSEN [Anderberg, 1976] in Steelfire - Finite Element program for Nonlinear Analysis of Steel Frames exposed to Fire (1983) - uses a " σ - ϵ " curve approximated by piecewise linear lines.

Codes

The European Recommendations for the Fire Safety of Steel Structures by ECCS - Technical Committee 3 [ECCS, 1983] in order to apply elementary plastic theory introduce the concept of effective yield stress. It is defined as the level of stress for which the stress-strain curves are cut-off (Fig.4.35) and it decreases at increasing temperature.

For steel grade Fe360, the stress-strain curves are given in Fig. 4.36 assuming, for practical purposes, that the heating rate does not significantly influence the

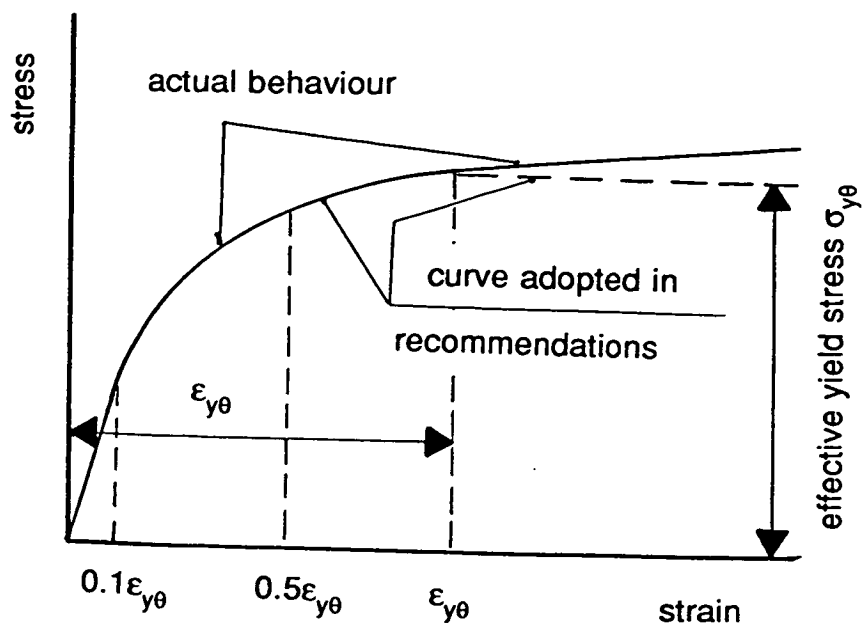


Fig. 4.35 Simplification of the stress-strain curves of steel at elevated temperatures (ECCS, 1983).

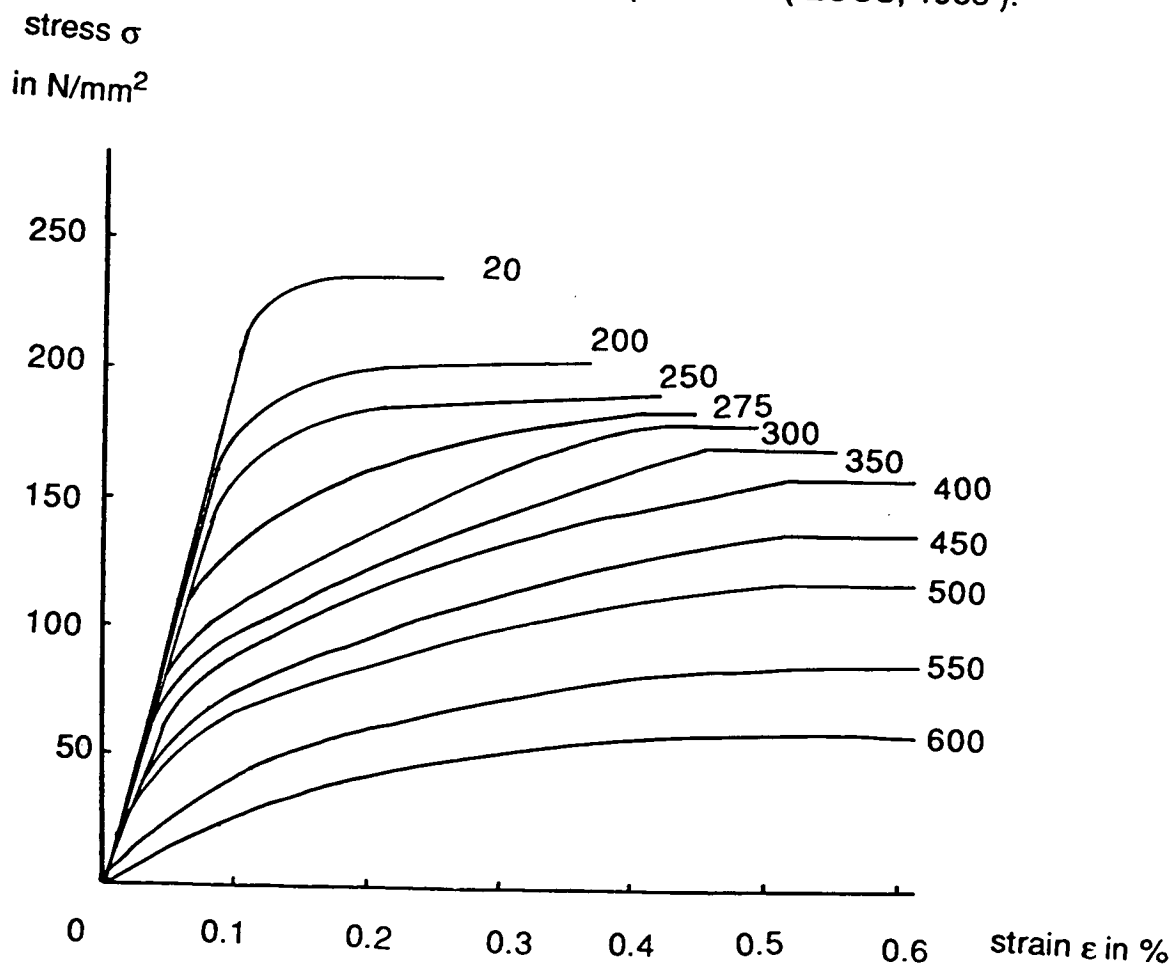


Fig. 4.36 Stress-strain relationships at elevated temperatures for steel grade Fe 360 (ECCS, 1983).

deformation behaviour of steel structures under fire conditions - the creep is included in an implicit way.

Stress-strain relationships for other grades of steel can be obtained through recommended transformation based on the following assumptions:

a/ For a given grade of steel the reduction in the effective yield stress at a temperature θ_s is assumed to be the same as for Fe360. That means $\varphi = \frac{\sigma_{y\theta}}{\sigma_{y20}}$ for

all grades of steel.

b/ For any grade of steel, the modulus of elasticity $E_{t\theta}$ at zero stress is assumed to be equal to that of Fe360 throughout the temperature range.

The recommended transformation is illustrated in Figure 4.37 and the analytical expression is given below:

$$\sigma = \sigma^* + \varphi (\sigma_{y20} - 235) \quad [\text{N/mm}^2] \quad (4.11)$$

$$\varepsilon = \varepsilon^* + \left[\varphi \frac{\sigma_{y20} - 235}{E_{t\theta}} \right] \times 100 \quad [\text{N/mm}^2]$$

where:	σ	=actual stress
	σ^*	=stress parameter applying to Fe360
	$\varphi = \frac{\sigma_{y\theta}}{\sigma_{y20}}$	=temperature dependent coefficient
	σ_{y20}	=nominal used stress of the used steel at room temperature as specified in Euronorm 25-72
	$\sigma_{y\theta}$	=effective yield stress of steel at elevated temperature
	ε	=actual strain
	ε^*	=strain parameter applying to Fe360
	$E_{t\theta}$	=temperature dependent modulus of elasticity for $\sigma=0$

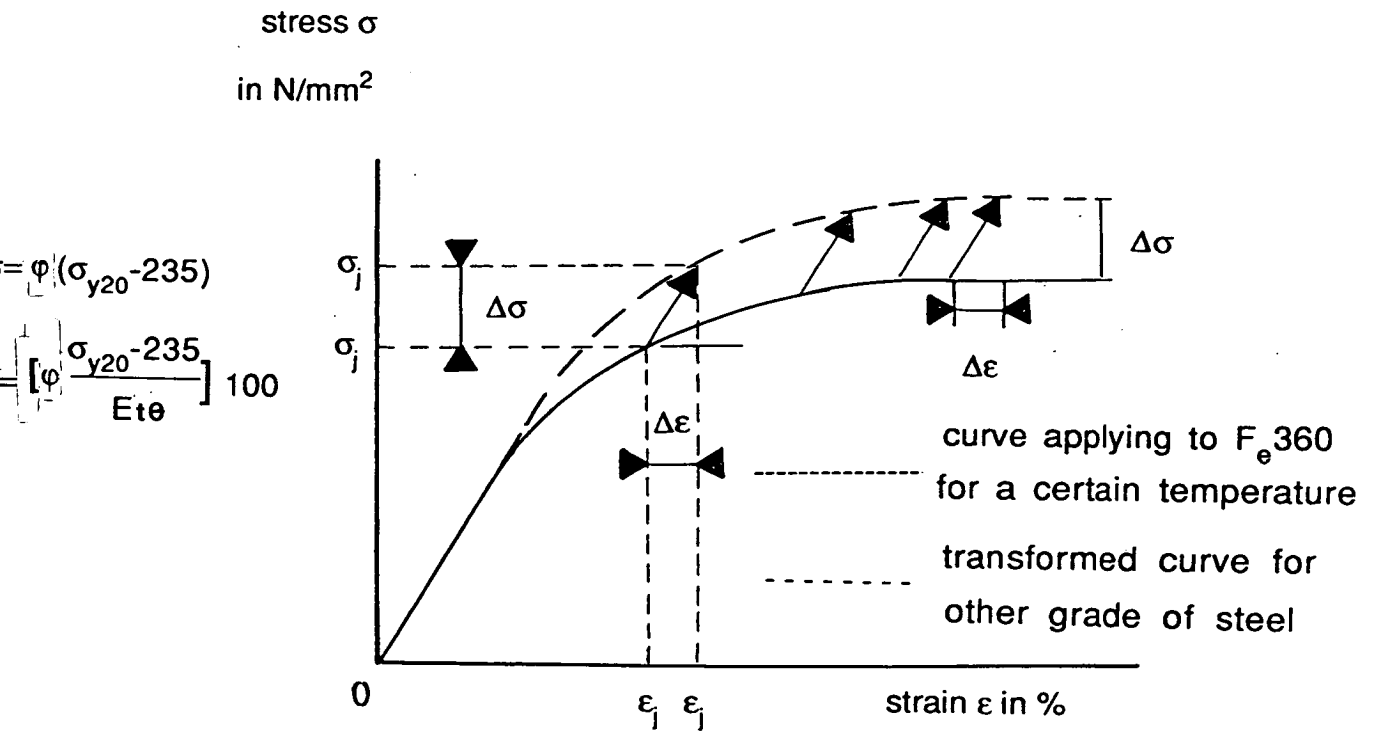


Fig. 4.37 Transformation of the stress-strain curves for Fe 360 to other grades of steel (schematic) (ECCS, 1983).

Tabulated values of the recommended stress-strain relations are given in the ECCS Document for steel grades Fe310, Fe360, Fe430, Fe 510.

BS5950: Part 8 uses British Steel data which correlates better with the large scale beam and column tests, both in terms of the heating rates experienced and also the strains developed.

The British Steel data is consistent with a heating rate of 10°C/minute. Similar data can be obtained for other heating rates. It can be shown that the faster the rate of heating, the higher the temperature at which a particular steel strength is attained, for a given strain.

Full data is presented for strains up to 2% (Tables 4.2a,b). In BS 5950: Part 8, Table 1 data is only given for 0.5%,1.5%,2.0% strains because according to experimental tests:

- a strain limit of 2.0% has been selected as being representative for the design of composite members in bending protected with fire protection materials which have demonstrated their ability to remain intact at this level of strain.
- a strain limit of 1.5% has been selected for the design of non-composite members in bending which are unprotected or protected with fire protection materials which have demonstrated their ability to remain intact.
- a strain limit of 0.5% applies for any other member (including columns).

The above strains should not be exceeded unless it has been demonstrated in fire resistance tests that a higher level of strain can be satisfactorily developed in the steel and that the fire protection material has the ability to remain intact.

British Steel data only covers the steel grades 43 and 50 complying with BS4360. Data for cold finished steels complying with BS2989 is given in Appendix B of the same document. Data for other grades of steel should be established on the basis of elevated temperature tensile tests.

Table 4.2a

Elevated temperature stress/strain data for BS4360: Grade 43A:1979 structural steels derived from transient tests (heating rate = 10 °C/min)

Strain %	Stress in N/mm2 for Various Temperatures, Deg. C																		
	20	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	*
0.01	18.4	18.4	18.4	17.3	16.6	15.8	15.6	14.5	13.3	11.7	9.4	6.9	5.6	4.1	2.0	2.0	1.8	*	*
0.02	36.7	36.7	35.7	34.9	33.2	31.9	31.4	29.1	26.8	23.5	19.1	13.5	11.5	8.2	4.1	3.8	3.3	*	*
0.03	55.1	54.1	54.1	53.3	49.7	47.7	46.9	43.3	40.0	35.2	28.6	20.4	17.1	12.2	6.1	5.9	5.6	*	*
0.04	73.4	72.4	71.7	70.6	66.3	63.5	62.5	57.9	53.3	46.9	38.3	27.0	22.7	16.6	8.2	7.9	7.4	*	*
0.05	91.8	89.3	90.0	88.2	82.9	79.3	78.0	72.4	66.8	58.6	47.7	33.9	28.3	20.7	10.5	9.9	9.4	*	*
0.06	110.2	109.1	107.4	105.6	99.2	95.4	93.8	87.0	80.1	70.1	57.1	40.8	34.2	24.7	12.5	11.7	11.2	*	*
0.07	128.3	126.7	125.7	123.2	115.8	111.2	109.4	101.2	93.3	81.9	66.8	47.4	39.8	28.8	14.5	13.8	13.0	*	*
0.08	146.6	144.8	143.3	141.5	132.3	127.0	124.9	115.8	106.6	93.6	76.2	54.3	45.1	32.9	16.6	15.3	14.0	*	*
0.09	165.0	163.2	161.7	158.9	148.9	143.1	132.1	124.4	116.8	105.3	85.7	60.9	48.4	36.0	18.6	16.3	14.0	*	*
0.10	183.3	181.6	179.0	176.5	165.5	158.9	136.4	129.0	121.6	111.7	95.4	67.8	51.3	38.0	20.7	17.3	14.0	*	*
0.12	220.1	217.5	214.7	211.4	198.6	181.8	144.6	138.0	131.6	118.3	102.3	81.3	56.1	41.3	24.7	19.4	14.3	*	*
0.14	255.0	247.1	234.1	225.4	208.1	188.2	152.2	145.9	139.5	124.7	108.6	87.2	60.7	43.9	26.8	20.4	14.3	*	*
0.16	255.0	247.1	238.4	229.5	213.4	193.8	158.9	152.2	145.9	130.8	113.5	92.6	64.3	46.2	28.1	21.4	14.5	*	*
0.18	255.0	247.1	242.3	232.6	217.5	198.4	164.5	157.8	151.2	135.9	118.6	96.9	67.6	48.2	29.6	22.2	14.8	*	*
0.20	255.0	247.1	244.8	234.6	221.1	202.2	169.8	163.2	156.3	140.3	122.9	100.0	70.9	50.0	31.1	23.0	15.0	*	*
0.25	255.0	247.1	246.1	237.7	229.2	208.8	181.3	174.7	167.8	150.2	132.3	105.8	77.3	54.3	34.9	25.0	15.6	*	*
0.30	255.0	247.1	246.1	239.7	233.8	213.9	191.8	184.9	177.7	158.9	140.3	110.7	82.9	58.4	38.3	27.0	16.1	*	*
0.35	255.0	247.1	246.1	241.0	237.4	217.3	199.7	192.8	185.9	166.5	146.4	115.5	87.5	62.5	41.1	28.6	16.6	*	*
0.40	255.0	247.1	246.1	241.7	239.2	219.8	207.1	199.7	192.3	172.9	151.5	119.3	91.3	65.3	43.6	30.1	17.1	*	*
0.50	255.0	247.1	246.1	243.8	241.2	225.4	217.8	210.6	203.5	183.9	158.6	125.5	96.4	68.6	47.4	32.4	18.1	*	*
0.60	255.0	247.1	246.1	244.0	241.7	230.0	225.7	218.5	211.4	192.5	165.5	131.8	100.5	70.9	49.5	33.4	19.1	12.8	*
0.70	255.0	247.1	246.1	244.3	242.3	233.6	230.8	225.2	219.6	199.9	172.4	137.4	103.8	72.9	50.7	34.2	20.1	13.8	*
0.80	255.0	247.1	246.1	244.5	242.8	237.4	235.4	231.5	227.5	206.0	177.2	142.0	107.4	75.0	51.8	34.9	21.2	14.5	*
0.90	255.0	247.1	246.1	244.5	243.0	240.7	238.7	235.6	232.8	210.9	181.3	145.9	110.2	76.5	52.8	35.4	22.2	15.3	*
1.00	255.0	247.1	246.1	244.8	243.5	242.8	241.2	239.7	236.9	214.7	184.1	148.9	111.7	77.8	53.5	36.2	23.2	16.1	*
1.20	*	*	*	*	*	*	245.6	243.8	240.2	221.3	188.2	152.5	114.2	80.1	55.1	37.2	25.2	17.3	*
1.40	*	*	*	*	*	*	*	245.8	242.8	226.4	191.3	155.0	116.3	82.1	56.4	38.3	27.0	18.4	14.8
1.60	*	*	*	*	*	*	*	*	244.8	231.3	193.8	157.1	118.1	83.9	57.4	39.0	28.1	19.1	15.3
1.80	*	*	*	*	*	*	*	*	246.3	235.4	196.1	158.6	119.6	85.2	58.4	39.8	28.8	19.9	15.6
2.00	*	*	*	*	*	*	*	*	*	238.2	197.9	159.9	120.9	85.9	59.2	40.3	29.3	20.1	15.8

*No data.

Table 4.2b

Elevated temperature stress/strain data for BS4360: Grade 50B:1979 structural steels derived from transient tests (heating rate = 10 °C/min)

Strain %	Stress in N/mm2 for Various Temperatures, Deg. C																				
	20	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900		
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	*	*		
0.01	18.8	18.8	18.8	17.8	16.3	15.6	15.6	14.6	13.1	11.7	9.6	6.7	5.7	4.3	2.1	2.1	1.8	*	*		
0.02	37.6	37.6	36.6	35.9	33.0	31.6	31.2	28.8	26.6	23.4	18.8	13.5	11.4	8.2	4.3	3.9	3.9	*	*		
0.03	56.1	55.4	55.4	54.3	49.3	47.2	46.5	43.3	39.8	35.1	28.4	20.2	17.0	12.4	6.0	6.0	5.7	*	*		
0.04	74.9	74.2	73.1	72.4	65.7	63.2	62.1	57.5	53.3	46.5	38.0	27.0	22.4	16.3	8.2	7.8	7.5	*	*		
0.05	93.7	92.7	91.9	90.2	82.4	78.8	77.7	72.1	66.4	58.2	47.6	33.7	28.0	20.6	10.3	9.9	9.2	*	*		
0.06	112.5	111.5	109.7	107.9	98.7	94.8	93.4	86.6	79.5	69.9	56.8	40.5	33.7	24.5	12.4	11.7	11.4	*	*		
0.07	131.0	129.2	128.5	125.7	115.0	110.4	109.0	100.8	93.0	81.7	66.4	47.2	39.4	28.8	14.2	13.8	13.1	*	*		
0.08	149.8	148.0	146.3	144.5	131.7	126.4	124.3	115.4	106.1	93.4	76.0	54.0	45.1	32.7	16.3	15.6	14.9	*	*		
0.09	168.6	166.9	165.1	162.2	148.0	142.0	139.9	129.6	119.3	105.1	85.2	60.7	50.8	36.9	18.5	17.8	16.7	*	*		
0.10	187.4	185.7	182.8	180.3	164.4	158.0	155.1	143.8	132.4	116.4	94.8	67.4	56.4	40.8	20.6	19.5	18.5	*	*		
0.12	224.7	222.2	219.4	215.8	197.4	189.6	183.2	171.1	159.0	139.5	113.6	80.9	66.0	49.0	24.5	22.0	19.9	*	*		
0.14	262.3	259.5	256.0	252.4	230.4	221.2	198.8	188.5	177.9	158.0	132.8	94.4	74.2	53.6	28.8	24.5	19.9	*	*		
0.16	299.6	296.1	292.5	288.3	263.4	251.0	211.6	201.6	191.3	170.0	145.2	107.9	81.7	58.2	33.0	26.6	20.2	*	*		
0.18	337.3	333.7	328.4	313.8	287.9	264.8	221.9	212.6	203.1	180.3	156.2	120.3	88.0	62.5	36.6	28.4	20.6	*	*		
0.20	355.0	345.1	333.7	318.8	300.7	273.0	231.8	222.2	213.0	188.5	165.8	130.6	94.1	66.0	39.8	30.2	20.9	*	*		
0.25	355.0	348.6	338.7	326.6	316.3	287.2	250.6	241.8	232.5	206.3	182.8	145.6	105.1	74.5	46.2	33.7	21.7	*	*		
0.30	355.0	349.7	341.2	332.6	325.2	296.8	267.0	257.0	247.4	220.5	195.3	154.1	115.0	81.3	51.5	36.6	22.4	*	*		
0.35	355.0	350.0	342.6	336.5	330.5	302.5	278.0	268.4	258.8	231.8	203.8	160.8	121.8	87.0	57.2	39.8	23.1	*	*		
0.40	355.0	350.0	344.0	338.7	333.0	306.0	288.3	278.0	267.7	240.7	210.9	166.1	127.1	90.9	60.7	41.9	23.8	*	*		
0.50	355.0	350.4	345.4	340.8	335.8	313.8	303.2	293.2	283.3	256.0	220.8	174.7	134.2	95.5	66.0	45.1	25.2	*	*		
0.60	355.0	350.4	345.8	341.2	336.5	320.2	314.2	304.2	294.3	268.0	230.4	183.5	139.9	98.7	68.9	46.5	26.6	17.8	*	*	
0.70	355.0	350.7	346.1	341.9	337.3	325.2	321.3	313.5	305.7	278.3	240.0	191.3	144.5	101.5	70.6	47.6	28.0	19.2	*	*	
0.80	355.0	350.7	346.5	342.2	338.0	330.5	327.7	322.3	316.7	286.8	246.7	197.7	149.5	104.4	72.1	48.6	29.5	20.2	*	*	
0.90	355.0	350.7	346.8	342.6	338.3	335.1	332.3	328.0	324.1	293.6	252.4	203.1	153.4	106.5	73.5	49.3	30.9	21.3	*	*	
1.00	355.0	351.1	347.2	343.3	339.0	338.0	335.8	333.7	329.8	298.9	256.3	207.3	155.5	108.3	74.5	50.4	32.3	22.4	*	*	
1.20	*	*	*	*	*	*	341.9	339.4	334.4	308.1	262.0	212.3	159.0	111.5	76.7	51.8	35.1	24.1	*	*	
1.40	*	*	*	*	*	*	*	342.2	338.0	315.2	266.3	215.8	161.9	114.3	78.5	53.3	37.6	25.6	20.6	*	*
1.60	*	*	*	*	*	*	*	*	340.8	322.0	269.8	218.7	164.4	116.8	79.9	54.3	39.0	26.6	21.3	*	*
1.80	*	*	*	*	*	*	*	*	342.9	327.7	273.0	220.8	166.5	118.6	81.3	55.4	40.1	27.7	21.7	*	*
2.00	*	*	*	*	*	*	*	*	*	331.6	275.5	222.6	168.3	119.6	82.4	56.1	40.8	28.0	22.0	*	*

*No data.

ECCS data [ECCS, 1983] is conservative relative to the anisothermal data produced by British Steel. They use an effective yield strain of 0.5% for temperatures above 400°C even though in fire tests on beams and columns it can be shown that much higher strains are experienced. For the above reason, British Steel data replaced that of ECCS in the proposed Eurocode 3 [Eurocode No. 3, 1990].

The model for stress-strain relationships of steel at elevated temperatures in the proposed Eurocode 3 is given in Figure 4.39.

where:

$E_{a(\theta)}$	slope of the linear elastic range of the stress-strain relationship of steel in the fire situation.
$\sigma_{amax(\theta)}$	effective yield stress of steel in the fire situation
$\sigma_{apr(\theta)}$	proportional limit of steel in the fire situation
$\epsilon_{amax(\theta)}$	strain corresponding to the effective yield stress of steel in the fire situation
$\epsilon_{apr(\theta)}$	strain corresponding to the proportional limit of steel in the fire situation
subscript a	means steel

According to this proposed model and for heating rates 2-50°C/min, we can distinguish three regions which are : elastic, non-linear, plastic (Fig.4.38). The stress-strain curves can be described by the slope of linear elastic range (E_{af}), the proportional limit (f_{pf}) the effective yield stress (f_{yf}). The variation of the characteristic values of these parameters related to their temperature values are given in Figure 4.38.

The stress-strain relationships in British Standards and Eurocode 3 are based on British Steel data which have been derived from small scale tensile tests. These relationships may also be applied to steel in compression.

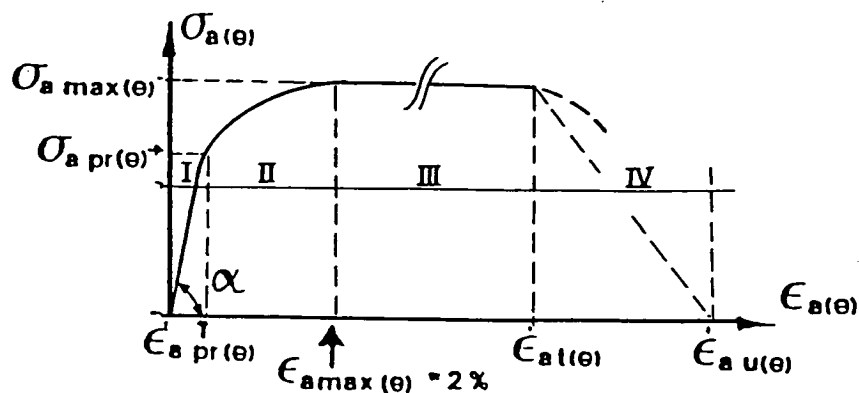


Fig. 4.38 Mathematical model for stress-strain relationships of steel at elevated temperatures (Eurocode 4, 1990).

Range	$\sigma_{a(\theta)} =$	$\bar{E}_{a(\theta)} =$
I elastic	$\bar{E}_{a(\theta)} \cdot \epsilon_{a(\theta)}$	$E_{a(\theta)}$
II transit elliptical	$\frac{b}{a} \sqrt{a^2 - (\epsilon_{a \max(\theta)} - \epsilon_{a(\theta)})^2} + \sigma_{a \text{ pr}(\theta)} - c$ <p>with</p> $a^2 = \frac{E_{a(\theta)} (\epsilon_{a \max(\theta)} - \epsilon_{a \text{ pr}(\theta)})^2 + c (\epsilon_{a \max(\theta)} - \epsilon_{a \text{ pr}(\theta)})}{E_{a(\theta)}}$ $b^2 = E_{a(\theta)} (\epsilon_{a \max(\theta)} - \epsilon_{a \text{ pr}(\theta)}) \cdot c + c^2$ $c = \frac{(\sigma_{a \max(\theta)} - \sigma_{a \text{ pr}(\theta)})^2}{2(\sigma_{a \text{ pr}(\theta)} - \sigma_{a \max(\theta)}) + E_{a(\theta)} (\epsilon_{a \max(\theta)} - \epsilon_{a \text{ pr}(\theta)})}$	$\frac{b (\epsilon_{a \max(\theta)} - \epsilon_{a(\theta)})}{a \sqrt{a^2 - (\epsilon_{a(\theta)} - \epsilon_{a \max(\theta)})^2}}$
III plastic	$\sigma_{a \max(\theta)}$	0

Fig. 4.39 Definition of the various parameters of the mathematical model of figure 4.38 (Eurocode 4, 1990).

4.4.3 CREEP STRAIN

The models by which the mechanical properties of steel at elevated temperatures are described can basically be distinguished in sophisticated and pragmatic models. Sophisticated models include creep explicitly. Pragmatic models present the strength and deformation properties by a set of temperature-dependent stress-strain relationships and creep is included in an implicit way.

Models of creep are in most cases based on the theory put forward by Dorn (1954)[Thor, 1973] which permits consideration of the variation of temperature with time.

The creep strain to be modelled here is based on Dorn's theory - i.e it is dependent on the magnitude of the stress and on the temperature-compensated time given by the following relation:

$$\theta = \int_0^t e^{\frac{-\Delta H}{RT}} dt \quad (4.12)$$

where: ΔH =activation energy required for creep (cal/mol),
 R =universal gas constant (cal/mol K),
 T =temperature (K),
 t =time (hours).

The relation between the creep strain ϵ_{cr} and the temperature-compensated time θ (t), for different stresses σ is shown in Figure 4.41. The curve expressing this relation contains a portion of constant slope ($d\epsilon_{cr}/d\theta=Z$) which is dependent only on the magnitude of stress and defines the secondary phase of creep. The primary phase is defined by a parabolic equation. The transfer from the primary to the secondary phase occurs at time θ_0 and the intersection between the straight line and the creep strain axis is called ϵ_{cr0} .

Steel Temperature (°C)	$\frac{\bar{E}_a(\Theta)}{E_{a(20^\circ\text{C})}}$	$\frac{\sigma_{a\text{pr}}(\Theta)}{f_{ay(20^\circ\text{C})}}$	$\frac{\sigma_{a\text{max}}(\Theta)}{f_{ay(20^\circ\text{C})}}$
20	1,00	1,00	1,00
100	1,00	1,00	1,00
200	0,90	0,807	1,00
300	0,80	0,613	1,00
400	0,70	0,42	1,00
500	0,60	0,36	0,78
600	0,31	0,18	0,47
700	0,13	0,075	0,23
800	0,09	0,05	0,11
900	0,0675	0,0375	0,06
1000	0,045	0,025	0,04
1100	0,0225	0,0125	0,02
1200	0,00	0,00	0,00

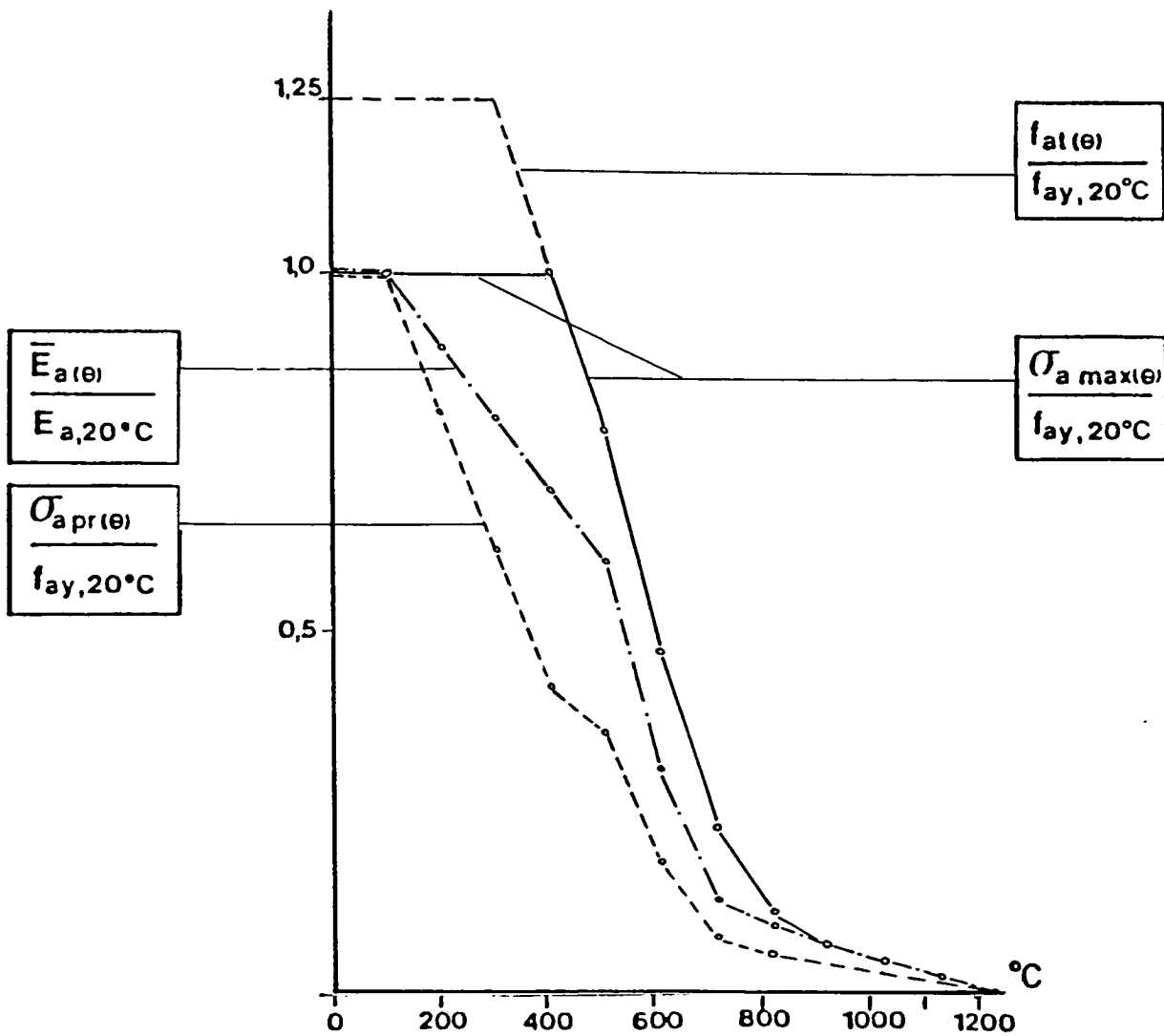
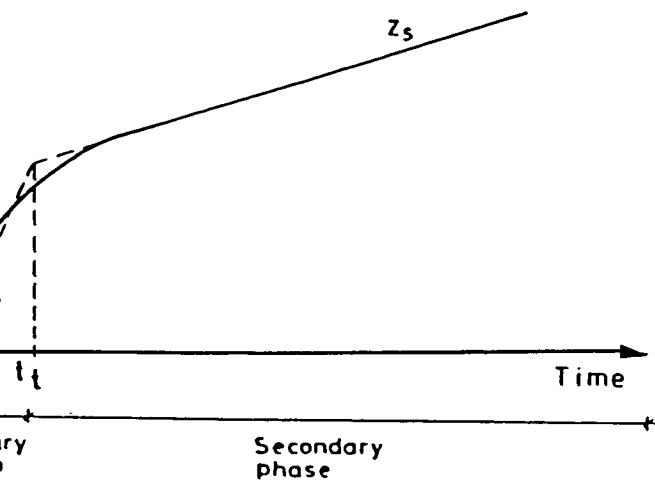
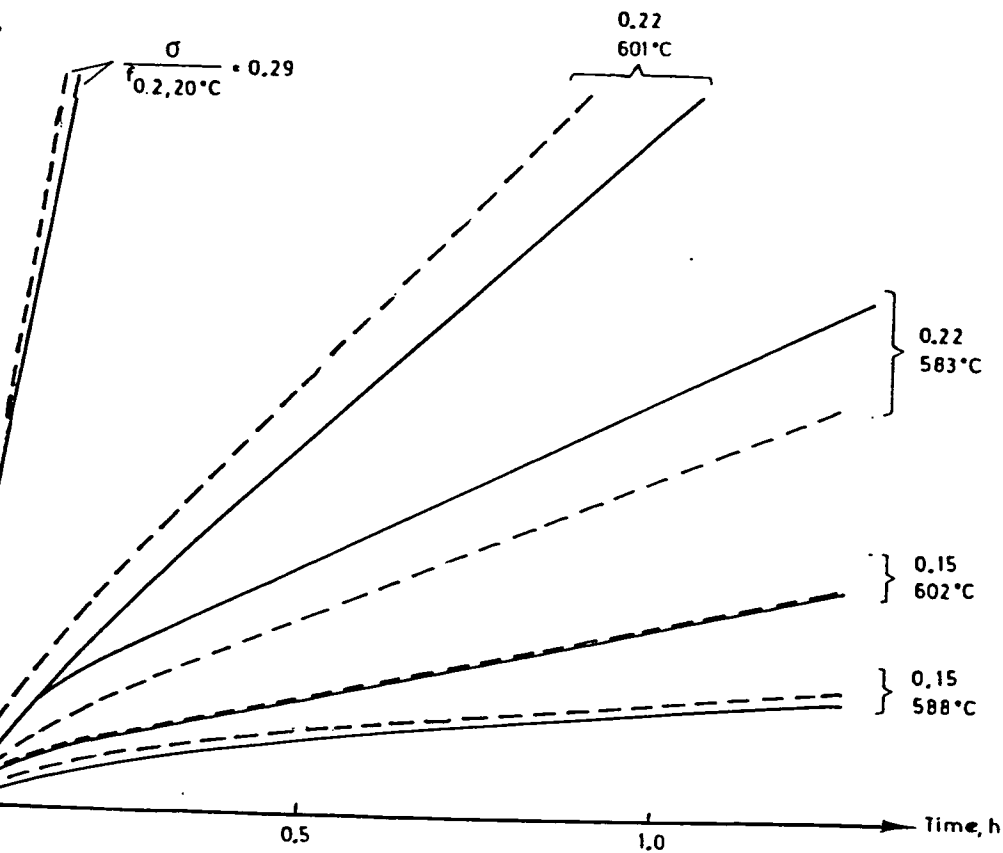


Fig. 4.40 Parameters for stress-strains relationships of structural steel at elevated temperatures (Eurocode 4, 1990).



4.41 Measured creep curve approximated by two straight branches with slopes Z_p, Z_s (Anderberg, 1986).



4.42 Measured and predicted creep (modification Dorn-Harmathy theory) at different stress levels. Reinforcing steel $K_s 60 \phi 8$, $f_{0.2 20^\circ C} = 710 \text{ MPa}$ (Anderberg, 1983)

Yngve Anderberg [Anderberg, 1986] proposes the following mathematical formula for the evaluation of creep strain:

$$\begin{aligned} \epsilon_{cr} &= \epsilon_{cr0} \left(2 \sqrt{Z \frac{\theta}{\epsilon_{cr0}}} \right) & 0 \leq \theta \leq \theta_0 & \quad (4.13) \\ \epsilon_{cr} &= \epsilon_{cr0} \left(1 + \left(Z \frac{\theta}{\epsilon_{cr0}} \right) \right) & \theta \geq \theta_0 & \end{aligned}$$

$$\theta_0 = \frac{\epsilon_{cr0}}{Z} \quad (4.14)$$

Harmathy (1967) has derived the following analytical expression :

$$\epsilon_{cr} = \frac{\epsilon_{cr0}}{\ln 2} \operatorname{arcosh} \left(2^{Z\theta/\epsilon_{cr0}} \right) \quad (4.15)$$

This relationship is not so practical and according to Yngve Anderberg [Anderberg, 1986] is unnecessarily complicated. It has been used by Jorgen Thor [Thor, 1973] for the evaluation of deformations and critical loads of steel beams under fire exposure conditions.

The following equations are mostly used to evaluate the parameters ϵ_{cr0} , Z :

$$\begin{aligned} \epsilon_{cr0} &= A \sigma^B \\ & C \sigma^D \quad \text{If } \sigma \leq \text{SIG } 1 & \quad (4.16) \\ Z &= \\ & H e^{F \sigma} \quad \text{If } \sigma > \text{SIG } 1 \end{aligned}$$

The coefficients in the above equations are given in Table 4.3 for steels accounted for in the following three sources: 1 Thor (1972) 2. Harmathy and Stanzak (1970) 3. Anderberg (1978).

The creep parameters $\Delta H/R$, Z , ϵ_{cr0} can be determined from creep tests under constant temperature and stress.

Jorgen Thor (1972) produced Table 4.4 which lists the values of $\Delta H/R$ and the relation between the stress and Z and stress and ϵ_{t0} (ϵ_{cr0}) for all steels. The

Table 4.3

Creep parameters for different kind of steels (1) Thor (1972), (2) Harmathy-Stanzak (1970) and (3) Anderberg (1978)

Stål	$f_{0.2, 20^{\circ}\text{C}}$ MPa	A	B	C min^{-1}	D	H min^{-1}	F	$\frac{\Delta H}{R}$ K	SIG 1 MPa	
(1) {	1312 (test 1)	254	$5.56 \cdot 10^{-6}$	1.722	$6.083 \cdot 10^{+9}$	7.808	$1.383 \cdot 10^{+23}$	0.0578	55 800	108
	1312 (test 2)	263	$2.66 \cdot 10^{-7}$	2.248	$8.95 \cdot 10^{+8}$	7.644	$5 \cdot 10^{+21}$	0.0601	53 900	108
	1411 (Thor)	340	$3.52 \cdot 10^{-7}$	2.08	$6.767 \cdot 10^{+12}$	8.402	$4.417 \cdot 10^{+27}$	0.0603	66 000	118
	A36-66	304	$4.07 \cdot 10^{-6}$	1.75	$6.217 \cdot 10^{+6}$	4.70	$2 \cdot 10^{+14}$	0.0434	38 900	103
	2172	331	$2.085 \cdot 10^{-8}$	2.30	$1.33 \cdot 10^{+10}$	5.38	$1.083 \cdot 10^{+19}$	0.0446	50 000	108
	G40-12	333	$1.766 \cdot 10^{-7}$	1.00	$4.733 \cdot 10^{+7}$	3.25	$6.17 \cdot 10^{+12}$	0.0319	36 100	103
(2)	A421-65	1470	$9.262 \cdot 10^{-5}$	0.67	$3.253 \cdot 10^{+6}$	3.0	$1.368 \cdot 10^{+12}$	0.0145	30 600	172
(3) {	Ks 40 \emptyset 10	483	$28.5 \cdot 10^{-9}$	1.037	$1.16 \cdot 10^{+9}$	4.7	$4.3 \cdot 10^{+16}$	0.0443	45 000	84
	Ks 40 \emptyset 8	456	$3.39 \cdot 10^{-7}$	0.531	$7.6 \cdot 10^{+5}$	4.72	$1.25 \cdot 10^{+13}$	0.0512	40 000	90
	Ks 40 \emptyset 8	504	$19.9 \cdot 10^{-6}$	1.28	$4.05 \cdot 10^{+4}$	7.26	$5.0 \cdot 10^{+17}$	0.0384	47 000	120
	Ks 40 SE \emptyset 8	558	$38.6 \cdot 10^{-9}$	1.117	$5.8 \cdot 10^{+7}$	3.83	$4.133 \cdot 10^{+13}$	0.0414	40 000	96
	Ks 60 \emptyset 8	710	$2.06 \cdot 10^{-6}$	0.439	$5.111 \cdot 10^{+7}$	2.93	$2.65 \cdot 10^{+14}$	0.0313	40 000	90
	Ps 50 \emptyset 5	500	$1.10 \cdot 10^{-6}$	0.557	$9.738 \cdot 10^{+6}$	4.47	$2.133 \cdot 10^{+15}$	0.0368	40 000	100
	Ps 50 \emptyset 8	749	$1.28 \cdot 10^{-7}$	0.844	$1.367 \cdot 10^{+8}$	3.94	$1.02 \cdot 10^{+15}$		41 000	133

Table 4.4

Steel group	Steel	Yield stress at room temperature (kgf/cm ²)	Analysis (%)													$\Delta H/R$ (h ⁻¹)	Z (h ⁻¹)	ϵ_{to}
			C	Si	Mn	P	S	Cr	Cu	N	Al	Nb	Ni	Mo				
Carbon steels	1312 (Test 1)	2590	.11	.05	.57	.025	.036	.028	.006	.008	-	-	-	-	55800	$f_{0r} \begin{cases} \sigma \leq 1100 & 4.89 \cdot 10^3 \cdot \sigma^{7.808} \\ \sigma > 1100 & 8.3 \cdot 10^{24} \cdot e^{-0.00547 \cdot \sigma} \end{cases}$	$1.02 \cdot 10^{-7} \cdot \sigma^{1.722}$	
	1312 (Test 2)	2680	.13	.07	.75	.010	.026	.035	.052	.005	-	-	-	53900	$f_{0r} \begin{cases} \sigma \leq 1100 & 10.53 \cdot 10^2 \cdot \sigma^{7.644} \\ \sigma > 1100 & 3.0 \cdot 10^{23} \cdot e^{-0.00590 \cdot \sigma} \end{cases}$	$1.44 \cdot 10^{-8} \cdot \sigma^{2.247}$		
	1411	3400	.18	.13	.75	.035	.020	.020	.015	.013	-	-	-	66000	$f_{0r} \begin{cases} \sigma \leq 1200 & 1.37 \cdot 10^6 \cdot \sigma^{8.4919} \\ \sigma > 1200 & 1.45 \cdot 10^{29} \cdot e^{-0.00492 \cdot \sigma} \end{cases}$	$2.82 \cdot 10^{-8} \cdot \sigma^{2.09}$		
	A36-66	3100	.18	.09	.71	.007	.030	-	-	-	-	-	-	38900	$f_{0r} \begin{cases} \sigma \leq 1050 & 6.80 \cdot 10^3 \cdot \sigma^{4.70} \\ \sigma > 1050 & 1.2 \cdot 10^{16} \cdot e^{-0.00426 \cdot \sigma} \end{cases}$	$7.00 \cdot 10^{-8} \cdot \sigma^{1.75}$		
Carbon-manganese steels	2172	3380	.17	.33	1.57	.023	.020	-	-	.010	-	-	-	50000	$f_{0r} \begin{cases} \sigma \leq 1100 & 3.00 \cdot 10^6 \cdot \sigma^{5.38} \\ \sigma > 1100 & 6.50 \cdot 10^{20} \cdot e^{-0.00439 \cdot \sigma} \end{cases}$	$1.00 \cdot 10^{-10} \cdot \sigma^{2.34}$		
	Q40.12	3400	.18	.02	1.40	.015	.019	.010	.040	-	<.01	-	.03 ppm	36100	$f_{0r} \begin{cases} \sigma \leq 1050 & 1.50 \cdot 10^6 \cdot \sigma^{5.25} \\ \sigma > 1050 & 3.70 \cdot 10^{14} \cdot e^{-0.00313 \cdot \sigma} \end{cases}$	$1.80 \cdot 10^{-8} \cdot \sigma^{1.00}$		
Drain-refined steels	Drain refinement with aluminium	3730	.21	.30	1.48	.017	.019	-	-	.005	.027	-	-	40900	$f_{0r} \begin{cases} \sigma \leq 1300 & 2.10 \cdot 10^{11} \cdot \sigma^{5.78} \\ \sigma > 1300 & 8.00 \cdot 10^{16} \cdot e^{-0.00455 \cdot \sigma} \end{cases}$	$1.48 \cdot 10^{-9} \cdot \sigma^{2.15}$		
	Drain refinement with niobium	4050	.17	.43	1.31	.018	.020	.018	.014	.007	.007	.021	-	45000	$f_{0r} \begin{cases} \sigma \leq 1250 & 5.03 \cdot \sigma^{4.644} \\ \sigma > 1250 & 1.2 \cdot 10^{19} \cdot e^{-0.00434 \cdot \sigma} \end{cases}$	$4.78 \cdot 10^{-9} \cdot \sigma^{1.95}$		

Table also gives the analyses of the various steels and their yield stresses at room temperatures. One can notice from the same table that in some cases large differences occur even when steels that belong to the same group are compared. This is due to the yield stresses which vary even for steels in the same group. Another reason is that the determination of $\Delta H/R$ is sometimes sensitive and may therefore be subject to some inaccuracy. The values of $\Delta H/R$ affect the magnitude of Z derived from the equation:

$$Z = \left(\frac{d\varepsilon_t}{d\theta} \right) = \left(\frac{d\varepsilon_t}{dt} \frac{dt}{d\theta} \right) = \varepsilon_{ts} e^{\Delta H/RT} \quad (4.17)$$

where: ε_{ts} = rates of creep during the secondary creep stages of the conventional creep tests- creep tests at constant temperature and stress.

Since it is not the value of $\Delta H/R$ or Z in themselves but a combination of these, some uncertainty in the value of $\Delta H/R$ can normally be accepted. It is shown that that calculated creep strains at stresses which are the same in relation to the yield stress of the steel concerned at room temperature shows good agreement within the same principal group, in spite of differences in the values of $\Delta H/RT$ and Z . The values of the above parameters are regarded not as exact material data but rather as empirical values and relationships which when used in the equations for the determination of temperature-compensated time and creep strain produce creep curves which are in good agreement with those plotted on the basis of tests.

Anderberg used the Harmathy expression to compare test results with analytical results for reinforcing steel Ks 60 $\phi 8$. (Fig. 4.40).

A simplified way to approximate the curve which illustrates the relationship between the creep strain and the temperature-compensated time, is given in Fig.4.41 which contains two straight branches with slopes Z_p and Z_s . The analytical expression is given below:

$$\varepsilon_{cr}(t, T, \sigma) = t Z(T, \sigma) \quad (4.18)$$

$$\varepsilon_{cr} = Z_p(T, \sigma) \quad \text{if } 0 \leq t \leq t_t$$

$$\varepsilon_{cr} = Z_s(T, \sigma) \quad \text{if } t > t_t$$

$$\frac{E^*}{E_{o,20}} = 0.4 \times 10^{-2} \quad (4.19)$$

Comments

ANDERBERG accepts a simplified model for creep [Anderberg, 1986]. He comments that a simplified model where the creep strain is incorporated in the stress-strain relationship in approximate way is appropriate for design purposes but is not accurate enough for analytical studies. Using the approximate model, one obtains conservative values on the safe side.

The choice of the creep model depends on the structural member to which it will apply. For example, the influence of creep on the deformation behaviour of fire exposed slender columns is greater than that of fire-exposed beams.

Existing Computer Programs

In CEFICOSS, creep is not taken into account in an explicit way. Of course, it is considered implicitly by means of softened stress-related strain ε_{σ} . This choice is due to lack of useful data and due to the belief of the authors that in real composite structures this phenomenon is not of significant influence. In their opinion the strainhardening effect in the stress-strain relationship which has been introduced in their code is of much greater importance.

Codes

In the European Recommendations for the Fire Safety of Steel Structures, ECCS recommends a more approximate method for analysing the deformation behaviour than the one using the creep law where the stress and temperature

history should be taken into account. It is shown that for practical purposes the heating rate does not significantly influence the deformation behaviour of steel structures under fire conditions. Except for this, using creep laws is rather complicated and has been evaluated only for beams.

Creep is taken into account explicitly in neither BS5950:Part 8 nor EC3. They are based on the anisothermal data of British Steel which have been derived from tests with heating rate of 10°C. It is shown that for heating rates 2-50°C/min, the effect of high temperature creep is included in the stress-strain relationship.

4.4.4 Total strain

Any test - either steady state or transient state test - can be simulated by using the behaviour model as expressed in Equation 4.5. The creep strain can only be directly measured in steady state tests. The creep from steady state tests can be used to predict the creep strain in transient state tests.

Anderberg (1983) found that there is good agreement between transient state tests and calculation. However, there is a discrepancy in the temperature region 100-300°C which is due to an instability phenomenon in the material called "thermal activated flow". This phenomenon occurs only in transient tests and it has been observed that it can differ, even in identical tests.

CHAPTER FIVE

Design of Steel Columns in a Fire Environment

5.1 Fire resistance periods

5.1.1 Building Regulations

The most common objectives of fire safety requirements are:

- Life Safety
- Property Protection
- Prevention of Conflagration.

The Life Safety and the Prevention of Conflagration form the basis of the statutory requirements for life safety. The protection of the property is usually a matter arranged between the owner / occupier and his insurance company. The level of this protection depends on the willingness of the owner to invest in the protection of his property.

The Requirements that serve the above objectives are given in a form of well defined measures in the Building Regulations. Each of these Requirements has multi-level specifications for components involved in the design of any building however complex it is.

Such Regulations as above are :

- The Building Regulations 1985 with Amendments
- The Building Standards (Scotland) Regulations 1981
- The Building Regulations (Northern Ireland) 1977 with Amendments
- London Building (Constructional) By-Laws 1972, and amending By-Laws 1974 and 1979.
- London Building Acts (Amendment) Act 1939, Section 20 with addendum Code of Practice.

Although these various Regulations have some detailed differences, they are based on the same concept and seek to achieve a high degree of personal safety. Regarding this objective, it seems that they have been

successful because in disaster situations in the U.K. to date there has not been serious loss of life.

For office buildings, the set of Regulations to which England and Wales are subjected recommends thirty (30) minutes as the fire resistance period in contrast to the one applied to Scotland which recommends sixty (60) minutes as the fire resistance period.

5.1.2 The Time Equivalent Calculation Method

Though the exposure of a structural element in a real fire can be very different to that in the standard test, it is acceptable to equate the "fire resistance" of an element of a structure with the time to failure in the standard test.

Ingberg (1928) proposed that the "severity" of a fire (which is an ill-defined term referring to the ability of the fire to cause damage) could be related to the fire resistance requirement using the "equal area hypothesis" in which it is assumed that if the areas under the temperature-time curves (above a baseline of 150°C or 300°C) of two fires are equal, then the severities are equal. If one of these "fires" is the "standard" temperature-time curve, then "severity" and "fire resistance" can be equated. Ingberg, based on the above, developed Table 5.1 from which the fire resistance requirement of a particular compartment can be obtained directly from the measured or anticipated fire load.

Law (1971) also derived such a relationship by analysing the thermal responses of an insulated column exposed to the standard temperature-time curve and to a real fire. The relationship is given by the following formula:

$$T_e = \frac{k L}{\sqrt{A_T A_w}} \quad (5.1)$$

Fire load(lb/ft ² of floor)	Standard Fire Duration (hours)
10	1
15	1.5
20	2
:	:
:	:
60	7.5

lb/ft²=1.49kg/m²

Table 5.1

where: T_e =time equivalent in minutes
 k =constant which has a value around unity
 L =total fire load in the compartment measured in kilograms
 A_T =total surface area of the compartment excluding the area of openings i.e. windows and doors in m^2
 A_w =area of opening i.e. area of windows and doors in m^2

The above equation has been adopted for compartments where the area of openings exceeds 10% of the floor area. Because it is based on an analysis of insulated columns it does not hold for exposed steel work.

Pettersson, Magnusson and Thor (1976) proposed an alternative which is to abandon the furnace test completely and rely on calculating the fire protection necessary on the basis of a predicted temperature-time curve including information on the height of the windows and the compartment boundaries.

The proposed equation is:

$$t_e = \frac{0.067 \times q}{\sqrt{\text{Opening factor}}} \quad (5.2)$$

where: q =fire load density expressed in MJ/sq m of bounding surface area of the compartment

$$\text{Opening Factor} = \frac{A\sqrt{h}}{A_t} \quad (5.3)$$

where: A =area of windows
 h =the mean height of windows
 A_t =the total bounding surface area of the compartment

The above relationship, in a simplified form, was adopted by the CIB Organisation in the 1985 and its recent form is given below:

$$T_e = c w q_f \quad (5.4)$$

where: q_f = fire load density expressed in MJ/m² for the floor area of the compartment i.e. the amount of combustible material per unit area of compartment floor.

w = ventilation factor

c = conversion factor min/MJ/m² (it accounts for the thermal properties for the surrounding structural members).

The fire load density q_f is derived from:

$$q_f = \frac{1}{A_f} \sum M_i H_{ui} \quad (\text{MJ/m}^2) \quad (5.5)$$

where: A_f = floor area of fire compartment (m²)

M_i = amount of combustible materials

H_{ui} = lower calorific values of the combustible materials (MJ/kg)

The above equation applies to unprotected (and permanent) fire loads.

The value w can be calculated for a specific compartment as shown below:

$$w = w^1 \frac{A_f}{\sqrt{A}} \quad (5.6)$$

where: $w^1 = \frac{A_f}{\sqrt{A_f} \sqrt{h}} \quad (5.7)$

A_f =floor area of the compartment (m^2)

A =total area of floor and window openings (m^2)

h =average height of the windows weighted

A_i =total interior surface area of the compartment

including openings.

The conversion factor c is related to the thermal properties of the enclosure by means of thermal conductivity k and the heat capacity ρc_p which are combined to render the thermal inertia $b=\sqrt{k\rho c_p}$.

Values of c to be adopted for various ranges of values of b are given in the following table:

b ($Wh^{1/2}/m^2K$)	c (mins/MJ/ m^2)
Over 42	0.05
Between 12 and 42	0.07
Less than 12	0.09

Table 5.2 Conversion factor c

Values of the thermal inertia b for various materials are given below:

Normal weight concrete	38 ($Wh^{1/2}/m^2K$)
Light weight concrete	14
Aerated concrete	7
Steel	250
Brick	20
Timber	10

Table 5.3 Thermal inertia b

For medium weight construction, the value of $c=0.07$ can be adopted. The value of $c=0.1$ will render a conservative estimate for different types of wall lining.

A value of $w=1.5$ for the ventilation factor will provide a conservative estimate corresponding to an opening area greater than 10% of the floor area.

Equation (5.6) was basically derived for vertical opening areas. It is recommended to confine the application of equation (5.6) to fairly high fire compartments ($H \gg 3.5\text{m}$) and to average height of opening areas which are well above ground level and are much larger than one metre ($h \gg 1.0\text{m}$). The greater effectiveness of horizontal roof openings may be considered by calculating the total area A of the openings from the equation $A=A_v+\alpha_f A_h$ with α_f equal or greater to one and A_h the horizontal opening area.

For office buildings, the average fire load density varies from less than 100 MJ/m² in the lobby to 1500 MJ/m² in file storage rooms. The Codes use 80% fractile values i.e. 80% of rooms within the various occupancy groups have fire load densities less than the value adopted in the calculation. For offices, the 80% fractile values vary from 520 to 720 MJ/m².

5.2. Codes related to the Performance of Constructions in Fire.

5.2.1 BS 5950:Part 8 Code of Practice for Fire Resistant Design

BS 5950:Part 8 [BSI, 1990] gives recommendations for the following methods:

- a/ fire resistance derived from tests in accordance with BS 476: Part 20 - 21[BSI, 1987].
- b/ fire resistance derived from calculations.

The above methods may also be applied to members for which the required fire resistance has been derived from the consideration of natural fires.

5.2.1.1 Fire Resistance derived from testing

Different test procedures are used for different types of construction. BS 476:Part 20-24 [BSI, 1987] presents the above methods within general guidelines which are appropriate for the specific construction under test.

The common feature of all these methods is that the tested construction is heated using the BS 476:Part 20 heating curve which is suitable for simulating the thermal exposure encountered in a fire involving "cellulosic" materials. To simulate hydrocarbon fires another temperature-time curve must be applied i.e. MOBIL curve etc.

Failure is defined by a variety of criteria depending upon the function of the component. For example, in the case of loadbearing members failure is deemed to occur when they cannot support the test load.

The fire resistance of bare columns is taken either as the time to the limit of stability or the limiting steel temperature of 550°C for members loaded to their maximum permissible design stress.

The fire resistance of columns in walls is normally taken as the time to the limit of stability. Though the exposed portions of the members attain very high temperatures, the applied load can still be supported by the much cooler areas of the sections' profile which were protected by the blockwork. However, the steep temperature gradients created across the sections' profile cause bowing. Of greater concern to the designer is the case where the height of the wall is considerably greater than that evaluated in the fire test.

Test data related to the fire behaviour of unprotected beams and columns are presented in a British Steel publication "Compendium of UK Standard Fire Test Data on Unprotected Structural Steel" [ASFPCM, 1988]. The series

of fire tests carried out on bare steel columns is limited. The columns are standardized to a three (3) m length. The column ends are fixed. The column is subject to nominally concentric axial load and the end moments are negligible. Failure occurs when the rate of vertical or horizontal deflection is so great that the load cannot be sustained.

The data applies to members which are loaded in fire conditions in accordance with BS 449 (permissible stress limits which are approximately 60% of the ambient temperature yield stress). Fire resistance can be achieved for ^{unprotected} beams and columns of certain proportions. For columns in simple construction (as defined in BS 5950: Part 1), according to fire test results, a fire resistance of 30 minutes without fire protection is achieved if the section factor H_p/A does not exceed the value $H_p/A=50 \text{ m}^{1/2}$. In practical terms, only very heavy steel members satisfy this limit. Lighter sections can achieve 30 minutes of fire resistance if the applied load is reduced.

5.2.1.2 Fire Resistance derived from calculation

The principal method of evaluating the fire resistance of structural members is the "load ratio" method. According to this method, the temperature in a critical part of the member (determined from the design temperatures or from fire test data) is related to its reduced strength under fire conditions. The load ratio gives a measure of the stress in the member at the fire limit state relative to the design strength of the member. The higher the load ratio, the higher the required retention of strength of the member in a fire, the lower the temperature of the critical element to resist the applied loads. The assumption under which the above method holds is that the members are not subject to second order effect resulting from deflections during the fire.

5.2.1.2.1 Limiting temperature method

The fire behaviour of columns in compression, may be determined using the limiting temperature method. According to this method, one can determine the design temperature q_D at a required period of fire resistance (T_{req}), the limiting temperature q_L at factored load and compare them. Where the limiting temperature is not less than the design temperature, the member may be considered to have adequate fire resistance without protection. When the limiting temperature is less than the design temperature, protection must be provided.

Design temperature is defined as the temperature that the critical element, the element of a section that would reach the highest temperature in fire conditions, will attain at the end of the specified period of fire resistance in a test in accordance with BS 476:Part 20 and 21[BSI, 1987].

Limiting temperature is defined as the temperature of the critical element of a member at failure under fire conditions and it depends on the following:

- the ratio of the load carried during the fire to the load capacity at 20°C,
- the temperature gradient within the member,
- the stress profile through the cross section,
- the dimensions of the section.

The load ratio for columns exposed on up to four sides should be given by the following according to the type of construction:

-For columns in simple construction, designed in accordance with BS 5950: Part 1[BSI, 1985], the following equation holds:

$$R = \frac{F_f}{A_g p_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{p_y Z_y} \quad (5.8)$$

where: A_g =the gross area,
 p_c =the compressive strength,

- p_y = design strength of the steel,
 Z_y = elastic modulus about the minor axis,
 M_b = buckling resistance moment (lateral torsional),
 F_f = axial load at the fire limit state,
 M_{fx} = maximum moment about the major axis at the
 fire limit state,
 M_{fy} = maximum moment about the minor axis at the
 fire limit state.

- For columns in continuous construction designed in accordance with BS 5950:Part 1, the following equations hold:

$$R = \frac{F}{A_g p_y} + \frac{M_{fx}}{M_{cx}} + \frac{M_{fy}}{M_{cy}} \quad \text{or} \quad (5.9)$$

$$R = \frac{F}{A_g p_y} + \frac{m M_{fx}}{M_b} + \frac{m M_{fy}}{p_y Z_y} \quad (5.10)$$

where: $A_g, p_c, p_y, Z_y, F, M_x, M_y, M_b$ are as defined above,

M_{cx}, M_{cy} , as appropriate to the axis of bending, are the moment capacity of section about the major and minor axes in the absence of axial load,

m is the equivalent uniform moment factor.

Once the load ratio is determined, the limiting temperature for the applicable load ratio is given in Table 5 [Appendix A] for different types of members. The data in Table 5 [Appendix A] have been derived -where possible- from tests. In some cases, they have been supplemented by computer modelling. For compressive members of low slenderness ($\lambda \leq 70$), the limiting temperatures are based on test results supplemented with information from the ECCS Recommendations. The test data on columns relates to

slenderness values less than 50 because of the restraints of the furnace. In Table 5, the limiting temperatures have been extended up to a value corresponding to a slenderness value less than 70 because this is the case for columns in buildings.

It was found that the ECCS simplified column method provided an excellent fit to the limited test data. The load ratio R is determined from the following relationship:

$$R = \text{Load multiplier} \times \text{steel strength reduction factor}$$

The load multiplier was taken as 0.85 (as in the ECCS Recommendations) to take into account some end fixity of the columns and also the fact that on average the steel should be stronger than the specified minimum. The strength reduction factor is based on 0.5% strain in the steel because of the relatively low strains that are experienced in columns just prior to failure.

For columns with slenderness ratio $\lambda > 70$ and $\lambda \leq 180$, the adopted strain limit is 0.5% and the load multiplier is increased to 1. The effect of end restraint is less significant. High strains in the steel affect the stability of the member by reducing the effective elastic modulus which leads to increased lateral deflections. The use of an upper limiting temperature corresponding to a load ratio of 0.3 avoids the potential risk of damage if normally stressed members because of bracing, for example, are subject to additional loading. For slender columns, the same as above apply. Slender columns benefit more from continuity in a fire than stocky columns but because of lateral restraint secondary forces and eccentricities may cause a reduction in the column strength. The information related to the behaviour of slender columns in a fire is very limited.

5.2.1.2.2 Design temperature

The thermal response of I section columns and beams has been measured in the furnaces and correlated with analytical methods. For reference

purposes, these temperatures may be presented in terms of the section factor (H_p/A) or alternatively the flange thickness t_f . For beams, the flange thickness gives a slightly better measure of performance than the section factor which is averaged over the cross-section. Columns exposed to fire from four sides may be considered uniformly heated. In this case, the section factor gives a good measure of the average temperature rise in the member. The design temperature as defined in the previous Section depends on the following:

- Section configuration
- Section dimensions

In the Code, design temperatures are presented as a function of flange thickness for unprotected columns and beams. The data has been determined partly from computer analyses using FIRES-T and partly from fire tests. From BS 5950:Part 8 and for common periods of fire resistance, the design temperature may be determined according to:

- Table 6 for unprotected columns and tension members,
- Table 7 for unprotected beams supporting concrete floors.

For periods of fire resistance other than those presented in the Code, design temperatures can be determined from tests or by using computer analysis.

5.2.2 European Recommendations for the Fire Safety of Steel Structures.

The required performance of a structural element is defined in terms of a particular failure criterion. This criterion may be either a limit of deformation or the capacity to support a given load.

5.2.2.1 Deformation Behaviour - Limit state of deformation

The analysis of the deformation behaviour is based on the material properties of steel at elevated temperatures as given in the same document where creep is taken into account implicitly. Using creep laws, the method of calculation has been worked out so far only for beams.

The limit state of deformation is considered to be attained if it may be assumed that actual failure occurs at extreme deformation, which has as a result the function that adjacent members cannot be fulfilled and the spread of fire to other rooms cannot be contained. The actual value of this limit of deformation has an arbitrary character.

For beams and slabs in fire tests, the recommended deformation criterion can be expressed as follows:

$$\delta \geq \frac{l^2}{800h_x} \quad [\text{mm}] \quad (5.11)$$

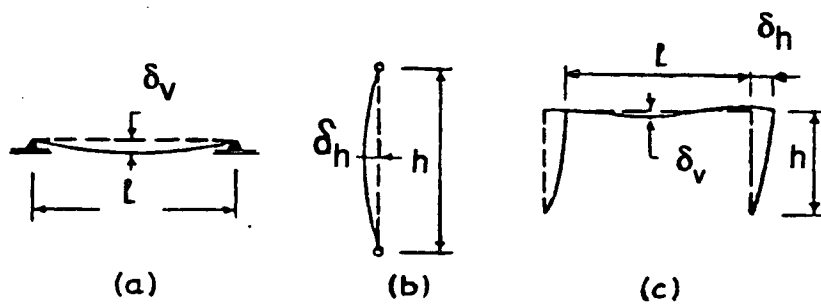
where: δ =max. deflection [mm]

l =span [mm]

h_x =height of steel section [mm]

For a practical range of the span to height ratio, which is $15 > \frac{l}{h_x} > 25$, this criterion tends to $\frac{\delta}{l} \geq \frac{1}{30}$. where l represent the characteristic dimensions described in Figure 5.1.

If the analysis is based on the determination of a complete deformation behaviour, the deformation criterion can be checked. If the analysis is based on the determination of the ultimate load bearing capacity, the deformation will be unknown and the deformation criterion cannot be checked. However, for practical values of the height to span ratio of structural steel members and if no membrane forces are involved, the collapse criterion and the deformation criterion, as described in the following Section, give similar



characteristic dimension: (a)= l
 (b)= h
 (c)= δ_h (lateral deflection)
 δ_v (vertical deflection)

Fig. 5.1 Characteristic dimensions (ECCS, 1983).

values of fire resistance. There is therefore no need to check the deformation criterion.

5.2.2.2 Ultimate Load Bearing Capacity - Limit state of failure

The load bearing capacity at elevated temperatures can be calculated using elementary plastic theory, where the same theory has been used at normal temperatures (which implies that instability does not influence the load bearing capacity significantly).

If the use of elementary plastic theory is prohibited, the reliability of the method of calculation must be demonstrated explicitly. In the case of columns, buckling curves at elevated temperatures are recommended and are derived from the ECCS buckling curves at room temperature. The description of these curves is given in Section 5.2.2.3.

The limit state of failure is reached when the load-bearing capacity of the structural load-bearing element or structural assembly decreases to the level corresponding to the load considered to act on the structure under fire conditions. The limit state of failure corresponds to an infinitely high rate of deformation which is often almost impossible to determine because of difficulties with the loading equipment or the limited depth of the furnace.

For beams, a maximum permissible deformation rate, which will be on the safe side can be defined as below :

$$\frac{d\delta}{dt} \geq \frac{l^2}{15h_x} \quad (5.12)$$

where: δ =max deflection [mm]
 l =span [mm]
 h_x =height of the steel section[mm]
 t =time

The combination of the deformation and failure criteria for beams, known as the Robertson-Ryan criterion, is given below:

$$\frac{d\delta}{dt} \geq \frac{f^2}{15h_x} \quad [\text{mm/h}] \quad \text{and} \quad (5.13)$$

$$\delta \geq \frac{f^2}{800h_x} \quad [\text{mm/h}] \quad (5.14)$$

According to the Code, the above criteria does not apply to columns. In contradiction to this statement, the same Code recommends the application of the criterion of the maximum deflection not exceeding 1/30 th of the characteristic dimension of the structural element under consideration for columns as well as frames.

5.2.2.3 Buckling of steel columns

There is little information available for the solution of the stability problem related to columns and their behaviour in fire environment. The reasons why limited information exists are given below:

- a/ A very limited number of laboratories in Europe are equipped with testing apparatus allowing full scale tests ,
- b/ The main effort has been placed on the evaluation of the fire resistance of protected columns.

ECCS (European Convention of Constructional Steelwork) proposes a design method, to tackle the stability problem in fire conditions, which is based on the European Recommendations for Steel Construction at room temperature.

- European Recommendations at room temperature

In the European Recommendations for Steel Construction, the ECCS has proposed five basic buckling curves which are applicable to cross-sections of different shapes.

The European curves are presented as relationships between N , the ratio of the column critical stress to its yield stress, and its slenderness factor λ , the ratio between the column slenderness ratio and the critical slenderness ratio

$$\lambda_E = \pi \sqrt{\frac{E}{\sigma_r}} \quad (5.15)$$

Because of the use of N and λ , the curves are independent of the material properties.

The analytical expression for the five non-dimensional buckling curves that were adopted is as follows:

$$N = \frac{1 + \alpha(\lambda - 0.2) + \lambda^2}{2\lambda^2} - \frac{1}{2\lambda^2} \times \sqrt{[1 + \alpha(\lambda - 0.2) + \lambda^2]^2 - 4\lambda} \quad (5.16)$$

with

Curve	a_0	a	b	c	d
α	0.125	0.206	0.339	0.489	0.756

Table 5.4

For beam-columns, where the moment or the load eccentricity has to be taken into account, the same expression applies by introducing a factor η^* .

In this case, the following equations holds:

$$N = \frac{1 + \eta^* + \lambda^2}{2\lambda^2} - \frac{1}{2\lambda^2} \times \sqrt{[1 + \eta^* + \lambda^2]^2 - 4\lambda^2} \quad (5.17)$$

with
$$\eta^* = \alpha (1 - 0.2) + \frac{e_{eq}}{i^2/v} \quad (5.18)$$

where: e_{eq} = equivalent eccentricity,
 i = radius of gyration,
 v = half the height of the profile.

- European Recommendations in fire conditions

In fire conditions, it is impossible to follow the same theoretical procedure as the one developed by ECCS for bare steel columns at ambient temperatures because the evaluation of the geometrical imperfections, the residual stresses, the scatter of the yield stress within a cross-section, the stress-strain relationship of the steel in compression and in traction e.t.c. is affected by so many uncertainties.

The ECCS proposal for a design method adopts a range of buckling curves corresponding to different temperatures which are derived from the ECCS buckling curves at room temperature.

In order to transform the equations of the non-dimensional buckling curves at room temperature to take into account the temperature variation, variations of yield stress and Young's modulus with temperature have to be included in the analytical equations for the fire situation. In the ECCS Regulations, the equations describing the buckling curves at ambient temperature are fitted to include the temperature effect by substituting $\sigma_{r\theta}$ to σ_r and E_θ to E as follows:

$$N_\theta = k_1 N \quad \text{and} \quad (5.19)$$

$$\lambda_{\theta} = \sqrt{\frac{k_1}{k_2}} \lambda \quad (5.20)$$

where:

$$k_1 = \frac{\sigma_{r\theta}}{\sigma_r} = 1 + \frac{\theta}{767 \ln \frac{\theta}{1750}} \quad (5.21)$$

for $0 \leq \theta \leq 600^\circ\text{C}$

$$k_1 = \frac{\sigma_{r\theta}}{\sigma_r} = \frac{108(1 - \frac{\theta}{1000})}{\theta - 440}$$

for $0 \leq \theta \leq 1000^\circ\text{C}$

$$k_2 = \frac{E_{\theta}}{E} = \frac{[-17.2 \times 10^{-12} \theta^4 + 11.8 \times 10^{-9} \theta^3 - 34.5 \times 10^{-7} \theta^2 + 15.9 \times 10^{-5} \theta + 1]}{\quad} \quad (\text{N/mm}^2) \quad (5.22)$$

Since, $\sqrt{\frac{k_1}{k_2}} \approx 1$ for $0 < \theta < 600^\circ\text{C}$,

$$\lambda_{\theta} = \lambda \quad (5.23)$$

The dimensionless buckling curves at elevated temperatures are given by the following approximate equations:

-for axially loaded columns in fire conditions:

$$N_{\theta} = k_1 N \quad (5.24)$$

or

$$N_{\theta} = \frac{\sigma_{r\theta}}{\sigma_r} \left[\frac{1 + \alpha(\lambda - 0.2) + \lambda^2}{2\lambda^2} - \frac{1}{2\lambda^2} \times \sqrt{[1 + (\lambda - 0.2) + \lambda^2]^2 - 4\lambda^2} \right] \quad (5.25)$$

The coefficient α (dimensionless) varies with the selected curve and the values are the same as at ambient temperature.

-for eccentrically loaded columns in fire conditions:

$$N_{\theta}^* = \frac{\sigma_{r\theta}}{\sigma_r} \left[\frac{1 + \eta^* + \lambda^2}{2\lambda^2} - \frac{1}{2\lambda^2} \times \sqrt{[1 + \eta^* + \lambda^2]^2 - 4\lambda^2} \right] \quad (5.26)$$

The buckling curves in a fire environment that have been presented are derived for bifurcation buckling. They do not take into account other types of instability. For safety reasons, only curve "c" is recommended at the present time for all classes of profiles according to which the value of the coefficient α is equal to 0.489.

The above design method has been compared with a large number of experimental results obtained in Belgium, Denmark, and Germany on steel columns in fire conditions with slenderness ratios varying between 25 and 167. The agreement is shown to be excellent and on the safe side after the application of a correction procedure that has been introduced to achieve an improved consistency between the analytical and experimental fire resistances.

In the same proposal, since there is no criterion for buckling under fire conditions in national or international standards, the time of failure is the time at which the thermal elongation is cancelled out by the shrinkage of the column. The advantage of this criterion is that it is easy to apply and measure and that it is independent of the column length. This proposed

time of failure is not subject to interpretation and it is based on scientific experience.

5.2.2.4 Critical Temperature of Columns

The critical temperature of a load bearing steel element or structural assembly is the temperature at which the limit state of failure is expected to be attained.

The critical temperature of columns according to the result of a conventional fire resistance test in which the test specimen is concentrically loaded may be given by approximate rules based on the buckling curves (Section 5.2.2.3) . Under the above conditions of the standard test, the critical temperature can be given from Figure 5.2 for a particular value of ratio of the effective yield stress at elevated temperature to the nominal yield stress at room temperature (ψ).

Assuming that the column failure occurs when the design load (kN^*) equals the critical buckling load ($N_{cr\theta} A \sigma_{y20}$) the following equation holds:

$$kN^* = N_{cr\theta} A \sigma_{y20} \quad (5.27)$$

$$\text{From the definition of coefficient } \psi, N_{cr\theta} = \psi N_{cr} \quad (5.28)$$

From equations (1) and (2),

$$kN^* = \psi N_{cr} A \sigma_{y20} = \psi N_{cr} \quad \text{with } N_{cr} = N_{cr} A \sigma_{y20} \quad (5.29)$$

So, the limit state of failure is given by the equation:

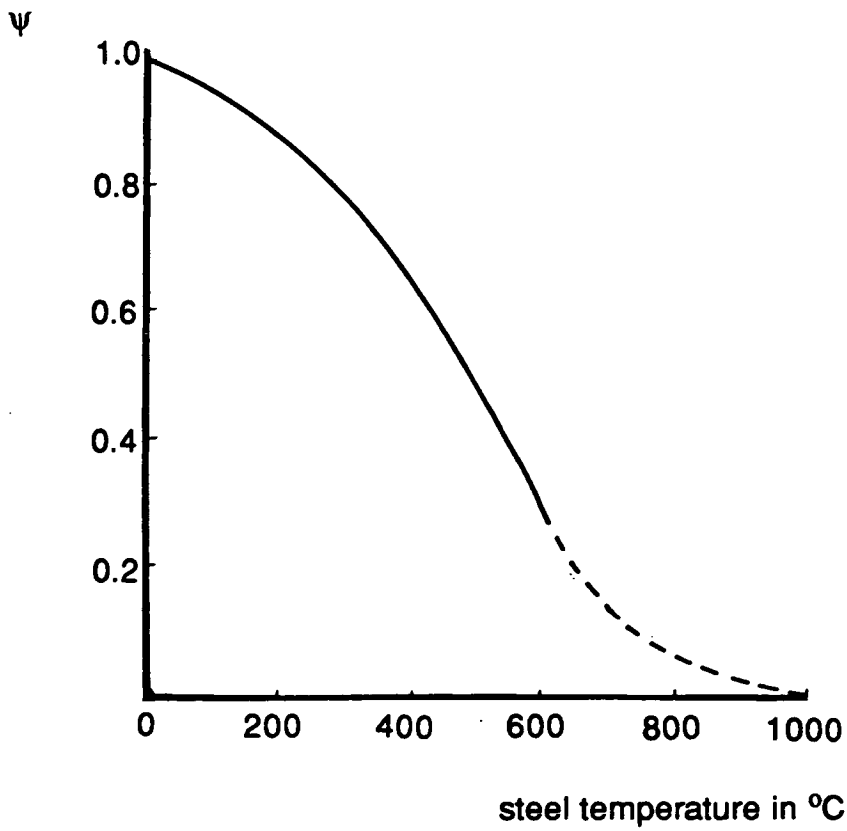


Fig. 5.2 Ratio ψ of the effective yield stress at elevated temperature to the nominal yield stress at room temperature as a function of the steel temperature (ECCS, 1983).

$$\psi = \frac{kN^*}{N_{cr}} \left(= \frac{\sigma_{y\theta}}{\sigma_{y20}} \right) \quad (5.30)$$

where:

ψ = temperature dependent coefficient,

k = load multiplier equal to 0.85,

N^* = normal load considered to act on the column during a fire which is the design load (dead load + characteristic live load),

$N_{cr} = A \sigma_{y20} N_{cr}$

ECCS buckling load at room temperature corresponding to the so-called c-curve,

where: A = cross sectional area,

σ_{y20} = nominal yield stress of the steel at room temperature,

N_{cr} = dimensionless buckling load at room temperature.

The dimensionless buckling load at room temperature is given by the following equation:

$$N_{cr} = \frac{1 + 0.489(\lambda - 0.2) + \lambda^2}{2\lambda^2} - \frac{1}{2\lambda^2} \sqrt{[1 + 0.489(\lambda - 0.2) + \lambda^2]^2 - 4\lambda^2}$$

for $\lambda > 0.2$ (5.32)

or

$$N_{cr} = 1$$

for $\lambda \leq 0.2$

where: λ = modified slenderness ratio

$$\lambda = \frac{\lambda}{\pi \sqrt{\frac{E}{\sigma_{y20}}}} \quad (5.33)$$

- λ =slenderness ratio at room temperature
- E =modulus of elasticity at room temperature
 = 2.1×10^5 N/mm².

In the above calculation of critical temperature, the normal load N^* is defined as the design load (dead load + characteristic live load) which is considered to act on the structure throughout the fire.

The above approach is rather conservative because the probability of the occurrence of a fully developed fire while, at the same time, the full characteristic load is present is only very small. The possibility of reduction of the live load should be introduced.

5.2.3 Fire Engineering Design Manual

If a steel temperature of 550°C is taken as the criterion for the onset of failure, the 'fire resistance' of structural steel members can be calculated if the temperature-time curve is known. The model developed by Pettersson *et al.* (1976) can be used as input to iterative heat transfer calculations in order to determine the 'fire resistance' of a particular member.

Pettersson has developed the above method further to analyse the stability of the structural element (under load) at its maximum temperature taking into account the temperature-dependence of the factors which determine the strength of the steel, such as modulus of elasticity, yield stress e.t.c. In this way, a definition of a 'failure temperature' has been avoided.

In Figure 1 [Appendix C], a flow chart which illustrates the design procedure according to Pettersson is given. Following this procedure, one must determine the static load which shall not cause the structure to collapse

under fire exposure conditions, the design fire load and the opening factor. Conversion of the fire load and opening factor for the fire compartment in question into equivalent fire load and opening factor follows. If the fire load is low and the conditions are favourable in other respects, one has to check if the structure can be constructed without insulation or wherever it has to be protected. For unprotected structures, one has to determine the resultant emissivity, calculate the F_s/V_s ratio and determine the maximum steel temperature. For columns, the degree of expansion has to be assessed if expansion is restrained, and the critical load has to be determined. To determine the critical load, the designer has three cases to design for:

- compression only,
- simultaneous flexure and compression,
- the risk of out-of-plane instability.

Design Static Load

Pettersson, regarding the determination of design static load, divides the buildings according to their design and type of activity as follows:

- buildings in which complete evacuation of people in the event of fire cannot be assumed,
- buildings in which complete evacuation of people in the event of fire can be assumed.

Buildings in which complete evacuation of people in the event of fire cannot be assumed

In buildings such as large hotels, blocks of flats and offices, it is not possible to assume that complete evacuation of people takes place in the case of a fire. In this way, the design has to include the possibility of fire being in progress in limited parts of the building without complete evacuation of people taking place. In this type of building, the designer has to show that

the load bearing structural components will not collapse due to the most critical combination of dead load, live load (with load factor 1.4) and snow load. (with load factor 1.2).

The **dead load** is to be calculated in the conventional way.

For the **live load**, the following values apply:

Type of premises	Static load		Mobile load	
	kgf/m ²	{kN/m ² }	kgf/m ²	{kN/m ² }
Dwellings and hotel rooms, hospital wards, etc	35	0.35	70	0.70
Offices and schools (classrooms and group study rooms)	35	0.35	100	1.00
Shops, department stores, assembly halls (excl. records rooms and warehouses containing compact stacked loading)	35	0.35	250	2.50

Table 5.5 Live load [Pettersson, 1976].

The values of the **snow load** applied for fire design are taken as 80% of the values according to current loading regulations.

Buildings in which complete evacuation of people in the event of fire can be assumed

In some cases, such as single-storey buildings, it is possible to assume that complete evacuation of people takes place. For the fire design of these buildings, the load factors remain the same as above but there are

differences regarding the values of live load (mobil load particularly) which are given below:

Type of premises	Static load		Mobile load	
	kgf/m ²	{kN/m ² }	kgf/m ²	{kN/m ² }
Dwelling and hotel rooms, hospital wards, etc	35	0.35	35	0.35
Offices and schools (classrooms and group study rooms)	35	0.35	55	0.55
Shops, department stores, assembly halls (excl. records rooms and warehouses containing compact stacked loading)	35	0.35	70	0.70

Table 5.6 Live load [Pettersson, 1976].

The values of the mobile load are changed because of the evacuation of the building in the event of fire.

In some cases, the designer has to estimate the local increase of the live (mobile) load which may increase due to people moving from the part of the building which is affected by the fire to another part of the building or in conjunction with the evacuation of the building.

The above values, concerning loads and load factors, are conservative.

Design Fire Load

The fire load in a fire compartment is the total quantity of heat released upon complete combustion of all the combustible material contained inside the fire compartment, inclusive of building frame, furnishings, cladding and floor coverings.

The fire load per unit area is given by the total internal surface area of the fire compartment and is calculated from the relationship:

$$q = \frac{\sum m_v H_v}{A_t} \quad (\text{Mcal/m}^2) \{ \text{MJ/m}^2 \} \quad (5.34)$$

where:

- m_v = the total weight of each individual combustible material constituent, v , in the fire compartment (kg),
- H_v = the effective calorific value of each individual combustible material constituent, v , in the fire compartment (Mcal/kg) {MJ/kg},
- A_t = the total internal surface area of the fire compartment (walls, floor and ceiling) (m^2).

Complete combustion of all the combustible material in a fire compartment does not take place during a fire.

The fire load per unit area can be calculated, to take into account the degree of combustion of each fire load component, by the following relationship:

$$q = \frac{\sum \mu_v m_v H_v}{A_t} \quad (5.35)$$

where: $\mu_v =$ a non dimensional factor with a value between 0 and 1 which indicates the actual degree of combustion for each fire load component ,v.

For example, bookcases and floor coverings that have a very low degree of combustion, have μ values appreciably below unity.

The effective calorific value of some solid, liquid and gaseous materials are given in the following Table:

Solid Materials	Effective Calorific value(H)	Liquids	Effective Calorific value(H)	Gases	Effective Calorific value(H)
<i>Dry materials</i>	Mcal/kg $\times 4.2\{ \text{MJ/kg} \}$		Mcal/kg $\times 4.2\{ \text{MJ/kg} \}$		Mcal/m ³ n
Clothes	4.0-5.0	Crude oil	10.3	Acetylene	13.6
Leather	4.0-5.0	Diesel oil	9.7-10.1	Carbon Monoxide	3.0
Polyvinyl Chloride(PVC)	4.4-5.2	Paraffin	9.8	Coal gas	4.0
Rubber waste	5.0	Petrol	10.4	Hydrogen	34.0
Paper and cardboard	3.8-4.2	Spirits	7.6-8.2		
Wood	4.1-4.7	Tar	9.0		

Moist**materials**

$$H_F = H(1 - 0.01F) - 0.006F$$

where: F=moisture content
of material in % by weight

Table 5.7 Effective Calorific Values

[Pettersson (1976)].

According to the above, the design fire load can be determined by direct calculation {Equation (5.34) and (5.35)}. However, the designer requires sufficient knowledge of types and quantities of furniture and fittings in the building. Equation (5.35) is more difficult to apply due to a lack of experimentally substantiated and verified values of μ .

To allow for any future alterations or rearrangements in the building, the design must not be rigid regarding these aspects. It is safer therefore to base the design on statistical investigations concerning the magnitude of the fire load for the type of building in question (Table 2, Appendix C).

The above interpretation of the fire load is valid for a uniformly distributed fire load inside the fire compartment. This assumption does not introduce problems caused by the violent turbulence which develops during the fire causing small temperature differences in different parts of the compartment.

The present design method does not cover the case where the fire load is extremely uneven and causes significant temperature effects in certain structural elements.

Opening factor

Pettersson presents a procedure whereby it is possible to calculate the temperature-time curve of the fire process (both the flame phase and the

cooling phase for any fire load). The theoretical procedure is based on a relationship which, for each time, describes the balance between the heat energy produced and consumed per unit time in a fire compartment. This relationship is given below:

$$I_C = I_L + I_W + I_R + I_B \quad (5.36)$$

where:

- I_C = the heat released during combustion,
- I_L = the heat removed due to the replacement of hot gases by cold air,
- I_W = the heat dissipated to and through the wall, ceiling and floor structures,
- I_R = the heat dissipated by radiation through openings in the fire compartment,
- I_B = the quantity of heat stored in the gas volume in the fire compartment per unit time.

The above equation holds under the following simplifying assumptions:

- combustion is complete and takes place exclusively inside the fire compartment,
- at every instant, the temperature is uniformly distributed within the entire fire compartment,
- at every instant, the surface coefficient of heat transfer for the internal enclosing surface of the fire compartment is uniformly distributed,
- the heat flow to and through the enclosing structures is unidimensional and with the exception of any door and window openings, is uniformly distributed for each type of enclosing structures.

Because the terms I_C and I_L in the heat balance equation are proportional to the air flow factor $A\sqrt{h}$ and the term I_W is proportional to the internal surface area A_i of the fire compartment, the opening factor is a combination of the above geometrical quantities and it is defined as:

$$\text{Opening factor} = A\sqrt{h}/A_i. \quad (5.37)$$

where: $A_i =$ the total internal surface area of the fire compartment (the area of the walls, floor, ceiling, inclusive of openings) $\{m^2\}$,

$A =$ the total area of the vertical openings in the fire compartment (windows, ventilation openings and other vertical openings) $\{m^2\}$,

$h =$ a mean value of the height of these openings, calculated according to the following equation:

$$h = \frac{\sum A_v h_v}{\sum A_v} \quad (5.38)$$

where: $A_v =$ the area of each opening $,v$, in the fire compartment $\{m^2\}$,

$h_v =$ the height of each opening v in the fire compartment $\{m^2\}$.

For a fire compartment which also contains horizontal openings (as in Fig.2, Appendix C) and supposing that the flow through the horizontal openings is not dominant, the opening factor can be calculated from the expression:

where: $\left(\frac{A\sqrt{h}}{A_t}\right)_v$ =the opening factor for the vertical openings ($m^{1/2}$)

f_k =correction coefficient from the nomogram in Fig.5.3 given as input data the ratios $(A\sqrt{h_1}/A\sqrt{h})$ and $(A\sqrt{h}/A)$ and the fire temperature in the compartment in question.

When the flow of air and gases takes place mainly through the horizontal openings, the flow becomes unstable and difficult to describe by means of a simple theoretical model.

The above calculation for opening factor assumes that the windows and doors are immediately destroyed when a fire breaks out. This assumption is valid for doors and windows with ordinary glass but it does not directly apply to doors of a certain fire resistance or windows with reinforced glass. In the latter cases, to be on the safe side, the design has to be carried out using a value of opening factor whereby the calculation will give a higher maximum temperature of the steel structure.

Conversion to equivalent fire load and opening factor

Using the design data presented in the form of tables and diagrams in the Swedish Fire Engineering Design Manual [Pettersson, 1976], the designer can directly determine the maximum steel temperature, with the restriction that the compartment in question must be identical with the description of the fire compartment type A (characterised by its surrounding construction and given in Table 3, Appendix C).

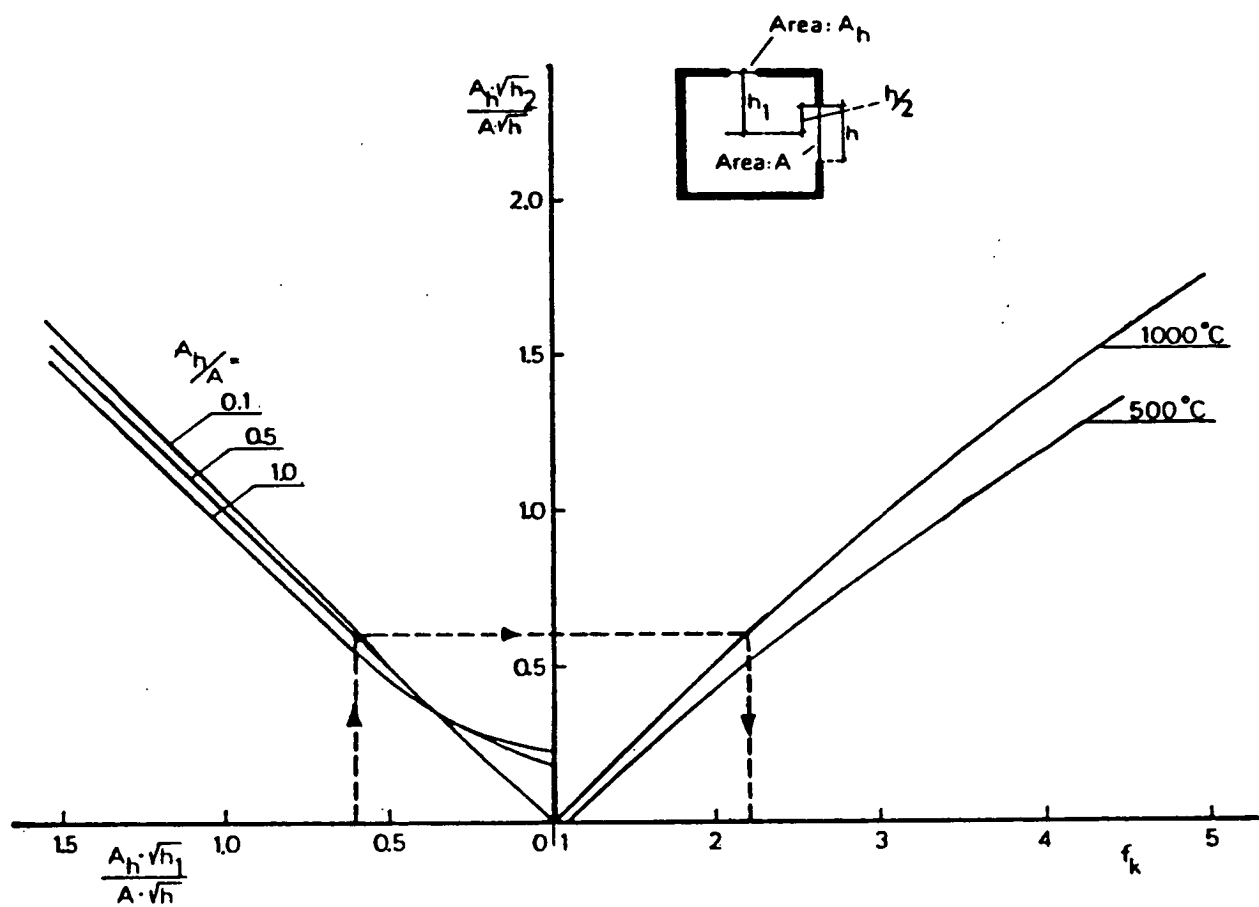


Fig. 5.3 Nomogram for calculation of the coefficient f_k
(Pettersson, 1976).

The above mentioned design data may be used for the design of other types of fire compartments by converting the temperature-time curve in the compartment in question into the temperature-time curve in the fire compartment type A. Employing a factor k_f , the actual fire load and opening factor can be converted into an equivalent fire load and opening factor by multiplying the actual values by k_f .

Values of k_f are given in Table 3 (Appendix C) for different types of fire compartment (characterised by their surrounding construction) and as a function of the size of the opening factor. In some cases, the value of k_f can vary as a function of the magnitude of the fire load as well. Because the relationships used to determine the factor k_f are not linear, its values determined by linear interpolation for types of compartment differ from those listed in Table 3 (Appendix C). The fire compartments used in interpolation therefore should be those which yield the lowest values of k_f . In determining k_f , a combination of different types of fire compartments cannot be made which results in a negative contribution for k_f .

The temperature-time curves for fire compartment type A for variable fire loads (q) and opening factors ($A\sqrt{h}/A_f$), given in Figure 5.3, are based on the assumption that the combustion process is ventilation controlled and the mean rate of the combustion of wood (in kg per unit time) during the flame phase is known and proportional to the air flow factor ($A\sqrt{h}$).

The fire load controlled combustion process is complicated due to uncertainties involved regarding the determination of the rate of combustion, the size of the fire load, the method of storage and the degree of distribution of the fuel. Except for fire compartments with large openings, in which fire load combustion process mainly occurs, there is no reliable theoretical model available to assess the flow configuration. The gas flow through openings, with velocity components in both the horizontal and vertical directions, reduces the interchange of gases between the fire compartment

and its surroundings. For gas flow through the small openings, it has been confirmed experimentally that the horizontal velocity component is the dominant one. So, because of the uncertainties listed above, it is rather risky for the designer to assume a fuel-controlled fire

A CIB research programme which examined fully developed fires, showed that a maximum well was within the ventilation-controlled regime (Fig 5.4). Assuming a fuel-controlled fire is less severe, savings can be made on the cost of structural fire protection. Because of the complexity of the problem and the uncertainties involved (as stated above), it is recommended that the design should assume a ventilation-controlled fire. This yields results on the safe side.

Resultant emissivity ϵ_r

Radiation is the transmission of energy by electromagnetic "waves".

A surface which absorbs all radiation incident upon it is called a black surface ($\epsilon=1$).

Real surfaces are less efficient absorbers and emitters than black surfaces ($\epsilon<1$) and their absorptivity and emissivity depend on the wavelength, λ .

Emissivity is then defined as follows:

$$\epsilon(\lambda) = \frac{E(\lambda)}{E_b(\lambda)} \quad (5.40)$$

where: $E(\lambda)$ =the emissive power at wavelength λ

$E_b(\lambda)$ =total emissive power of a blackbody

at wavelength λ

$$= \sigma T^4 \text{ \{kW/m}^2\} \text{ with } \sigma = 56.7 \times 10^{-12} \text{ \{kW/m}^2\text{K}^4\}$$

(Stefan-Boltzman Constant)

(5.41)

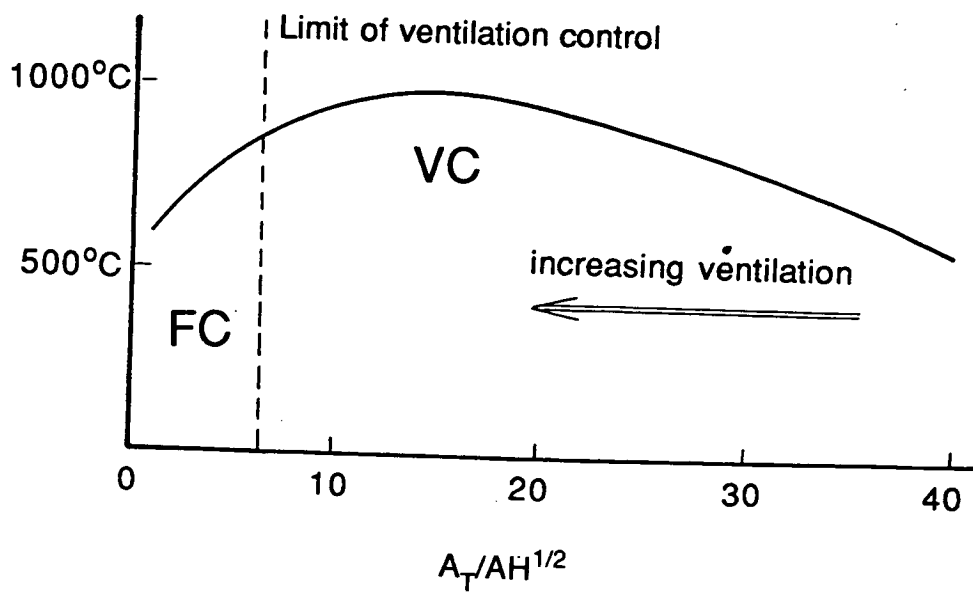


Fig. 5.4 Fuel - controlled and ventilation - controlled fire regimes (Drysdale, 1989-90).

Grey surfaces are the ones for which emissivity is independent of wavelength. Emissivity is then defined as follows:

$$\epsilon = \frac{E}{E_b} \quad (5.42)$$

where: E = the emissive power of a grey body
 $= \epsilon \sigma T^4$ {kW/m²} with $\sigma = 56.7 \times 10^{-12}$ {kW/m²}
 (Stefan-Boltzman Constant)

The resultant emissivity (ϵ_r) is dependent on the emissivities of the flames (ϵ_f) and the steel structure (ϵ_s), on the sizes of the flames and the steel structure and their positions relative to each other.

For the simple case of two infinitely large parallel surfaces where all radiation from the one surface falls on the other surface and vice versa, the resultant emissivity is given below:

$$\epsilon_r = \frac{1}{\frac{1}{\epsilon_f} + \frac{1}{\epsilon_s} - 1} \quad (5.43)$$

where: ϵ_f = emissivity of the flames
 $= 1 - \epsilon^{-1.1} x_f$ with x_f flame thickness (m) (5.44)
 $= 1$ for thick flames
 < 1 for thin flames

ϵ_s = emissivity of the steel section
 (treated as a grey body)

For a column exposed to fire on all sides, the emissivities of the flames and the steel structure are assumed to be 0.85 and 0.80 respectively which give a resultant emissivity of $\epsilon_r=0.7$. For the case of a column outside a facade, because it is exposed to less radiation, a value of $\epsilon_r=0.3$ can be used.

Recommended values for use for resultant emissivity ϵ_r , which yield results on the safe side, are given in Table 5a.

F_s/V_s ratio

where F_s is the fire exposed surface of the steel section and

V_s is steel section volume per unit length

The F_s/V_s ratio varies as a function of the section dimensions and the method of construction.

Values of the ratio F_s/V_s are given in Table 5b for rolled I-profile sections exposed to radiation on all sides and with one flange concealed. Examples of calculation of the ratio F_s/V_s for different types of construction are given in Figure 3 (Appendix C).

Maximum steel temperature

The quantity of heat transferred to the steel section per unit length and over a short interval is as follows:

$$Q = \alpha F_s (\theta_t - \theta_s) \Delta t \quad (\text{kcal/m}) \quad \{\text{J/m}\} \quad (5.45)$$

where: α = the surface coefficient of heat transfer of the boundary layer

$$= 23 + \frac{5.77\epsilon_r}{\theta_t - \theta_s} \left[\left(\frac{\theta_t + 273}{100} \right)^4 - \left(\frac{\theta_s + 273}{100} \right)^4 \right] \{\text{W/m}^2 \text{ } ^\circ\text{C}\}$$

(5.46)

- F_s =the surface of the steel section per unit length
 exposed to fire (m²/m),
 θ_t =the gas temperature in the fire compartment at
 time t (°C),
 θ_s =the temperature of a steel section at time t (°C),
 Δt =the length of time interval (h) {s}.

This quantity of heat per unit length causes the temperature of the steel section to rise by an amount determined by the thermal capacity of the steel as given below:

$$Q = c_{ps} \Delta\theta_s V_s \gamma_s \quad (\text{kcal/m}) \quad \{\text{J/m}\} \quad (5.47)$$

- where: c_{ps} = the specific heat capacity of the steel
 (kcal/kg °C) {J/kg°C},
 $\Delta\theta_s$ =temperature rise in the steel section (°C),
 V_s =volume per unit length of the steel section
 (m³/m),
 γ_s =density of the steel (kg/m³)
 =7850 kg/m³

By equating (a) and (b):

$$\Delta\theta_s = \frac{\alpha}{\gamma_s c_{ps}} \frac{F_s}{V_s} (\theta_t - \theta_s) \Delta t \quad (^\circ\text{C}) \quad (5.48)$$

In order to derive the increase of the steel temperature over a short time interval, the following assumptions have been made:

- the heat flow is unidimensional,
- the steel section is heated uniformly at any time.

The first assumption can be satisfied assuming that the structural element in question is of infinite length and the corners are so small they do not affect the validity of the calculation.

The second assumption is satisfied when the actual structural element is not subjected to thermal gradients of significant value (column built into a wall with one flange and part of the web exposed).

The maximum steel temperature derived from equation (5.48) is dependent on the F_s/V_s ratio, the value of the resultant emissivity ϵ_r , the gas temperature θ_t in the fire compartment which is determined on the basis of the opening factor and the fire load of the fire compartment.

Maximum temperature values calculated by computer using equation (5.48) for the fire compartment type A are presented in Table 6 (Appendix C).

Critical load

Steel structure subject to an axial compressive force.

At ambient temperatures, one may design steel columns subjected to axial loading and against buckling employing two principal methods which apply to the design of:

- initially straight columns,
- columns of representative initial curvature and unintentional load eccentricity.

According to the first approach which deals with the design of initially straight columns, the stress-strain curve of the material in question is taken into account and the ideal buckling load is more often determined by the tangent modulus theory. The permissible buckling load is obtained by dividing the buckling load by a safety factor to take into account the initial

curvature and unintentional load eccentricity. The safety factor is dependent on the slenderness ratio of the column.

According to the second approach which deals with the design of columns of representative initial curvature and unintentional load eccentricity, the maximum compressive stress in the column is determined. The critical axial compressive force is defined as the compressive force which causes the maximum compressive stress to attain a value which is either the yield stress or the 0.2% proof stress of the material. The permissible compressive force is obtained by dividing the buckling load by a safety factor which is independent of the slenderness of the column.

For the latter group and for a column which is free to expand while heated, the buckling stress (σ_k) can be calculated by the following expression:

$$\sigma_k^2 - \sigma_k \left[\sigma_{0.2} + \pi^2 E \left(4.8 \times 10^{-5} + \frac{1}{\lambda^2} \right) \right] = - \sigma_{0.2} \frac{\pi^2 E}{\lambda^2} \quad (5.49)$$

where: σ_k =buckling stress (kgf/cm²) {MPa},
 $\sigma_{0.2}$ =yield stress or 0.2% proof stress at the actual steel temperature (kgf/cm²) {MPa},
 E =modulus of elasticity at the actual steel temperature (kgf/cm²) {MPa},
 $\lambda = \frac{\beta L}{i}$ =effective slenderness ratio,

where: L =length of the column (cm) {m},
 β =nondimensional coefficient ,
 i =the radius of gyration of the cross-section with respect to the neutral axis through the centroid (cm) {m}.

In the above equation (5.49), the initial curvature and the unintentional eccentricity have been taken into account in the calculation. They were

included in the form of a pure initial curvature, the maximum value of which is given below:

$$f = 4.8 \times 10^{-5} \frac{(\beta L)^2}{d} \quad (5.50)$$

where: d = distance of the neutral axis to the extreme fibre of the cross section in compression at the section which governs design (m).

The nondimensional coefficient β is a function of the fixity conditions of the column, the variation of cross section and the variation of the axial compressive force along the column.

The stress $\sigma_{0.2}$ can be determined from the stress-strain curves of the steel material in question at different steel temperatures. Because the stress-strain curve at elevated temperatures is softly rounded, to obtain a better approximation of the buckling stress, the secant modulus of elasticity is used instead of the initial modulus of elasticity and the 0.2% proof stress ($\sigma_{0.2}$) instead of the 0.5% proof stress ($\sigma_{0.5}$).

Applying the above relationship (5.49), calculated relationships between the buckling stress and the slenderness ratio for steel columns made of materials with yield stresses at room temperature of 220,260,320 {MPa} at different temperatures θ_s have been obtained. These are presented in Fig.5.5. The temperature - dependence of the proof stress 0.5% and the stress and temperature dependence of the secant modulus E (as in Fig. 5.6) are taken into account for the calculation of the $\sigma_k - \lambda$ curves. These material properties have been determined from tensile tests where the test

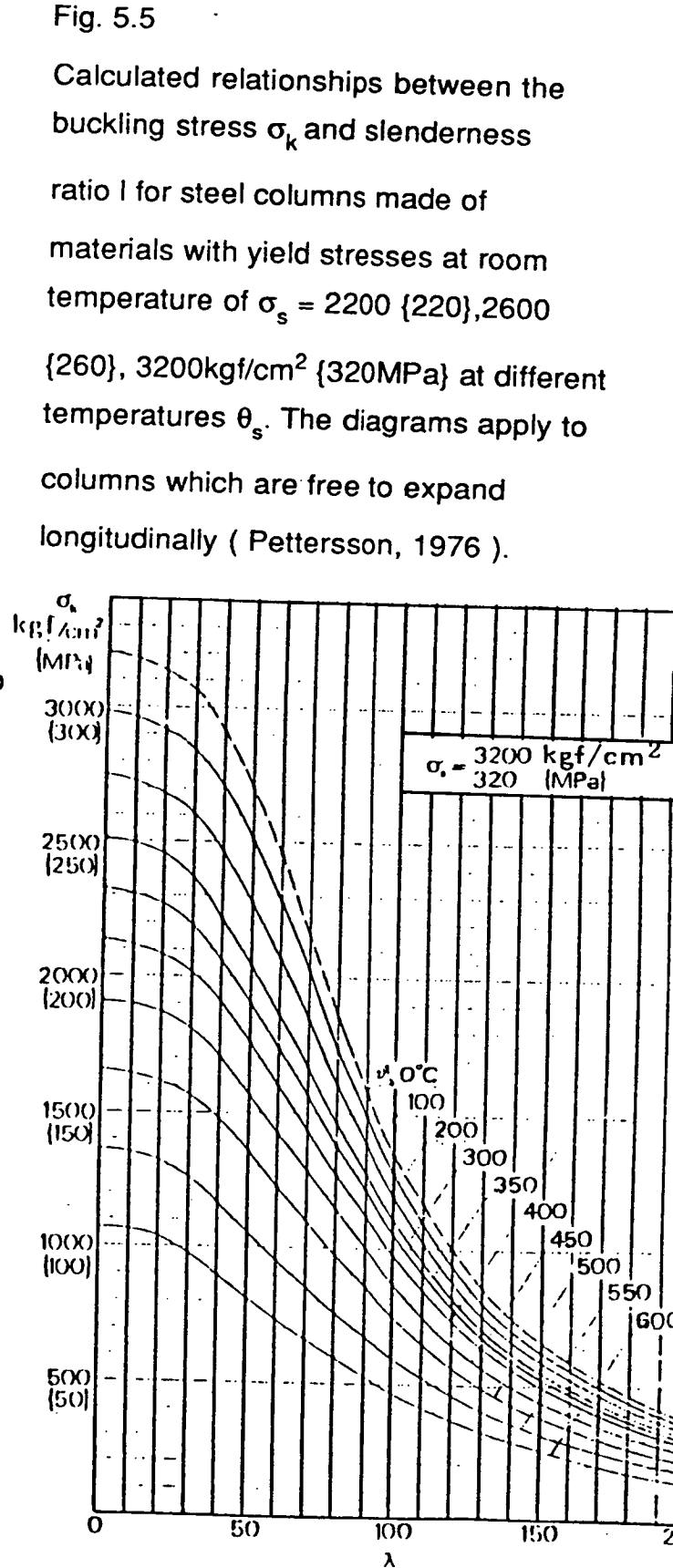
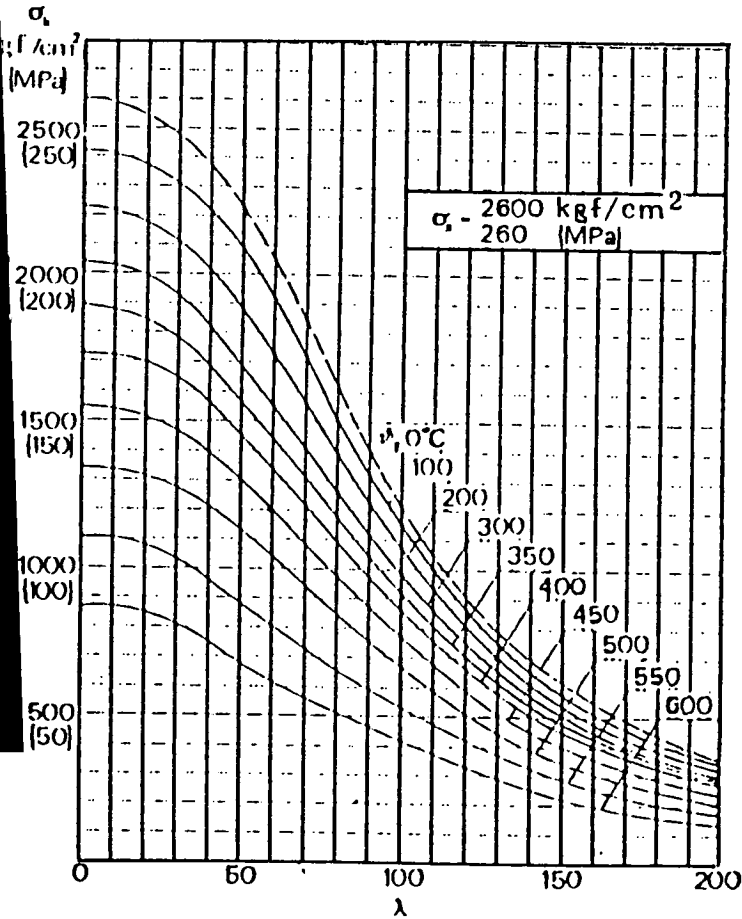
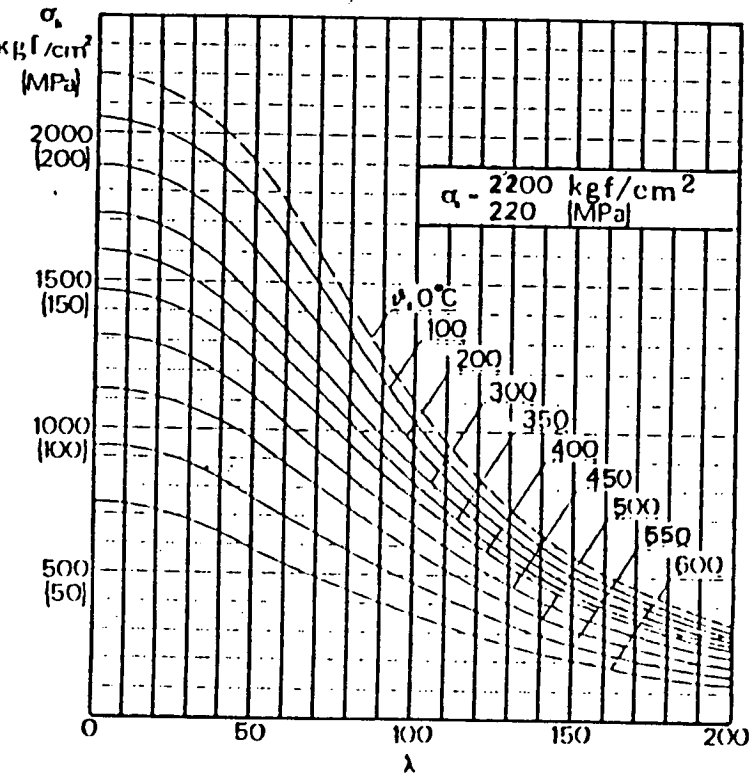


Fig. 5.5

Calculated relationships between the buckling stress σ_k and slenderness ratio λ for steel columns made of materials with yield stresses at room temperature of $\sigma_s = 2200 \text{ (220)}, 2600 \text{ (260)}, 3200 \text{ kgf/cm}^2 \text{ (320 MPa)}$ at different temperatures θ_s . The diagrams apply to columns which are free to expand longitudinally (Petterson, 1976).

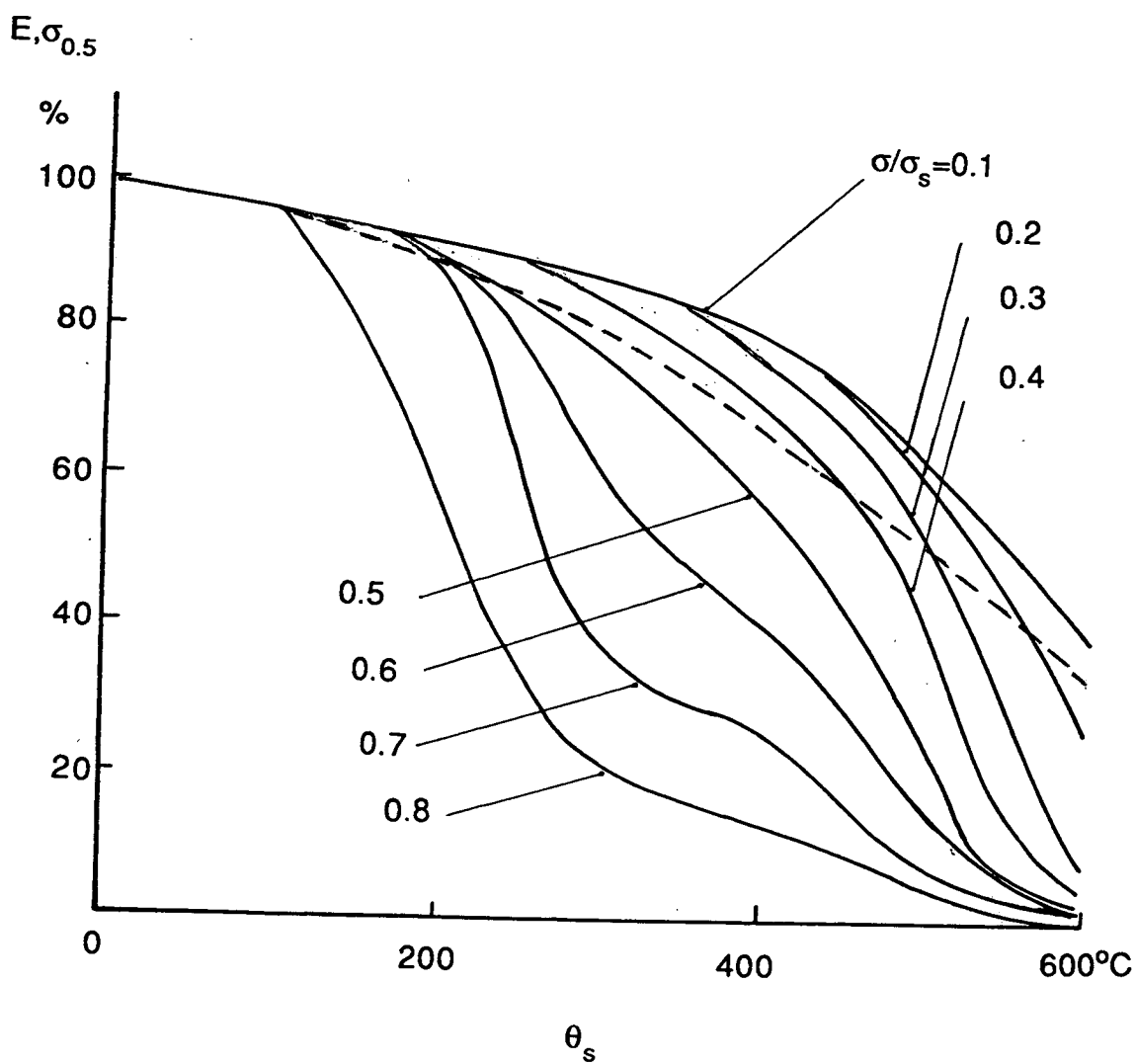


Fig. 5.6 The modulus of elasticity (secant modulus) E as a function of the steel temperature θ_s for different values of the ratio of the stress σ to the yield stress at room temperature θ_s . The dashed curve indicates the variation with temperature of the 0.5% proof stress (Petterson, 1976).

specimens were loaded at a constant rate of loading of 18 kgf/cm²/min {1.8 MPa/min} to include the effects of the short-term creep.

In this way, from the above mentioned σ_k - λ curves, for a max steel temperature θ_{max} (Table 6 for uninsulated columns) and a slenderness ratio λ , the critical stress can be obtained and the buckling load of the column is calculated as follows:

$$\sigma_k = \frac{N_k}{A} \quad (\text{kgf/cm}^2) \{ \text{MN/m}^2 \} \quad (5.51)$$

where: N_k = buckling load of the column (kgf) {MN},

A = area of cross section of the column (cm²) {m²}.

The design criterion for the loadbearing function is that the minimum loadbearing capacity given by the buckling load (N_k) must not be less than the axial force the cross section is subjected to.

When the longitudinal expansion of the column is partially prevented in a fire, the compressive force in the column increases and the column buckles at a lower imposed external load. The longitudinal expansion can be partially prevented because of the reduced value of the modulus of elasticity of steel as the steel temperature increases or because of the adjacent structural elements.

The reduction in the expansion of the column due to reduced value of the modulus of elasticity is:

$$\Delta L = \frac{NL}{A} \left(\frac{1}{E} - \frac{1}{E_0} \right) \quad (\text{cm}) \{ \text{m} \} \quad (5.52)$$

where: E = modulus of elasticity (secant modulus) of the steel at the stress and steel temperature in question (kgf/cm²) {MPa},

- E_o =modulus of elasticity of the steel at ordinary room temperatures (kgf/cm²) {MPa},
- A =area of cross section of the column (cm²) {m²},
- N =magnitude of the compressive force (kgf) {MN}.

The expansion of an unloaded steel column when there is no restraint on longitudinal expansion is:

$$\Delta L' = \alpha \theta_s L \quad (\text{cm}) \{m\} \quad (5.53)$$

- where: α =coefficient of linear expansion (°C⁻¹)
- θ_s =steel temperature (°C),
- L =original length of the column (cm) {m}.

So, the resultant expansion ΔL of the column is:

$$\Delta L = \Delta L' - \Delta L = \alpha \theta_s L - \frac{NL}{A} \left(\frac{1}{E} - \frac{1}{E_o} \right) \quad (\text{cm}) \{m\} \quad (5.54)$$

The reduction in the expansion of the column because of the adjacent structural members is:

$$\Delta L_r = \gamma \Delta L \quad (\text{cm}) \{m\} \quad (5.55)$$

- where: γ =degree of expansion, a nondimensional coefficient with values between 0 and 1.
- =1 (no restraint on longitudinal expansion)
- =0 (the expansion is completely prevented)

Determination of the degree of expansion γ according to the above equation is a complicated process. Values of the degree of expansion γ for some common cases are given in the diagram in Figure 4 (Appendix C).

Critical buckling stresses as a function of the steel temperature, slenderness ratio and degrees of expansion for steel columns made from materials with yield stresses of 220, 260, 320 MPa at ordinary room temperatures are given in Figure 5 (Appendix C). Apart from the degree of expansion, there is another parameter, the cross sectional factor i/d , the effect of which is relatively limited. For the curves in Figure 5 (Appendix C), the value $i/d=0.5$ was used for the calculation which for the normal type of cross section yields results on the safe side.

The above described design approach for the calculation of the buckling load is in good agreement with full scale tests. More accurate design based on the actual stress-strain curves would produce the deflection curve of the steel column and the $\sigma_k - \lambda$ curves will be dependent (apart from the maximum steel temperature) on the rates of heating and cooling as well.

In plane instability checks have to be carried out in the following cases:

- when the column subject to an axial compressive force is braced by adjacent structural elements against deflection at right angles to the plane of bending.
- when deformation can freely occur out of the plane of bending and the line of action of the axial compressive force coincides with the shear centre of the cross section.

In all the other cases, a slender column subjected to an axial loading must be checked for out-of-plane instability.

For the design of a column subject to an axial compressive force with regard to out-of-plane instability under fire exposure conditions, Pettersson states that no study has been made so far and he recommends a simplified method which employs a fictive comparative slenderness ratio λ_{fj} .

The fictive comparative slenderness ratio λ_{fj} is given from the expression:

$$\lambda_{fj} = \pi \sqrt{\frac{E}{\sigma_{kiR}}} \quad (5.56)$$

where: E =modulus of elasticity at the maximum steel temperature θ_{max} (kgf/cm²) {MPa}.

σ_{kiR} =the maximum compressive stress in the column cross section due to the ideal out-of-plane buckling load N_{kiR} at the maximum steel temperature θ_{max} , calculated for an initially straight structure according to the elastic theory.

The modulus of elasticity E and its variation with the max steel temperature for mild steel is given in Figure 5.7.

The ideal out-of-plane buckling load can be directly determined from handbooks.

Since the fictive comparative slenderness ratio has been calculated from equation (5.56), from the diagrams which are applicable to in-plane buckling, one can determine the maximum compressive stress σ_{kR} in the column cross section under fire exposure conditions corresponding to the actual out -of-plane buckling load, N_{kR} .

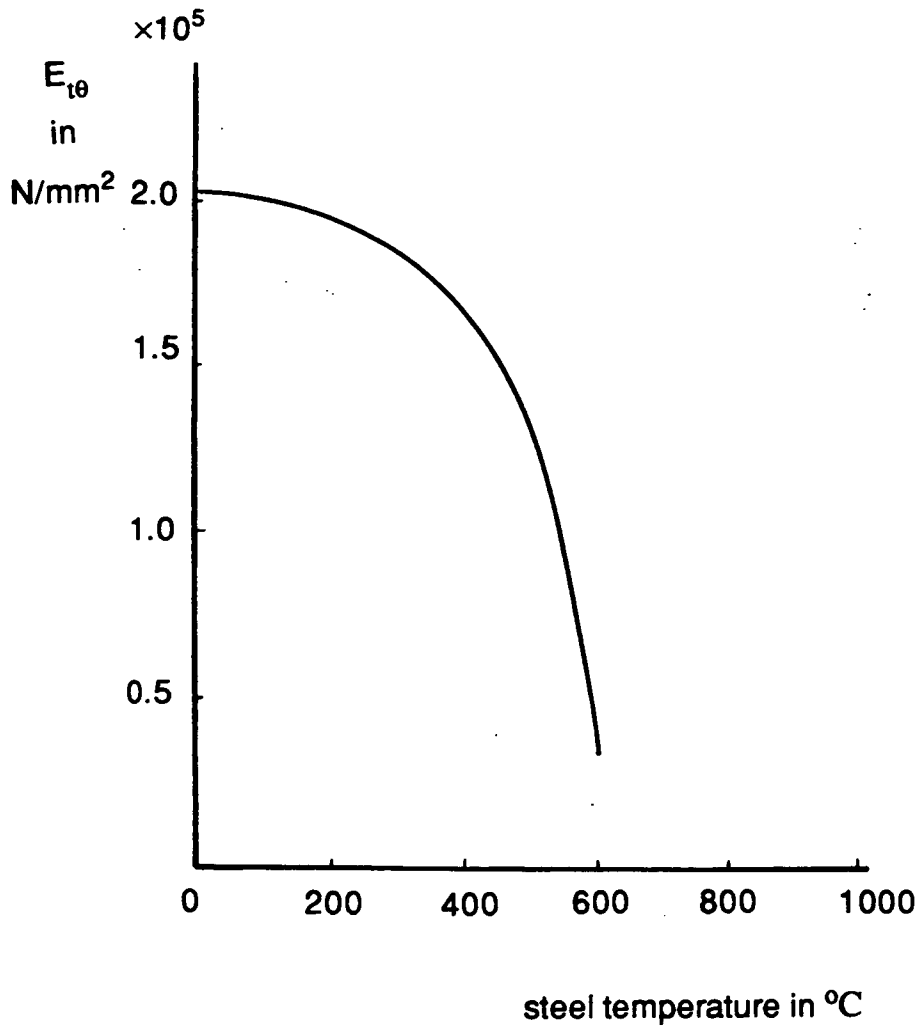


Fig. 5.7 Modulus of elasticity $E_{t\theta}$ as function of the steel temperature (ECCS, 1983).

5.2.4 Discussion

Fire resistance periods

The background to the objectives of the Building Regulations concerned with the behaviour of structural elements in fire, as well as the time equivalent calculation methods which define fire resistance requirements for elements of structure, has been reviewed. Though the fire engineering approach is sound, one has to be concerned with the accuracy and the limitations of the fire severity equations as well as the assumptions made for the determination of the fire load density.

In actual practice many more factors affect fire severity than those taken into account so far. Fire severity is affected by the fire load, surface area, fire load arrangement, wind and the properties of the walls of the fire compartment. The influence of all these factors is substantial. They may cause deviations of the order of 50% from the predicted fire severity. In comparison with the influence of the two most important factors, fire load and opening factor, which can have values in practice in a range extending over a factor of 10 their influence is relatively small. Therefore, it is justifiable to express the correlation between critical fire load and fire severity only in terms of the most important factors, and to take into account the influence of the other factors, which are often unpredictable and uncontrollable, by incorporating an appropriate factor of safety in the fire resistance requirements.

Because relatively small scale tests have been conducted to establish the fire severity relationships, also the fire load involved was timber, the above equations may not hold for the case of large compartments.

The determination of the value of the fire load and the fire load density is critical when using the fire severity equations. Usually, the values of the fire load are derived from several surveys of fire loads and are given in various design Codes. The problem is that the surveys have been carried out some

time ago and the character of various occupancy groups may have changed since the original surveys.

For large inter - connecting spaces in multi - functional buildings, it is difficult to determine the size of the openings.

If the function of the compartment is changed and the elements are designed with small factors of safety, upgrading may be necessary.

Standard fire resistance tests [Witteveen, 1981/82]

In the conventional BS 476 : Part 20 - 24 [BSI, 1987] fire tests, failure is defined as the time when the member can no longer support the test load which means that the load bearing capacity of the construction is reduced below the applied load. The fire environment is the one as defined in BS 476 and stability has been generally measured on fully exposed single elements subject to the maximum permissible load to simulate the worst possible case.

Data derived from conventional fire tests can be used to define failure temperatures which will depend on steel quality, degree of loading and structural features like composite action, restraint and continuity. The steel failure temperature of 550°C is usually adopted for the assessment of results from single element fully loaded fire tests where the temperature distribution throughout the members' cross section is uniform.

Though the fire resistance tests have been performed for more than a century without major changes in test procedure, some serious weaknesses can be observed. Factors like the the heat-flow characteristics of furnaces, variation in material properties, geometrical imperfections, residual stresses, temperature distribution across and along the members and restraint conditions may cause significant differences in test results of structural elements under the same load repeated in the same furnace (repeatability) or tested in different furnaces (reproducibility).

For columns, the length of the specimen generally varies between 3 and 4 m and in view of the capability of most of the existing furnaces the loading must be applied centrally and is kept constant during the test which is a rough approximation of the real situation where the column is loaded by both normal forces and bending moments. Because the design load specifications may be different from country to country, the test report must give detailed information about this matter in order to have comparative results when the test is reproduced in a different furnace. Another factor that affects repeatability as well as reproducibility of the test results is that neither the test load nor the test results are adjusted to take into account the difference between the actual random values and the characteristic values of the material strength, imperfections and residual stresses.

The determination of the ratio between the test load (P) and the ultimate load-bearing capacity (P_{b20}) at room temperature is of more importance than the actual test load. The ultimate load bearing capacity (P_{b20}) is a function of the yield strength for relatively small slenderness ratios (0.12) and a function of the modulus of elasticity for relatively high slenderness ratios (0.04). The coefficient of variation of the yield strength, which is well known from extensive investigations, is higher than the coefficient of variation of the modulus of elasticity. The load bearing capacity is also influenced by initial geometrical imperfections and residual stresses, the coefficient of variation of which is difficult to quantify. The yield strength of a random sample is higher than the characteristic value on which the design load is based. The level of imperfections and residual stresses is lower than the characteristic values specified in the Regulations and Codes. In this way, the ratio P/P_{b20} of a random test specimen will be lower than the design value which varies according to national specifications between 0.6 and 0.7. In a standard fire test, assuming a coefficient of variation of load bearing capacity equal to 0.08, the ratio P/P_{b20} will have a value between 0.5 and 0.67 for most cases.

The heating characteristics of the furnace, as well as the manner of exposure, influence the temperature variation in the longitudinal direction of the column which will affect the reproducibility of the test results. For pin-ended columns, the effects of a non-uniform temperature distribution on the critical temperature is smaller than for columns with both ends built in.

Critical temperature is defined as the temperature at collapse at mid-height of the column.

In a fire resistance test, the test specimen shall be supported and restrained in a manner which is as far as possible similar in nature to the corresponding structural element in service. In a real building, a column has rotational restraints which depend on the stiffness of the surrounding structural elements. In a column furnace, the loading device provides fixed rotational restraint, either pin-ended or one end fully built in and the other end pin-ended. In most of the cases, the real degree of restraint is quite often unknown and may vary during the test. Axial restraint is not accounted for in standard fire resistance tests. For tests on pin-ended columns, even a small degree of unintended rotational restraint, which is unavoidable, has a significant influence on the critical temperature. Fully built-in columns are less sensitive to small changes in restraint. Though fully built in columns are more sensitive to a non-uniform temperature distribution, the influence of the degree of restraint of pin-ended columns on the critical temperature is more critical. So, fire resistance tests on columns fully built in at both ends give more reliable results than the pin-ended ones.

Unavoidable eccentricity of the load results in a combined action of normal force and moment and may cause problems of repeatability as well as problems of reproducibility. It is better to prescribe a certain eccentricity of the load and to account for this eccentricity when determining the load level to be applied.

The factors responsible for inadequate reproducibility are dependent on the heating characteristics and the mechanical characteristics of the loading device of the furnace in question. In order to have better results, the above characteristics must be standardized.

As a conclusion, the fire resistance tests provide a poor instrument for the classification of fire exposed steel columns especially if the design is based on natural fire conditions where analytical methods must be employed to assess the fire resistance.

Analytical methods

In recent years, computation methods for the design of fire resistant steel structures has been developed as a result of further knowledge acquired from recent test programmes and theoretical studies. These methods involve consideration of the development and decay of fires of differing intensities, heat transfer to steel members with their temperature rise being considered in relation to their material properties etc. Unfortunately because of the lack of supporting evidence that justifies the use of the developed models, their use, at least in this country, is limited.

Further refinement and sensitivity can be offered with the application of finite element analysis to serve fire engineering design purposes. This type of analysis may be necessary when temperature gradients of significant value arise within the steel member. The case of a column built as a part of a wall where large temperature gradients occur along the web of the column should be analysed many finite element analysis.

The European Recommendations for calculation of the resistance of steel structural elements exposed to standardised fires have been compiled by Commission 3 "Fire Safety of Steel Structures" of the European Convention

on Metal Structures [ECCS, 1983] and they are the result of several years of study and research undertaken in various European laboratories.

The primary objective of the European Recommendations is to provide the designer of steel structures with a simple tool of calculation that will enable him to justify the duration of stability of his structure in fire and to prove that the design complies with regulations.

The European Recommendations are voluntarily limited to the presentation of a mathematical model which has as the objective of producing results identical to those that would be obtained by testing structural elements in a furnace. In this way the analysis is limited to individual structural elements subjected to the standard fire ISO R 834. Uniform temperature is assumed over the entire section and length of each element. Creep is included in an implicit way in the relationships that describe the variation of the mechanical behaviour with temperature. The design target is that the temperature attained at the stability time (after which the structural element will no longer be stable) as recommended by the Regulations or Standards should be equal to the critical temperature (temperature at collapse at the critical section). Fire resistance is defined as the time necessary for the steel element to reach its critical temperature.

In comparison with laboratory tests, the procedure is far more rapid and cheaper but is limited to the design of structural elements without temperature gradients and exposed to the standard fire.

Theoretically, temperature gradients within a member should improve performance. In an unevenly heated member, where one flange is heated faster than the other, the hot flange will reach its elastic limit while the cool flange is still acting elastically. The hot flange will yield plastically and it will transfer load to the cool flange. The member will retain stability until the load transferred into the cool flange is sufficiently high to cause it to yield plastically. The precise hot flange temperature, at which an unevenly heated

member fails, will depend on the severity of its temperature gradient and the level of applied stress. In this way, the failure temperature is a variable and the fixed assumption of the "critical temperature" must be replaced by another failure criterion.

The structural steelwork code, BS5950 Part 8 [BSI, 1990] introduced the concept of "Limiting temperature" which is the temperature that the hottest part of a section must reach at any given load in order to cause failure of the member.

The "limiting temperature" method is applied to unprotected structural elements and it is valid when the members are not subject to second order effects resulting from deflection in fire.

Some countries like Sweden have more sophisticated regulations which enable the design for natural fires [Pettersson, 1976].

Finite element analysis is worthwhile when applied to the case of columns embedded in walls where the exact deformation behaviour of the element is essential for the design. Though uneven heating of columns under axial load can substantially improve their fire resistance, temperature gradients can induce thermal bowing. Experimental work at the Fire Research Station has shown that design loaded, pin-ended I-section model steel columns first bow towards the heat source, straighten out and fail by bowing in the reverse direction [Cooke, 1987]. Figures 5.8 and 5.9 illustrate what is likely to happen to a steel portal frame building with timber purlins subjected to a low level fire and a high level fire respectively.

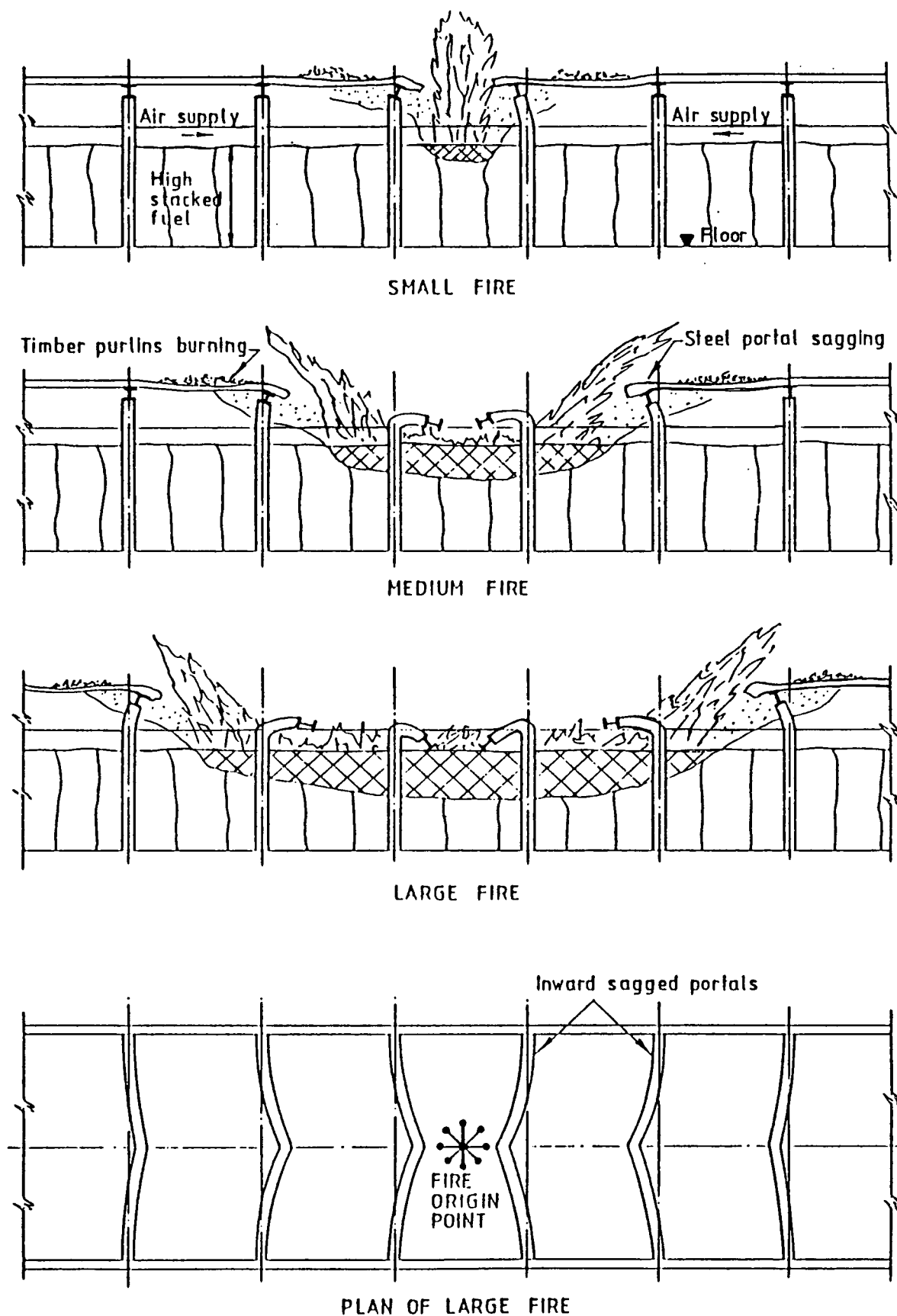


Fig 5.9 High-level fire inside steel portal building with timber purlins showing small, medium and large fires (Barnett, 1986).

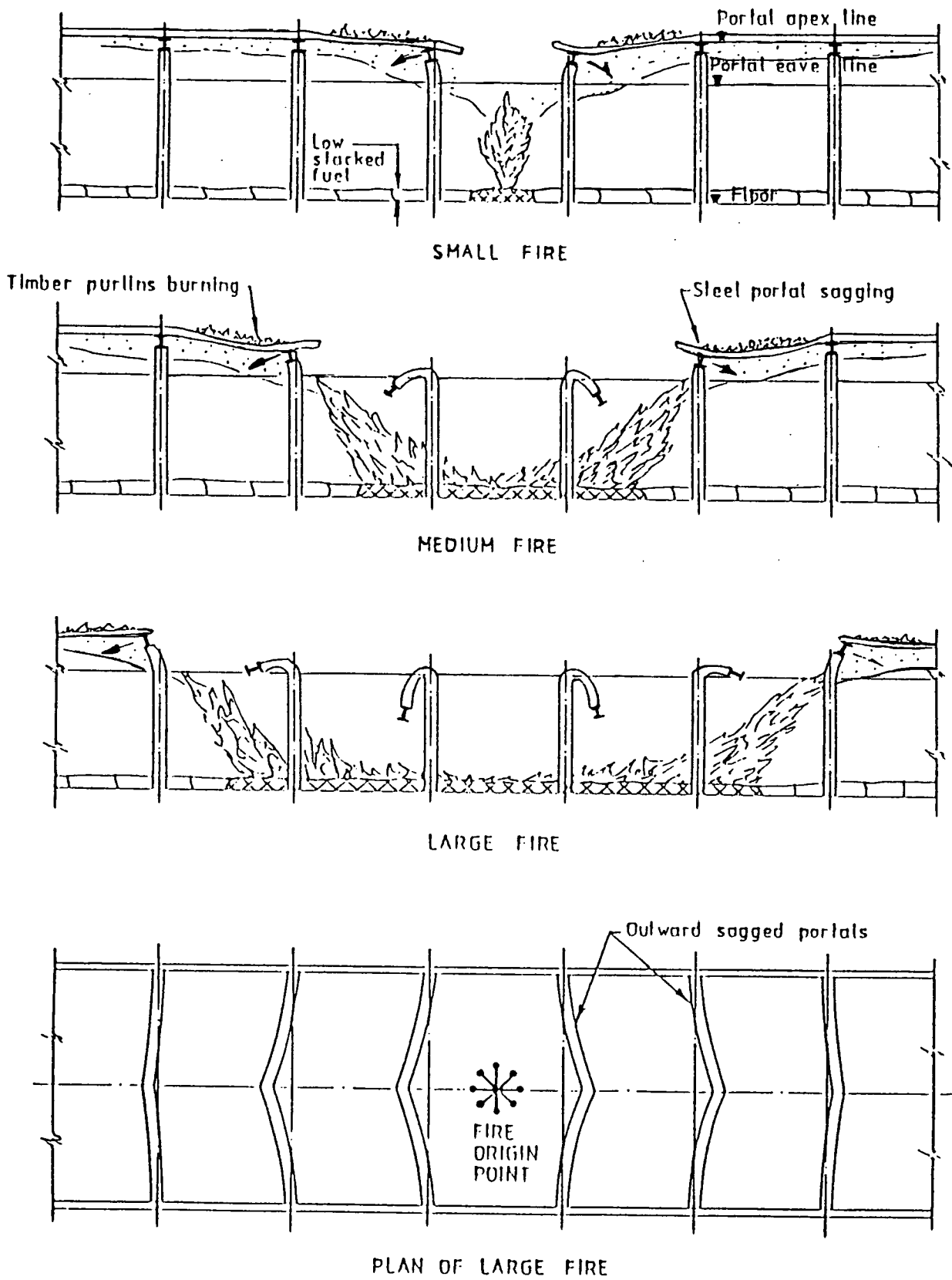


Fig. 5.8 Low-level fire inside steel portal building with timber purlins showing small, medium and large fires (Barnett, 1986).

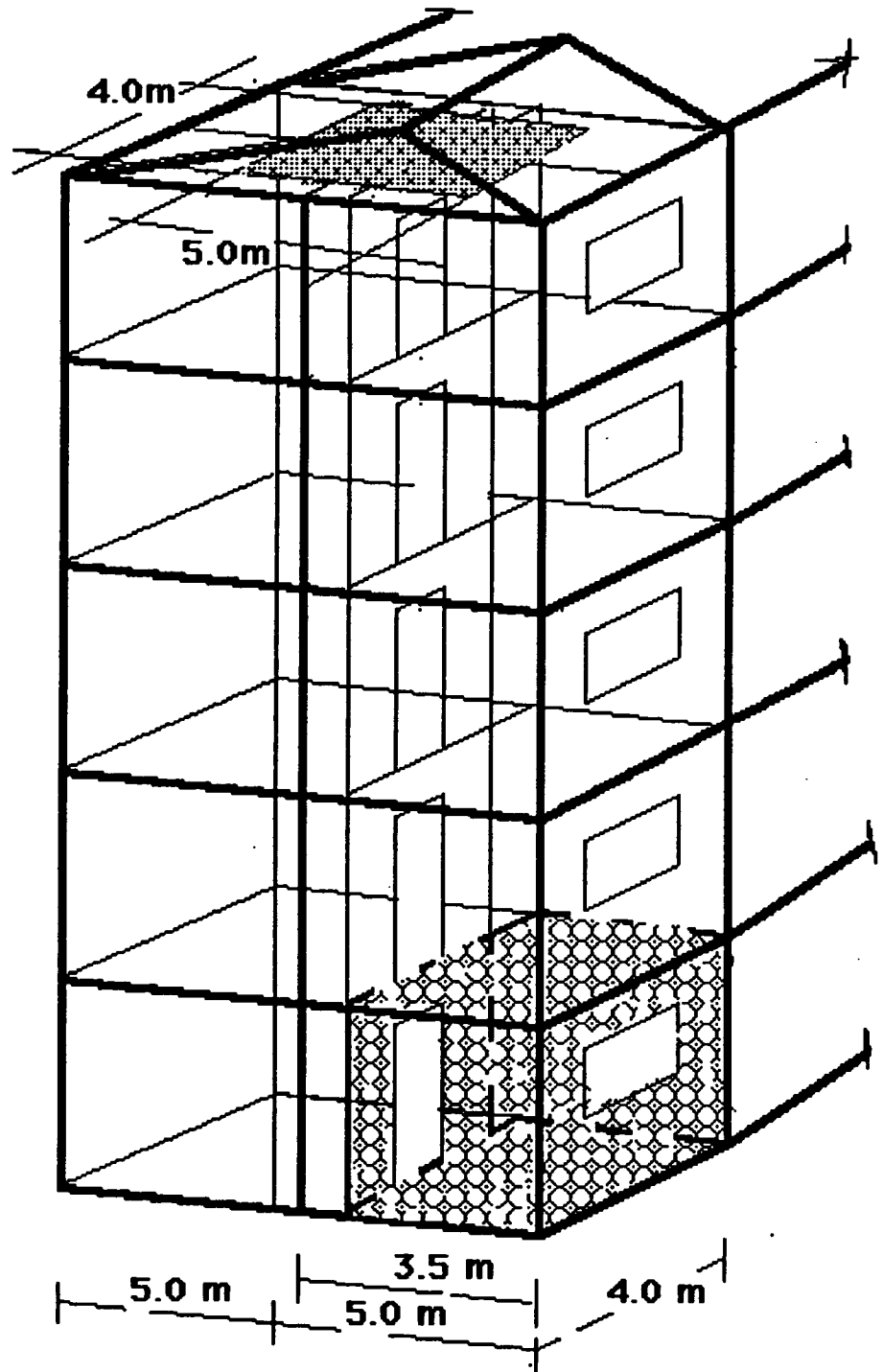
CHAPTER SIX**Design Example****FIRE ENGINEERING DESIGN OF LOAD-BEARING STEEL STRUCTURE IN A FIVE-STOREY OFFICE BUILDING**

The loadbearing frame in a five-storey office building (Fig. 6.1) consists of simply supported I-profile steel girders and H-profile steel columns. The dimensions (internal) of the fire compartment and the sizes of the openings are as shown in Figure 6.2.

Complete evacuation of people from the building in the event of fire cannot be assumed with absolute certainty.

The scope of this exercise is to check whether the columns must be provided with fire insulation. Where necessary the material is chosen and the insulation thickness required determined.

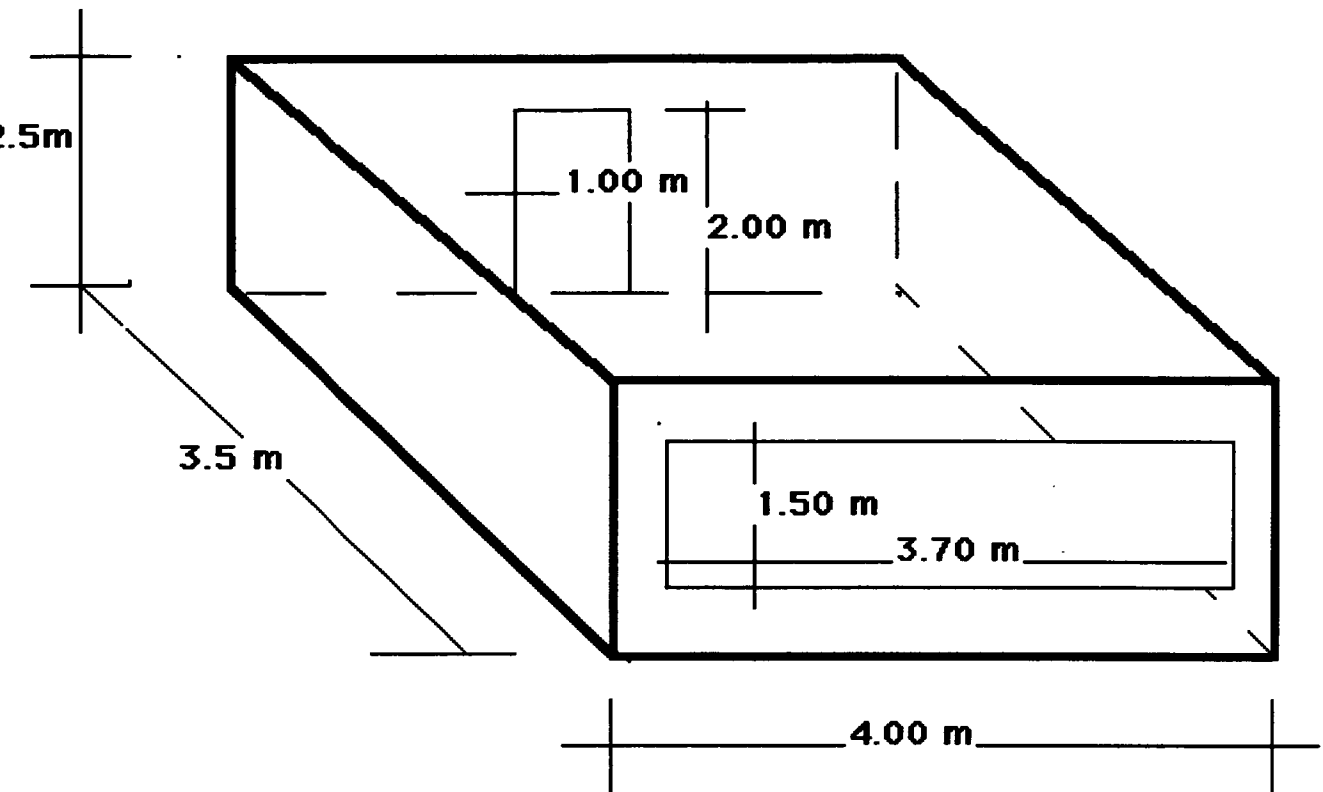
Fig. 6.1 Five storey office building



Compartment of Fire Origin



Roof Load Surface



WALLS

External

surface layers of steel sheeting with intermediate 100mm mineral wool

Internal

steel studs covered on each side with 2*13mm gypsum plaster sheets

Fig. 6.2 Internal dimensions of the fire compartment.

6.1 BS 5950:Part 8: Code of Practice for Fire Resistant Design

A check is to be made with regard to the central columns as well as the columns along the facade on the top and bottom storeys.

Here the structure is assumed to have columns in simple construction where the connections between members are assumed not to develop moments adversely affecting either the structural members or the structure as a whole. The distribution of the forces may be determined assuming that members intersecting at a joint are "pin-ended".

Example

STATIC LOAD

Ref.

Roof loading

Imposed: 0.60 kN/m² (without access)

Dead: 1.20 kN/m²

Factored load: $F_c = 1.4 \times 1.2 + 1.6 \times 0.60 = 2.64 \text{ kN/m}^2$

BS 6399:
Part 3:1988

Floor loading

Imposed: 2.50 kN/m² (offices for general use)

Dead: $h \times 24 = 0.25 \times 24 = 6.00 \text{ kN/m}^2$

where: Basic span/effective depth ratio = 20

$5000/d = 20$, $d = 4000/20 = 200 \text{ mm}$

Steel 20mm for reinforcement

C30, cover = 25mm

Nominal depth of slab = $200 + 10 + 25 = 235 \text{ mm}$

Assume $h = 250 \text{ mm} = 0.25 \text{ m}$

Factored load: $1.4 \times 6.0 + 1.6 \times 2.5 = 12.40 \text{ kN/m}^2$

BS 8110:
Part 1: 1984
Table 3.11

Example (Continue)**SLAB - DISTRIBUTION OF THE LOADING**

Every slab in the structure studied is a two-way slab with $l_y/l_x=1.25$. The reactions at the middle of the

edges of each slab are given below :

$$q_{x_{rm}} = \frac{pl_x}{2.02} \quad (1)$$

$$q_{y_{rm}} = \frac{pl_y}{1.99} \quad (2)$$

**BETON
KALENDER
(1983)**

Top level

$$p=2.64 \text{ kN/m}^2$$

$$\text{from equation (1): } q_{x_{rm}} = 2.64 \times 4.0 / 2.02 = 5.23 \text{ kN/m}$$

$$\text{from equation (2): } q_{y_{rm}} = 2.64 \times 5.0 / 1.99 = 6.63 \text{ kN/m}$$

Bottom level

$$p=12.40 \text{ kN/m}^2$$

$$\text{from equation (1): } q_{x_{rm}} = 12.40 \times 4.0 / 2.02 = 24.56 \text{ kN/m}$$

$$\text{from equation (2): } q_{y_{rm}} = 12.40 \times 5.0 / 1.99 = 31.16 \text{ kN/m}$$

**BS 5950:
Part 1:1985**

BEAMS**TOP LEVEL****Span 5.0m**

Assume 254×146×31 UB

Design strength p_y

$$T=12.7\text{mm} < 16\text{mm}, p_y=275\text{N/mm}^2$$

$$t=7.3\text{mm}$$

Example (Continue)**Ref.**Section class

-Outstand element of compressive flange

$$\frac{b}{t} = 8.49 < 15\epsilon = 15$$

$$\epsilon = 1$$

-Web with neutral axis at mid-depth

$$\frac{d}{t} = 35.9 < 120\epsilon = 120$$

$$\epsilon = 1$$

Non slender section

$$\text{Load } 2 \times 5.23 + 0.31 = 10.77 \text{ kN/m}$$

$$\text{Applied moment } M = \frac{10.77 \times 5.0^2}{8} = 33.66 \text{ kNm}$$

$$\text{Slenderness } \lambda = \frac{5000}{33.5} = 149$$

Lateral torsional buckling resistance moment $M_b = S_x p_b$

$$x = D/T = 251.5 / 8.6 = 29$$

$$\lambda/x = 5.10$$

Slenderness factor

$$v = 0.82$$

Buckling parameter

$$u = 0.9 \text{ (conservative)}$$

$$n = 1$$

TABLE 14
for rolled
I-profile
sections
4.3.7.5
for rolled
I-profile
sections
TABLE 13

Example (Continue)	Ref.
$\lambda_{LT} = 0.9 \times 0.82 \times 149 = 110$	4.3.7.5
$p_b = 109 \text{ N/mm}^2$	TABLE 11
$M_b = 109 \times 395.6 \times 10^{-3} = 43.12 \text{ kNm}$	
Since $M_x = 33.66 \text{ kNm} < M_b$	
Section adequate	
Span 4.0m	
Assume 254 × 102 × 28 UB	
<u>Design strength</u> p_y	
$T = 10 \text{ mm} < 16 \text{ mm}$, $p_y = 275 \text{ N/mm}^2$	TABLE 6
$t = 6.4 \text{ mm}$	
<u>Section class</u>	
- Outstand element of compressive flange	TABLE 7
$\frac{b}{T} = 8.49 < 15\epsilon = 15$	for semi-
$\epsilon = 1$	-compact
- Web with neutral axis at mid-depth	section
$\frac{d}{t} = 35.9 < 120$	
Non slender section	
<u>Load</u> $6.63 \times 2 + 0.28 = 13.54 \text{ kN/m}$	
<u>Applied moment</u> $M = \frac{13.54 \times 4.0^2}{8} = 27 \text{ kNm}$	

Example (Continue)	Ref.
Slenderness $\lambda = \frac{4000}{22.2} = 180$	
<u>Lateral torsional buckling resistance moment</u>	4.3.7.3.
$\lambda/x = 6.9$	
$v = 0.73$	TABLE 14 for I-profile sections
$u = 0.9$ (conservative)	4.3.7.5. for rolled I-sections
$n = 1$	TABLE 13
$\lambda_{LT} = 0.9 \times 0.73 \times 180 = 118.26$	4.3.7.5
$p_b = 99 \text{ N/mm}^2$	TABLE 11
$M_b = 99 \times 353.4 \times 10^{-3} = 35.0 \text{ kNm}$	
Since $M = 27.0 \text{ kNm} < M_b$	
Section adequate	
ANY OTHER LEVEL	
Span 5.0 m	
Assume 406 × 178 × 67 UB	
<u>Design strength p_y</u>	TABLE 6
$T = 14.3 \text{ mm} < 16 \text{ mm}$, $p_y = 275 \text{ N/mm}^2$	
$t = 8.8 \text{ mm}$	

Example (Continue)**Ref.**Section class

-Outstand element of compressive flange

$$\frac{b}{T} = 6.25 < 15\epsilon = 15$$

$$\epsilon = 1$$

OR

-Web with neutral axis at mid-depth

$$\frac{d}{t} = 41.0 < 120\epsilon = 120$$

$$\epsilon = 1$$

Non slender section

Load $24.56 \times 2 + 0.67 = 49.79 \text{ kN/m}$

Applied moment: $M = \frac{49.79 \times 5.0^2}{8} = 155.59 \text{ kNm}$

Slenderness $\lambda = \frac{5000}{40.0} = 125$

Lateral torsional buckling resistance moment

$$x = D/T = 409.4/14.3 = 28.63$$

$$\lambda/x = 4.5$$

$$v = 0.84$$

$$u = 0.9$$

TABLE 7

for semi-compact sections

BS 5950:**Part 1: 1985****4.3.7.3****TABLE 14**

for I-profile sections

4.3.7.5

for rolled I-sections

Example (Continue)**Ref.**

$$n=1$$

$$\lambda_{LT}=0.9 \times 0.84 \times 125 = 94.5$$

$$p_b=134 \text{ N/mm}^2$$

$$M_b=134 \times 1346 \times 10^{-3}=180.36 \text{ kNm}$$

$$\text{Since } M=155.59 \text{ kNm} < M_b$$

Section adequate

TABLE 13**4.3.7.5****TABLE 11****Span 4.0m**

Assume 406 × 178 × 54 UB

Design strength p_y

$$T=10.9\text{mm} < 16\text{mm}, \quad p_y=275\text{N/mm}^2$$

TABLE 6

$$t=7.6\text{mm}$$

Section class

- Outstand element of compressive flange

$$\frac{b}{T}=8.15 < 15\epsilon=15$$

TABLE 7**for I-profile****sections**

$$\epsilon=1$$

OR

-Web with neutral axis at mid-height

$$\frac{d}{T}=47.4 < 120\epsilon=120$$

$$\epsilon=1$$

Non slender section

Example (Continue)**Ref.**Load $31.16 \times 2 + 0.54 = 62.86 \text{ kN/m}$ Applied moment $M = \frac{62.86 \times 4.0^2}{8} = 125.72 \text{ kNm}$ Slenderness $\lambda = \frac{4000}{38.5} = 103.9$ Lateral torsional buckling resistance moment

$$x = D/T = 402.6 / 10.9 = 36.94$$

$$\lambda/x = 2.8$$

4.3.7.3

$$v = 0.92$$

**TABLE 14
for I-profile
sections**

$$u = 0.9 \text{ (conservative)}$$

**4.3.7.5
for rolled
I-sections**

$$n = 1$$

$$\lambda_{LT} = 0.9 \times 0.92 \times 103.9 = 86$$

**TABLE 13
4.3.7.5**

$$p_b = 154 \text{ N/mm}^2$$

TABLE 11

$$M_b = 154 \times 1048 \times 10^{-3} = 161 \text{ kNm}$$

$$\text{Since } M = 125.72 < M_b$$

Section adequate

Example (Continue)**Ref.****COLUMN DESIGN CHECK****Loads****TOP LEVEL****Dead loads - not factored**Floor load 1.20 kN/m²

$$q_{x_{rm}} = 1.20 \times 4.0 / 2.02 = 2.38 \text{ kN/m}$$

$$q_{y_{rm}} = 1.20 \times 5.0 / 1.99 = 3.02 \text{ kN/m}$$

The reaction at each end of the beam:

Beam with span of 4.0m

$$2 \times \frac{(3.02 + 0.28) \times 4.0}{2} = 12.64 \text{ kN}$$

Beam with span of 5.0m

$$2 \times \frac{(2.38 + 0.31) \times 5.0}{2} = 12.68 \text{ kN}$$

Imposed loads - not factoredFloor load 0.60 kN/m²

$$q_{x_{rm}} = 0.60 \times 4.0 / 2.02 = 1.19 \text{ kN/m}$$

$$q_{y_{rm}} = 0.60 \times 5.0 / 1.99 = 1.51 \text{ kN/m}$$

The reaction at each end of a beam:

Beam with span of 4.0m

$$2 \times \frac{1.51 \times 4.0}{2} = 6.04 \text{ kN}$$

Beam with span of 5.0m

$$2 \times \frac{1.19 \times 5.0}{2} = 5.95 \text{ kN}$$

Example (Continue)**Ref****Any other level****Dead loads**-not factoredFloor load 6.0kN/m^2

$$q_{x_{rm}} = 6.0 \times 4.0 / 2.02 = 11.88\text{kN/m}$$

$$q_{y_{rm}} = 6.0 \times 5.0 / 1.99 = 15.08\text{kN/m}$$

The reaction at its end of a beam:

Beam with span of 4.0m

$$2 \times \frac{(15.08 + 0.54) \times 4.0}{2} = 61.4 \text{ kN}$$

Beam with span of 5.0m

$$2 \times \frac{(11.88 + 0.67) \times 5.0}{2} = 61.08\text{kN}$$

Imposed loads-not factoredFloor load 2.5kN/m^2

$$q_{x_{rm}} = 2.5 \times 4.0 / 2.02 = 4.95\text{kN/m}$$

$$q_{y_{rm}} = 2.5 \times 5.0 / 1.99 = 6.28\text{kN/m}$$

The reaction of each end of a beam:

Beam with span of 4.0m

$$2 \times \frac{6.28 \times 4.0}{2} = 25.12 \text{ kN}$$

Beam with span of 5.0m

$$2 \times \frac{4.95 \times 5.0}{2} = 24.75 \text{ kN}$$

Example (Continue)**Ref.****6.1.1. Centre Column****TOP LEVEL****Design in ambient environment**Design load

$$F_c = 1.4 \times 2 \times (12.64 + 12.68) + 1.6 \times 2 \times (6.04 + 5.95) + 1.4 \times (0.23 \times 3) = 110.23 \text{ kN}$$

Assume 152 × 152 × 23 UC

Design strength p_y

$$T = 6.8 \text{ mm} < 16 \text{ mm}, \quad p_y = 275 \text{ N/mm}^2$$

$$t = 6.1 \text{ mm}$$

Section class

-Outstand element of compression flange

$$\frac{b}{T} = 11.2 < 15 \varepsilon = 15$$

$$\varepsilon = 1$$

-Web generally

$$\frac{d}{t} = 20.2$$

$$R = \frac{F_c}{p_y} = \frac{110.23}{29.8 \times 275} \times 10 = 0.14 < 0.5, \quad \text{positive}$$

$$\frac{d}{t} \leq \frac{120 \varepsilon}{1 + 1.6 R} = 98.0$$

Table 6**Table 7
for semi-compact sections****3.5.4**

Example (Continue)**Ref.**

-Web subject to compression throughout

$$\frac{d}{t} = 12.4 < 39\epsilon = 39$$

Table 7
for semi-
compact
sections

Section adequate

Effective length $L_E = 0.85 L$

$$\text{Slenderness } \lambda = \frac{3.0 \times 0.85 \times 10^2}{3.68} = 69.29$$

Check for compressionCompressive strengthH-section and for thickness ≤ 40 mm, 27cFor $\lambda=69.29$ and $p_y=275$ N/mm²

$$p_c = 183 \text{ N/mm}^2$$

Table 25
Table 27c
for yy axis
of buckling

Compressive resistance

$$P_c = A_g p_c = 29.8 \times 183 \times 10^{-1} = 545.34 \text{ kN}$$

Since, $P_c = 545 \text{ kN} > F_c$

4.7.4
for plastic,
compact,
semi-
compact
sections

Example (Continue)**Ref.**Check for local buckling

The following relationship must be satisfied:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

4.8.3.2where: $F_c = 115.70 \text{ kN}$

$$A_g = 29.8 \text{ cm}^2$$

$$p_y = 275 \text{ N/mm}^2$$

$$M_x = M_y = 0$$

4.7.6(3)

$$\text{So, } \frac{F_c}{A_g p_y} = \frac{110.23}{29.8 \times 275} \times 10 = 0.14 < 1$$

Section adequate

Check for overall buckling**4.7.7**

The requirements for overall buckling will be satisfied in this context by the "Simplified approach" (nominal moments applied).

$$\frac{F_c}{A_g p_c} + \frac{M_x}{M_{bs}} + \frac{M_x}{p_y Z_y} \leq 1$$

4.8.3.3.1where: $F_c = 110.23 \text{ kN}$

$$M_x = M_y = 0$$

$$A_g = 29.8 \text{ cm}^2$$

$$p_c = 183 \text{ N/mm}^2$$

$$m = 1$$

$$\text{So, } \frac{F_c}{A_g p_c} = \frac{110.23}{29.8 \times 183} \times 10 = 0.14 < 1$$

Section adequate

Example (Continue)	Ref.
Design in a fire environment	BS 5950:
<u>Design load</u>	Part 8:1990
Assume that the whole imposed load is non-permanent $F_c = 1.0 \times 2 \times (12.64 + 12.68) + 0.8 \times 2 \times (6.04 + 5.95) + 1.0 \times (0.23 \times 3.0) = 70.51 \text{ kN}$	
<u>Load ratio</u>	
$R = \frac{70.51}{29.8 \times 183} \times 10 = 0.13 < 0.2, A_g = 29.8 \text{ cm}^2, p_c = 183 \text{ N/mm}^2$	4.4.2.3
<u>Limiting temperature</u>	Table 5
$\lambda = 69.29 < 70$	
$\theta_L = 710^\circ\text{C}$	
<u>Design temperature</u>	
<u>Case 1. Column exposed from four sides.</u>	
For flange thickness = 6.8 mm	
<i>For fire resistance period of 90 min, 1006°C</i>	
<i>For fire resistance period of 60 min, 945°C</i>	
<i>For fire resistance period of 30 min, 841°C</i>	Table 6
So, $\theta_D > \theta_L$	
Protection required	
<u>Case 2. Column built next to a wall</u>	
For flange thickness 6.8 mm	
<i>For fire resistance period of 90 min, 1000°C</i>	Table 7
<i>For fire resistance period of 60 min, 940°C</i>	
<i>For fire resistance period of 30 min, 810°C</i>	

Example (Continue)**Ref.**

$$\theta_D > \theta_L.$$

Protection required

The design temperatures derived from Table 7 refer to beams and generally to heating from three sides. Because the furnace for beams has different thermo-properties than the one for columns, the values above may be slightly different from the ones if the table referred to columns.

Case 3. Column built into a wall

Because of the large temperature gradients developed, more exact method of calculation has to be used.

BOTTOM LEVEL**Design in ambient temperature****Design load**

$$F_c = 1.4 \times 2 \times \{(12.64 + 12.68) + (61.4 + 61.08) \times 4\} + 1.6 \times 0.6 \times 2 \\ \times \{(6.04 + 5.95) + (25.12 + 24.75) \times 4\} + 1.4 \times \{(0.86 \times 3.5) + \\ +(0.23 \times 3.0) \times 4\} = 1856.77 \text{ kN}$$

Assume 203×203×86 UC

Design strength p_y

For $T=20.5\text{mm}$ and $t=13.0\text{mm}$

Because $T > 16\text{mm}$ and $T < 40\text{mm}$, $p_y = 265\text{N/mm}^2$

TABLE 6

Example (Continue)**Ref.**Section class

-Outstand element of compression flange

$$\frac{b}{T} = 5.09 < 15\varepsilon = 15.30$$

$$\varepsilon = (275/p_y)^{1/2} = 1.02$$

TABLE 7
for semi-
compact
sections

-Web generally

$$\frac{d}{t} = 12.4$$

$$R = \frac{F_c}{p_y} = \frac{1856.77}{110.1 \times 265} \times 10 = 0.64 > 0.5, \text{ positive}$$

$$12.4 = \frac{d}{t} \leq (93 - 54R)\varepsilon = 58$$

3.5.4

OR

-Web subject to compression throughout

$$\frac{d}{t} = 12.4 < 39\varepsilon = 39.78$$

Non slender section

TABLE 7
for semi-
compact
sections

Check for compression

Effective length $L_E = 0.85 L$

4.7.2

Slenderness $\lambda = \frac{3.5 \times 0.85 \times 10^2}{5.32} = 55.92$

4.7.3

$\lambda < 180$ max slenderness for members

4.7.3.2

resisting loads other than wind loads

H - section and for thickness ≤ 40 mm, Table 27c

Table 25

Example (Continue)**Ref.**

M_{cx} = the moment capacity about the major axis in the
absence of axial load

M_{cy} = the moment capacity about the minor axis in the
absence of axial load

$$\text{So, } \frac{F_c}{A_g p_y} = \frac{1856.77}{110.1 \times 265} \times 10 = 0.64 < 1$$

Section adequate

Check for overall buckling

The requirements for **overall buckling** will be satisfied in this context by the "Simplified Approach" (nominal moments applied):

4.7.7

$$\frac{F_c}{A_g p_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{p_y Z_y} \leq 1.0$$

4.8.3.3.1

where: $F_c = 1856.77 \text{ kN}$

$M_x = M_y = 0$, the loads are balanced about the xx and yy axis.

$$A_g = 110.1 \text{ cm}^2$$

$$p_c = 202 \text{ N/mm}^2$$

$m =$ equivalent uniform moment factor $= 1$

$$\text{So, } \frac{F_c}{A_g p_c} = \frac{1856.77}{110.1 \times 202} \times 10 = 0.84 < 1$$

Section adequate

Example (Continue)**Ref.**

Compressive strength For $\lambda=55.92$ and $p_y=265 \text{ N/mm}^2$,
 $p_c=202 \text{ N/mm}^2$

TABLE 27c
for yy axis
of buckling

Compression resistance

$$P_c = A_g p_c = 110.1 \times 202 \times 10^{-1} = 2224 \text{ kN}$$

4.7.4

Since, $P_c > F_c$,

for plastic,
compact,
semi-
compact
sections

Section adequate

For axially loaded compression members with moments separate checks are required for local capacity and overall buckling.

Check for local buckling

The following relationship must be satisfied:

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

4.8.3.2

where: $F_c = 1856.77 \text{ kN}$

$$A_g = 110.1 \text{ cm}^2$$

$$p_y = 265 \text{ N/mm}^2$$

$$M_x = M_y = 0$$

4.7.6 (3)

Example (Continue)**Ref.****Design in a fire environment****BS 5950:****Design load****Part 8:1990**

Assume that the whole imposed load is non-permanent

$$F_c = 1.0 \times 2 [(12.64 + 12.68) + (61.4 + 61.08) \times 4] +$$

$$0.8 \times 0.6 \times 2 \times [(6.04 + 5.95) + (25.12 + 24.75) \times 4] +$$

$$1.0 [(0.23 \times 3.5) + (0.86 \times 3.0) \times 4] = 1239.26 \text{ kN.}$$

Load ratio**4.4.2.3**

$$R = \frac{F_f}{A_g p_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{p_y Z_y}$$

where:

A_g = gross area;

p_c = compressive strength;

p_y = design strength of steel;

z_y = elastic modulus about the minor axis;

M_b = buckling resistance moment (lateral torsional);

F_f = axial load at the fire limit state;

M_{fx} = max moment about the major axis at the fire limit state;

M_{fy} = max moment about the minor axis at the fire limit state.

$$\text{So, } R = \frac{1239.26}{110.1 \times 202} \times 10 = 0.56$$

Limiting temperature

$$\lambda = 55.92 < 70 \text{ and } R = 0.56$$

$$\theta_L = 560^\circ\text{C}$$

Table 5

Example (Continue)**Ref.**Design temperatureCase 1: Column exposed from four sides

For flange thickness 20.5 mm and

for fire resistance period of 90 min, $\theta=950^{\circ}\text{C}$

for fire resistance period of 60 min, $\theta=858^{\circ}\text{C}$

for fire resistance period of 30 min, $\theta=661^{\circ}\text{C}$

So, $\theta_D > \theta_L$,

Protection required

Table 7Case 2: Column exposed from three sides

For flange thickness 20.5 mm and

for fire resistance period of 90 min, $\theta=1000^{\circ}\text{C}$

for fire resistance period of 60 min, $\theta=928^{\circ}\text{C}$

for fire resistance period of 30 min, $\theta=716^{\circ}\text{C}$

So, $\theta_D > \theta_L$,

Protection required

Table 7

The design temperatures derived from Table 7 refer to beams.

Because the furnace for beams has different thermo-properties than the one for columns, the values above may be slightly from the the ones if the Table referred to columns.

Case 3: Column built into a wall

Because of the large temperature gradients developed, more exact method of calculation has to be used.

Example (Continue)**Ref.****6.1.2 Edge Column****TOP LEVEL****Design in ambient environment****Span 5.0m** Load=10.77kNm

Assume 254 × 146 × 31 UB

Check as for centre columns

Span 4.0m

Assume 254 × 102 × 22 UB

Design strength p_y

$$T=6.8\text{mm}<16\text{mm} \quad p_y=275 \text{ N/mm}^2$$

$$t=5.8\text{mm}$$

Section Class

- Outstand element of compressive flange

$$\frac{b}{T}=7.47, \quad \frac{b}{T}<15\varepsilon=15$$

$$\varepsilon=1$$

- Web with neutral axis at mid-depth

$$\frac{d}{t}=38.8<120$$

Non slender section

Load

$$6.63+0.22=6.85\text{kN/m}$$

Applied Moment

$$M=\frac{6.85 \times 4.0^2}{8}=13.7 \text{ kNm}$$

Slenderness

$$\lambda=\frac{4000}{20.5}=195.12$$

Table 6**Grade 43****Table 7****for semi-compact sections**

Example (Continue)**Ref.**Lateral torsional buckling resistance moment**4.3.7.3**

$$x = D/T = 254 / 6.8 = 37.35$$

$$\lambda/x=5.22, N=0.5$$

Slenderness factor $v=0.81$ **Table 14****for I-profile sections**

$$u = 0.9 \text{ (conservative)}$$

4.3.7.5**or rolled I-sections**

$$n = 1$$

Table 13

$$\lambda_{LT}=0.9 \times 0.81 \times 195.12 = 142.24$$

4.3.7.5

$$p_b=73\text{N/mm}^2$$

Table 11

$$M_b=73 \times 261.9 \times 10^{-3}=19.12\text{kNm}$$

Since $M_b > M$

Section adequate

Any other level**Span 5.0 m** Load=49.79 kN/m

Assume 406 × 178 × 67 UB

Check as for centre columns

Span 4.0mDesign strength p_y

$$T=14.3\text{mm} < 16\text{mm}$$

$$p_y=275 \text{ N/mm}^2$$

Table 6

$$t=8.8\text{mm}$$

Example (Continue)**Ref.**Section Class**Table 7**- Outstand element of compressive flange**for semi-**

$$\frac{b}{T} = 6.25 < 15$$

compact- Web with neutral axis at mid-depth**sections**

$$\frac{d}{t} = 41.0 < 120$$

Load: $31.16 + 0.39 = 31.55 \text{ kN/m}$

$$\text{Applied moment: } M = \frac{31.55 \times 4.0^2}{8} = 63.10 \text{ kNm}$$

$$\text{Slenderness } \lambda = \frac{4000}{40} = 100$$

Lateral torsional buckling resistance moment**4.3.7.3.**

$$D/T = 409.4/14.3 = 28.63$$

$$\lambda/x = 3.49$$

Table 14Slenderness factor $v = 0.91$ **for I-profile****sections**Buckling parameter $u = 0.9$ (conservative)**4.3.7.5**

$$n = 1$$

Table 13

$$\lambda_{LT} = 0.9 \times 0.91 \times 100 = 81.90$$

4.3.7.5

$$p_b = 159.50 \text{ N/mm}^2$$

Table 11

$$M_b = 159.50 \times 1346 \times 10^{-3} = 214.69 \text{ kNm}$$

Since $M_b > M$

Section adequate

Example (Continue)**Ref.****COLUMNS****TOP LEVEL****Design in ambient environment**Design load

$$F_c = 1.4 \times (2 \times 6.48 + 12.68) + 1.6 \times (2 \times 3.02 + 5.95) + 1.4 \times (0.23 \times 3) = 56.05 \text{ kN}$$

Eccentricity of connections

$$e_x = 152.4/2 + 100 = 176.2 \text{ mm}$$

$$e_y = 6.1/2 + 100 = 103.05 \text{ mm}$$

4.7.6(3)Nominal moments

$$M_{ex} = (1.4 \times 12.68 + 1.6 \times 5.95) \times 176.2 \times 10^{-3} = 4.81 \text{ kNm}$$

The loads are balanced about the y-y axis, $M_{ey} = 0$

Design strength

$$p_y = 275 \text{ N/mm}^2$$

Table 6Section class

Check as for centre columns

Non slender section

Check for local buckling**4.8.3.2**

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} =$$

$$= \frac{56.05}{29.8 \times 275} + \frac{4.84}{275 \times 184.3 \times 10^{-3}} = 0.10 < 1$$

Section adequate

Example (Continue)**Ref.**Check for overall buckling

$$\frac{F_c}{A_g p_c} + \frac{M_x}{M_b} + \frac{M_y}{p_y z_y} < 1$$

4.8.3.3.1

$$\text{where: } \lambda_{LT} = 0.85 L/r_y = 0.85 \times 300 / 3.68 = 69.29$$

$$p_b = 188 \text{ N/mm}^2$$

Table 11

$$\text{So, } \frac{56.05}{29.8 \times 275} + \frac{4.84}{188 \times 184.3 \times 10^{-3}} = 0.15 < 1$$

Section adequate

Design in a fire environment

BS 5950:

Design load

Part 8:1990

$$F_c = 1.0 \times (2 \times 6.48 + 12.68) + 0.8 \times (2 \times 3.02 + 5.95) + 1.0 \times (0.23 \times 3)$$

$$= 35.92 \text{ kN}$$

Nominal moments

$$M_{fx} = (1.0 \times 12.68 + 0.8 \times 5.95) \times 176.2 \times 10^{-3} = 3.07 \text{ kNm}$$

Load ratio

$$R = \frac{F_f}{A_g p_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{p_y z_y}, \quad M_{fy} = 0$$

4.4.2.3

$$R = \frac{35.92}{29.8 \times 183} + \frac{3.07}{188 \times 184.3 \times 10^{-3}} = 0.10$$

Limiting temperature

$$\lambda = 69.29 < 70 \text{ and } R = 0.10, \theta_L = 710^\circ\text{C}$$

Table 5

Example (Continue)

Ref.

Design temperatureCase 1: Column exposed from three sides

Table 6

For flange thickness 6.8mmfor fire resistance period of 90min, $\theta_D=1000^\circ\text{C}$ for fire resistance period of 60min, $\theta_D=940^\circ\text{C}$ for fire resistance period of 30min, $\theta_D=810^\circ\text{C}$ Since, $\theta_L < \theta_D$

Protection required

The design temperatures derived from Table 7 refer to beams. Because the furnace for beams has different thermo-properties than the one for columns, the values above may be slightly different from the ones if the table referred to columns.

Case 2: Column built into a wall

Because of the large temperature gradients developed, more exact method of calculation has to be used.

BOTTOM LEVEL**Design in ambient environment**Design load

$$F_c = 1.4 \times \{2 \times [6.48 + 4 \times 30.94] + 12.68 + 4 \times 61.08\} +$$

$$+ 1.6 \times 0.6 \times \{2 \times [3.02 + 4 \times 12.56] + [5.95 + 4 \times 24.75]\} +$$

$$+ 1.4 \times [(0.68 \times 3.5) + (0.23 \times 3.0) \times 4] = 934.68 \text{ kN}$$

Example (Continue)**Ref.**Eccentricity of connections

$$e_x = 211.15 \text{ mm}$$

$$e_y = 106.5 \text{ mm}$$

Nominal moments

$$M_{ex} = (1.4 \times 61.08 + 1.6 \times 24.75) \times e_x \times 10^{-3} = 26.42 \text{ kNm}$$

The loads are balanced about the y-y axis $M_{ey} = 0$

Design strength

$$p_y = 265 \text{ N/mm}^2$$

Table 6Section class

Check as for centre columns

Non slender section

Check for local buckling

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

4.8.3.2

where: $F_c = 934.68 \text{ kN}$

$$M_{cx} = p_y S_x = 265 \times 978.8 \times 10^{-3} = 259.38 \text{ kNm}$$

$$\text{So, } \frac{934.68}{110.1 \times 265} + \frac{26.42}{259.38} = 0.13 < 1$$

Section adequate

Example (Continue)**Ref.**Check for overall buckling

$$\frac{F_c}{A_g p_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{p_y z_y} \leq 1$$

4.8.3.3.1where: $F_c = 1013.83 \text{ kN}$

$$\lambda_{LT} = \frac{0.85L}{r_y} = \frac{0.85 \times 350}{5.32} = 55.92$$

Table 11

$$p_b = 219 \text{ N/mm}^2$$

$$M_{bs} = 219 \times 978.8 \times 10^{-3} = 214.36 \text{ kNm}$$

$$\text{Therefore, } \frac{934.68}{110.1 \times 202} \times 10 + \frac{26.42}{259.38} = 0.52 < 1$$

Section adequate

Design in a fire environment**BS 5950:****Part 8:1990**Design load

$$F_c = 1.0 \times \{2[6.48 + 30.94 \times 4] + 12.68 + 61.08 \times 4\} +$$

$$0.8 \times 0.6 \{2[3.02 + 12.56 \times 4] + [5.95 + 24.75 \times 4]\} +$$

$$1.0 \{[(0.86 \times 3.5) + (0.23 \times 3.0) \times 4]\} = 624.13 \text{ kN}$$

Load ratio**4.4.2.3**

$$R = \frac{F_f}{A_g p_c} + \frac{M_{fx}}{M_b} + \frac{M_{fy}}{p_y z_y}$$

where: $F_f = 624.13 \text{ kN}$

$$A_g = 110.1 \text{ cm}^2$$

$$p_c = 202 \text{ N/mm}^2$$

Example (Continue)**Ref.**

$$M_{fx} = (1.0 \times 61.08 + 0.8 \times 24.75) e_x \times 10^{-3} = 17.08 \text{ kNm}$$

$$\lambda_{LT} = \frac{0.5L}{r_y} = \frac{0.5 \times 350}{5.32} = 33$$

$$\rho_b = 265 \text{ N/mm}^2$$

$$M_{bs} = 265 \times 978.8 \times 10^{-3} = 259.38 \text{ kNm}$$

$$\text{Therefore, } R = \frac{624.13}{110.1 \times 202} \times 10 + \frac{17.08}{259.38} = 0.346$$

Limiting temperature

For $\lambda = 55.90 < 70$ and $R = 0.34$, $\theta_L = 640^\circ\text{C}$

Table 5Design temperatureCase 1: Column exposed from three sides

For flange thickness 14.3mm and

for fire resistance period of 90min, $\theta_D = 950^\circ\text{C}$

for fire resistance period of 60min, $\theta_D = 877^\circ\text{C}$

for fire resistance period of 30min, $\theta_D = 709^\circ\text{C}$

Since, $\theta_L < \theta_D$

Protection required

Table 6

The design temperatures derived from Table 7 refer to beams. Because the furnace for beams has different thermo-properties than the one for columns, the values above may be slightly different from the ones if the table referred to columns.

Example (Continue)**Ref.**Case 2: Column built into a wall

Because of the large temperature gradients developed, more exact method of calculation has to be used.

Example (Continue)**Ref.****6.2 European Recommendations for the Fire
Safety of Steel Structures***[Eur3,
1990]***6.2.1 Centre Column****BOTTOM LEVEL****Design in ambient environment**

203 × 203 × 86 UC

 $F_c = 1856.77 \text{ kN}$ F_e 430 $t \leq 40 \quad \sigma_y = 275 \text{ N/mm}^2$ **Section classification - Plastic section (Category 1)***Table 1**Appendix B***-Web subject to compression**

$$\frac{d}{t} = \frac{160.8}{13.0} = 12.37 < 33\epsilon = 30.36$$

*Table 3**Sheet 1**section**class 1**Appendix B***-Outstand Flange subject to compression**

$$\frac{c}{t} = \frac{208.8}{2 \times 20.5} = 5.09 < 10\epsilon = 9.20$$

*Table 3**Sheet 3**section**class 1-**-rolled**sections**Appendix B*

Example (Continue)**Ref.**

where: $\varepsilon = \sqrt{\frac{235}{\sigma_y}} = 0.92$

Non slender section

Slenderness

$$\lambda = \frac{L_E}{\pi} \sqrt{\frac{\sigma_y A}{EI}}$$

$$\lambda_y = \frac{0.85 \times 350}{\pi} \sqrt{\frac{2.75 \times 110.1}{2100 \times 9462}} = 0.37$$

$$\lambda_z = \frac{0.85 \times 350}{\pi} \sqrt{\frac{2.75 \times 110.1}{2100 \times 3119}} = 0.64$$

Check for overall buckling

For I-profile cross sections

$$\frac{h}{b} = \frac{222.3}{208.8} = 1.06 < 1.2$$

$$t_f = 20.5 \text{ mm} < 100 \text{ mm}$$

Curve b for buckling about yy

Curve c for buckling about zz

$$\text{For } \lambda_y = 0.37 \quad x = 0.9413$$

$$\lambda_z = 0.64 \quad x = 0.7672$$

$$N_{cr_y} = x N_{pl} = 0.9413 \times 2650 \times 110.1 = 274638 \text{ kp}, \quad 1 \text{ kN} = 100 \text{ kp}$$

$$N_{cr_z} = 0.7672 \times 2650 \times 110.1 = 223842 \text{ kp}$$

$$N_y \leq \gamma N_{cr} = 0.9 \times 274638 = 247174 \text{ kp}$$

$$N_z \leq \gamma N_{cr} = 0.9 \times 223842 = 201458 \text{ kp}$$

$$N = 185677 \text{ kp}$$

$$N < \gamma N_{cr}$$

Table 4**Appendix B****Table 2****Appendix B****5.5.1.1**

Example (Continue)**Ref.**

Section adequate

Section factor

$$F/V = \frac{2h+4b-2d}{\text{cross sectional area}}$$

$$= \frac{2 \times 22.23 + 4 \times 20.88 - 2 \times 1.3}{110.1} = 1.14 \text{ cm}^{-1} =$$

$$= 1.14 \text{ cm}^{-1} = 114 \text{ m}^{-1}$$

Design in a fire environment*for fire resistance period of 90min*Temperature of steel column $\theta_s = 998^\circ\text{C}$ **[ECCS,
1983]****Table 5****Appendix B**Corresponding yield stress

$$\psi = \frac{\sigma_{y \theta}}{\sigma_{y 20}} = \frac{108 \left(1 - \frac{\theta_s}{1000}\right)}{\theta_s - 440} = 0.00039$$

$$\sigma_{y \theta} = \psi \sigma_{y 20} = 0.00039 \times 265 = 0.10 \text{ N/mm}^2$$

2.3.3Buckling load at the temperature 998°C

$$P_u = 201458 \text{ kp}$$

$$P_{u \theta} = P_u \frac{\sigma_{y \theta}}{\sigma_y} = 201458 \times 0.00039 = 78.57 \text{ kp}$$

Example (Continue)**Ref.**Correction factor $k=0.85$ **5.4.3**Admissible load under fire conditions

$$P_u = \frac{P_{u\theta}}{k} = 92.44 \text{kp}$$

Since $P_u < N = 1239 \text{kN}$ **PROTECTION REQUIRED***for fire resistance period of 60min***Table 5**Temperature of steel column $\theta = 932^\circ\text{C}$ **Appendix B**Corresponding yield stress

$$\psi = 0.01493$$

2.3.3

$$\sigma_{y\theta} = 3.96 \text{N/mm}^2$$

Buckling load at the temperature 932°C

$$P_{u\theta} = 201458 \times 0.01493 = 3008 \text{kp}$$

Since $P_{u\theta} < N$ **PROTECTION REQUIRED***for fire resistance period of 30min*Temperature of steel column $\theta = 790^\circ\text{C}$ **Table 5****Appendix B**Corresponding yield stress

$$\psi = 0.06480$$

2.3.3

$$\sigma_{y\theta} = 17.17 \text{N/mm}^2$$

Example (Continue)**Ref.**Buckling load at the temperature 790°C

$$P_{u\theta} = 13054 \text{ kP}$$

Since $P_{u\theta} < N$

PROTECTION REQUIRED

TOP LEVEL**[Eur3,****Design in ambient environment****1990]**

152 × 152 × 23 UC,

$$F_c = 110.23 \text{ kN}$$

F_e 430

$$t \leq 40 \quad \sigma_y = 275 \text{ N/mm}^2$$

Table 1**Appendix B**Section classification (Category 3)

-Web subject to compression

$$\frac{d}{T} = \frac{123.4}{6.1} = 20.23 < 33\epsilon = 30.36$$

Table 3**Sheet 1****section****class 1****Appendix B**

-Outstand flange subject to compression

$$\frac{c}{t} = \frac{152.4}{2 \times 6.8} = 11.21 < 15\epsilon = 13.80$$

Table 3**Sheet 3****section****class 1****Appendix B**

Non slender section

Example (Continue)**Ref.**Slenderness

$$\lambda_y = \frac{L_E}{\pi} \sqrt{\frac{\sigma_y A}{EI}} = \frac{0.85 \times 300}{\pi} \sqrt{\frac{2.75 \times 29.8}{2100 \times 1263}} = 0.45$$

$$\lambda_z = \frac{0.85 \times 300}{\pi} \sqrt{\frac{2.75 \times 29.8}{2100 \times 403}} = 0.80$$

Check for overall buckling

$$\frac{h}{b} = \frac{152.4}{152.4} = 1 < 1.2$$

$$t_f = 6.8 \text{ mm} < 100 \text{ mm}$$

Curve b for buckling about yy

Curve c for buckling about zz

$$\text{For } \lambda_y = 0.45 \quad x = 0.905$$

$$\text{For } \lambda_z = 0.80 \quad x = 0.662$$

$$N_{cr_y} = x N_{pl} = 0.905 \times 2750 \times 29.8 = 74164.75 \text{ kp}$$

$$N_{cr_z} = 0.662 \times 2750 \times 29.8 = 54250.90 \text{ kp}$$

$$N_y \leq \gamma N_{cr_x} = 0.9 \times 74164.75 = 66748.28 \text{ kp}$$

$$N_z \leq \gamma N_{cr_y} = 0.9 \times 54250.90 = 48825.81 \text{ kp}$$

$$N = 11023 \text{ kp}$$

$$N < \gamma N_{cr}$$

Section adequate

Section factor

$$F/V = 30.27 \text{ m}^{-1}$$

Table 4**Appendix B****Table 2****Appendix B****5.5.1.1**

Example (Continue)**Ref.****Design in a fire environment****ECCS,***for fire resistance period of 90min***1983]**Temperature of steel column**Table 5**

$$\theta_s = 954^\circ\text{C}$$

Appendix BCorresponding yield stress

$$\psi = \frac{108 \left(1 - \frac{954}{1000}\right)}{954 - 440} = 0.00967$$

2.3.3.

$$\sigma_{y\theta} = \psi \sigma_{y20} = 0.00967 \times 275 = 2.66 \text{ N/mm}^2$$

Buckling load at the temperature 954 °C

$$P_u = 49713 \text{ kp}$$

$$P_{u\theta} = P_u \frac{\sigma_{y\theta}}{\sigma_y} = 49713 \times 0.00967 = 48.07 \text{ kp}$$

Correction factor $k=0.85$ **5.4.3**Admissible load under fire conditions

$$P_u = 48.07 / 0.85 = 56.55 \text{ kp}$$

$$N = 7051 \text{ kp} > 56.55 \text{ kp}$$

Protection required

Example (Continue)	Ref.
<i>or fire resistance period of 60min</i>	
<u>Temperature of steel column</u> $\theta_s = 814^\circ\text{C}$	Table 5
	Appendix B
<u>Corresponding yield stress</u>	
$\psi = 0.05371$	2.3.3
$\sigma_{y\theta} = 14.77\text{N/mm}^2$	
<u>Buckling load at the temperature $\theta_s = 814^\circ\text{C}$</u>	
$P_{u\theta} = 2670\text{kp}$	
Since $P_{u\theta} < N = 7051\text{kp}$	
Protection required	
<i>for fire resistance of 30min</i>	
<u>Temperature of steel column</u> $\theta_s = 484^\circ\text{C}$	Table 5
	Appendix B
<u>Corresponding yield stress</u>	
$\psi = 1.27$	2.3.3
<u>Buckling load at the temperature $\theta_s = 484^\circ\text{C}$</u>	
$P_u = 62964/0.85 = 74075\text{kp}$	
Since $P_{u\theta} > N = 7051\text{kp}$	
No protection required	

6.3 Swedish Institute of Steel Construction

Fire Load and Opening Factor

Compartment size = $(4.0 \times 3.5 \times 2.5) \text{ m}^3$

Construction: Dense concrete

Fire Load

From Table 2 [Appendix C] the design fire load is equal to 33.0 Mcal / m^2 . This value applies only to the fire load due to furniture and fittings.

The load due to floor and wall coverings can be derived from the following equation.

$$q = \frac{\sum m_v H_v}{A_t} \quad (\text{Mcal / m}^2) \quad (\text{MJ / m}^2) \quad (6.1)$$

where: m_v = the total weight of each individual combustible material constituent v in the fire compartment (kg).
 H_v = the effective calorific value of each individual combustible material constituent v in the fire compartment. (Mcal / kg) {MJ / KG}
 A_t = the total interval surface area of the fire compartment (walls, floor, ceiling) (m^2)

Assumptions

-weight of floor covering

$$1.5 (\text{kg / m}^2) \times (4.0 \times 3.5) (\text{m}^2) = 21.0 \text{ m}^2 \quad (6.2)$$

-calorific value of floor covering : 5.0 (Mcal / kg)

-weight of wall covering

$$0.2 (\text{kg / m}^2) \times 2.5 \text{ m}^2 = 5.0 \text{ kg} \quad (6.3)$$

-calorific value of wall covering : 5.0 (Mcal / kg)

From the above:

	m_v (kg)	H_v (Mcal / kg)	$m_v H_v$ (Mcal)
Floor covering	21.0	5	105.0
Wall covering	5.0	5	25.0
$\sum m_v H_v$			= 130.0 (Mcal)

$$A_t = 2 \times (3.5 \times 2.5) + 2 \times (4.0 \times 3.5) + 2 \times (4.0 \times 2.5) = 65.5 \text{ m}^2$$

From Equation 6.1 the fire load :

$$q = \left(\frac{130}{65.5} \right) \left(\frac{\text{Mcal}}{\text{m}^2} \right) = 1.98 \approx 2.00 \text{ (Mcal / m}^2\text{)}$$

$$\text{Total Design Fire Load: } 33.0 + 2.0 = 35.0 \text{ (Mcal / m}^2\text{)}$$

Opening Factor ($A\sqrt{h}/A_t$)

1/ Door closed (It remains intact during the fire)

$$A = \text{ventilation area} = (3.7 \times 1.5) \text{ (m}^2\text{)} = 5.55 \text{ m}^2$$

$$h = 1.5 \text{ m}$$

$$A_t = 65.5 \text{ m}^2$$

$$\frac{A\sqrt{h}}{A_t} = \frac{5.55\sqrt{1.5}}{65.5} = 0.104 \text{ m}^{1/2} \approx 0.105 \text{ m}^{1/2}$$

2/ Door Open

$$A = 5.55 + (2.0 \times 1.0) = 7.55 \text{ m}^2$$

$$\frac{A\sqrt{h}}{A_t} = \frac{7.55\sqrt{1.5}}{65.5} = 0.141 \text{ m}^{1/2} \approx 0.15 \text{ m}^{1/2}$$

For centre columns, the opening factor will be taken into account as equal to $0.15 \text{ m}^{1/2}$ to cover every possible case (door closed , open or intact during the fire).

For columns across the facade, the opening factor will be taken into account as equal to $0.105 \text{ m}^{1/2}$ to give results on the safe side.

3. Equivalent Fire Load and Opening Factor

DATA	Material	Area
Floor and Roof	Concrete	$2 \times (3.5 \times 4.0) = 28$ (49%)
Wall	Gypsum plaster	$2 \times 2.5 \times 3.5 + 2.5 \times 4.0$ $- 1.0 \times 2.0 =$ $= 25.5$ (44.0%)
	Steel Sheeting with mineral wool insulation	$4.0 \times 2.5 - 3.7 \times 1.5 =$ 4.40 (7%)

The fire compartment is a combination of the materials below:

Fire Compartment Type B	Concrete (100%)	
Fire Compartment Type G	Concrete (20%)	
	2 13 mm gypsum plaster - board	} 80%
	100 mm air gap	
Fire Compartment Type H	2 13 mm gypsum plaster board	
	Steel Sheeting	} 100%
	100 mm mineral wool	
	Steel Sheeting	

According to the above fire compartment:

$$k_f = \frac{49}{100} k_{f(B)} + \frac{7}{100} k_{f(H)} + \frac{44}{100} k_{f(P)} \quad (6.4)$$

a/ for opening factor $\frac{A\sqrt{h}}{A_t} = 105 \text{ m}^{1/2}$ and from Table 3 [Appendix C]

$$k_{f(B)} = 0.85$$

$$k_{f(H)} = 2.875 \approx 2.88$$

$$k_{f(G)} = 1.125 \approx 1.13$$

$$= 0.80 k_{f(G)} + 0.20 k_{f(P)} \Leftrightarrow k_{f(P)} = \frac{k_{f(G)} - 0.20 k_{f(B)}}{0.80} \quad (6.5)$$

From equations (6.4) and (6.5):

$$k_f = \frac{49}{100} 0.85 + \frac{7}{100} 2.88 + \left(\frac{44}{100}\right) \left(\frac{100}{80}\right) 1.13 - \left(\frac{44}{100}\right) \left(\frac{20}{80}\right) 0.85 = 1.14$$

b/ for Opening Factor $\frac{A\sqrt{h}}{A_t}$

$$k_{f(B)} = 0.85$$

$$k_{f(H)} = 2.00$$

$$k_{f(G)} = 0.90$$

$$0.80 k_{f(P)} + 0.20 k_{f(B)} \Leftrightarrow k_{f(P)} = \frac{k_{f(G)} - 0.20 k_{f(B)}}{0.80} \quad (6.6)$$

From Equations (6.4) and (6.6) :

$$k_f = \frac{49}{100} 0.85 + \frac{7}{100} 2.00 + \frac{44}{80} 0.90 - \left(\frac{44}{100}\right) \left(\frac{20}{80}\right) 0.85 = 0.96$$

Equivalent Fire Load = $k_f \times$ actual fire load

- for opening factor $\frac{A\sqrt{h}}{A_t} = 0.105 \text{ m}^{1/2}$

$$\Leftrightarrow q = 1.14 \times 21.5 = 24.50 \text{ (Mcal/m}^2\text{)}$$

$$k_f = 1.14$$

- for opening factor $\frac{A\sqrt{h}}{A_t} = 0.15 \text{ m}^{1/2}$

$$\Leftrightarrow q = 0.96 \times 21.5 = 20.6 \text{ (Mcal / m}^2\text{)}$$

$$k_f = 0.96$$

Equivalent Opening Factor = $k_f \times$ actual opening factor

- for opening factor $\frac{A\sqrt{h}}{A_t} = 0.105 \text{ m}^{1/2}$

$$\Leftrightarrow \frac{A\sqrt{h}}{A_t} = 1.14 \times 0.105 = 0.12 \text{ m}^{1/2}$$

$$k_f = 1.14$$

-for opening factor $\frac{A\sqrt{h}}{A_t} = 0.15 \text{ m}^{1/2}$

$$\Leftrightarrow \frac{A\sqrt{h}}{A_t} = 0.96 \times 0.15 = 0.14 \text{ m}^{1/2}$$

$$k_f = 0.96$$

Maximum Steel Temperature

Centre Column

Case 1: Exposed from four sides

Resultant emissivity

For type of construction 1 - column exposed to fire on all sides and from Table 4, the resultant emissivity $\epsilon_T = 0.7$

The accurate value for emissivity is given below:

$$\epsilon_T = \frac{1}{\frac{1}{\epsilon_t} + \frac{1}{\epsilon_s} - 1} = 0.7$$

where: ϵ_t = emissivity of the flames = 0.85

ϵ_s = emissivity of the steel section = 0.80

$\frac{F_s}{V_s}$ ratio

$$\frac{F_s}{V_s} = \frac{2h+4b-2d}{\text{cross sectional area}}$$

Top storey 152×152×23 UC

$$\frac{F_s}{V_s} = \frac{2 \times 152.4 + 2 \times 152.4 - 2 \times 6.1}{29.8 \times 10^2} = 0.3109 \text{ Mm}^{-1} = 311 \text{ m}^{-1}$$

Bottom storey 203×203×86UC

$$\frac{F_s}{V_s} = \frac{2 \times 222.3 + 2 \times 208.8 - 2 \times 13}{110.1 \times 10^2} = 0.11388 \text{ mm}^{-1} = 113.88 \text{ m}^{-1} \approx 114 \text{ m}^{-1}$$

Maximum steel temperature θ_{\max}

Equivalent Fire Load = 3360 {Mcal / m²} ≈ 34 {Mcal / m²}

Equivalent Opening Factor = 0.14 m^{1/2}

Resultant Emissivity = 0.7

$\frac{F_s}{V_s}$ = 114 m⁻¹ (bottom)

= 311 m⁻¹ (top)

F_s/V_s and $A\sqrt{h}/A_t$ values are out of the range of Table 6.

For a given resultant emissivity and opening factor, the maximum steel temperature increases with an increase of the ratio F_s/V_s .

For a given resultant emissivity and a given F_s/V_s ratio, the maximum steel temperature decreases with an increase of the opening factor.

So, $\theta > 770^\circ\text{C}$

This value is out of the range of temperature values of Figure 5.

Top storey

For $\sigma_s=2600 \text{ kgf / cm}^2\{260 \text{ MPa}\}$, $\gamma=1$, $\lambda=69.29$ and from Table 5, $\sigma_k<55$

MPa, $P<55\times 29.8\times 10^{-1}=163.9\text{kN}$

$P=70.51\text{kN}<163.9\text{kN}$

NO PROTECTION REQUIRED

Bottom storey

For $\sigma_s=2600\text{kgf/cm}^2\{260\text{MPa}\}$, $\gamma=1$, $\lambda=56$ and from Table 5, $\sigma_k<62\text{MPa}$,

$P<62\times 110.1\times 10^{-1}=682.6 \text{ kN}$

$P=1239\text{kN}> 682.6\text{kN}$

PROTECTION REQUIRED

Case 2: Next to a wallResultant emissivity

$\epsilon_r=0.7$ (to obtain steel temperatures on the safe side)

 F_s/V_s ratio

$$\frac{F_s}{V_s} = \frac{2h+3b-2d}{\text{cross sectional area}}$$

Top storey 152×152×23 UC,

$$\frac{F_s}{V_s} = \frac{2\times 152.4+3\times 152.4+2\times 6.1}{29.8\times 10^2} = 0.2516\text{mm}^{-1} = 251\text{m}^{-1}$$

Bottom storey 203×203×86 UC,

$$\frac{F_s}{V_s} = \frac{2\times 222.3+3\times 208.8-2\times 13}{110.1\times 10^2} = 0.09491\text{mm}^{-1} = 95\text{m}^{-1}$$

Max steel temperature θ_{\max}

$q_{\max}>770^{\circ}\text{C}$

Out of the range of Figure 5

For $\sigma_s=260 \text{ MPa}$, $\gamma=1$ we conclude the following:

Top storey

For $\lambda=69.29$, $\sigma_k<55\text{MPa}$, $P=70.51\text{kN}<163.9\text{kN}$

NO PROTECTION REQUIRED

Bottom storey

For $\lambda=56$, $P=1239.26\text{kN}>682.6\text{kN}$

PROTECTION REQUIRED

Case 3: Column built into a wall

Resultant emissivity

$$\epsilon_r=0.5$$

F_s/V_s ratio

$$\frac{F_s}{V_s} = \frac{b}{bt} = \frac{1}{t}$$

Top storey 152×152×23 UC,

$$\frac{F_s}{V_s} = \frac{1}{t} = \frac{1}{6.8} = 0.147\text{mm}^{-1} = 147\text{m}^{-1}$$

Bottom storey 203×203×86 UC,

$$\frac{F_s}{V_s} = \frac{1}{t} = \frac{1}{20.5} = 0.049\text{mm}^{-1} = 49\text{m}^{-1}$$

Max steel temperature θ_{\max}

$$\theta_{\max} > 540^\circ\text{C}$$

Critical buckling stress

Top storey

For $\sigma_s=260\text{MPa}$, $\gamma=1$, $\theta_{\max}=600^\circ\text{C}$, $\lambda=69$ and Fig.5, $\sigma_k=50\text{MPa}$

$$P=50 \times 29.8 \times 10^{-1} = 149\text{kN} > 70.51\text{kN}$$

NO PROTECTION REQUIRED

Bottom storey

For $\sigma_s=260\text{MPa}$, $\gamma=1$, $\theta_{\max}=540^\circ\text{C}$, $\lambda=56$ and from Fig. 5, $\sigma_k=85\text{MPa}$

$$P=85 \times 110.1 \times 10^{-1} = 935.9\text{kN} < 1239.26\text{kN}$$

PROTECTION REQUIRED

Edge column**Case 1: Next to a wall**

Resultant emissivity, F_s/V_s ratio, θ_{\max} as for Centre Column design, Case 2.

Top storey

$$P=35.92\text{kN}<163.9\text{kN}$$

NO PROTECTION REQUIRED

Bottom storey

$$P=624\text{kN}<682.6\text{kN}$$

NO PROTECTION REQUIRED

Case 2: Column built into a wall

Resultant emissivity, F_s/V_s ratio, θ_{\max} as for Centre Column design, Case 3.

Top storey

$$P=149\text{kN}>35.92\text{kN}$$

NO PROTECTION REQUIRED

Bottom storey

$$P=935.9\text{kN}>624.13\text{kN}$$

NO PROTECTION REQUIRED

6.4. Fire Protection Insulation

The fire protection insulation VULTEX is chosen from the yellow book [ASFPCM,'88]. The characteristics of the above material are given in the Appendix D. The profile protection technique is chosen.

Centre Column

TOP STOREY

$$F_s/V_s=311\text{m}^{-1}$$

for fire resistance period of 30min, fire protection thickness of 10mm,

for fire resistance period of 60min, fire protection thickness of 17mm,

for fire resistance period of 90min, fire protection thickness of 25mm.

BOTTOM STOREY

$$F_s/V_s=114\text{m}^{-1}$$

for fire resistance period of 30min, fire protection thickness of 10mm,

for fire resistance period of 60min, fire protection thickness of 16mm,

for fire resistance period of 90min, fire protection thickness of 22mm.

Edge Column

CASE 1

TOP LEVEL

$$F_s/V_s=251\text{m}^{-1}$$

for fire resistance period of 30min, fire protection thickness of 10mm,

for fire resistance period of 60min, fire protection thickness of 17mm,

for fire resistance period of 90min, fire protection thickness of 24mm.

BOTTOM LEVEL

$$F_s/V_s=95\text{m}^{-1}$$

for fire resistance period of 30min, fire protection thickness of 10mm,

for fire resistance period of 60min, fire protection thickness of 15mm,

for fire resistance period of 90min, fire resistance thickness of 22mm.

CASE 2**TOP STOREY**

$$F_s/V_s = 147\text{m}^{-1}$$

for fire resistance period of 30min, fire protection thickness of 10mm,

for fire resistance period of 60min, fire protection thickness of 16mm,

for fire resistance period of 90min, fire protection thickness of 23mm.

BOTTOM LEVEL

$$F_s/V_s = 48\text{m}^{-1}$$

for fire resistance period of 30min, fire protection thickness of 10mm,

for fire resistance period of 60min, fire protection thickness of 12mm,

for fire resistance period of 90min, fire protection thickness of 17mm.

6.5 Conclusions

The results of the fire insulation check are presented in the Tables 6.1, 6.2. According to those results, one may observe that consideration of the effect of natural fires can result in considerable benefit. In this context, natural fires are considered as the fires which build up and decay in accordance with the mass and energy balance within the compartment. The results obtained if one applies the British Code, BS 5950: Part 8 [BSI, 1990] and the European Recommendations[ECCS, 1983] are conservative in comparison with the Swedish Code results.

In the case of bare columns, which is the object of this study, the fire test results may not be representative because of the limited series of fire tests conducted. The test data on columns relates to slenderness values which are less than 50 because of the restraint of the test furnace. In BS 5950: Part 8 [BSI, 1990], the limiting temperatures for design have been extended up to the slenderness value of 70 and they are derived by assuming uniform heating on all the faces. In the design example, presented in this Chapter, the edge column was designed using limiting temperatures and assuming that the column is uniformly heated.

Because BS5950: Part 8 [BSI, 1990] applies only to members which are not subject to second order effects resulting from deflection during a fire, the design of the Column built into a wall requires the use of a more exact method of calculation. Because the design according to the European Recommendations [ECCS, 1983] is limited to individual elements subjected to the standard fire, the design of a Column next to a wall or built into a wall is excluded.

Further refinement and sensitivity can be offered with the application of finite element analysis.

Code	BS 5950: Part 8: Code of Practice for Fire Resistant Design			European Recommendation for Fire Safety of Steel Structures			Swedish Institute of Steel Constr.
Centre Column							
Cases of study	<i>Fire resistance period (min)</i>						
	<i>90</i>	<i>60</i>	<i>30</i>	<i>90</i>	<i>60</i>	<i>30</i>	
Top level							
Case 1	Yes	Yes	Yes	Yes	Yes	No	No
Case 2	Yes	Yes	Yes	-	-	-	No
Case 3	-	-	-	-	-	-	No
Bottom level							
Case 1	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Case 2	Yes	Yes	Yes	-	-	-	Yes
Case 3	-	-	-	-	-	-	Yes
<p>where:</p> <p>Case 1: Column exposed from four sides</p> <p>Case 2: Column next to a wall</p> <p>Case 3: Column built into a wall</p> <p>Yes: Protection required</p> <p>No: No protection required</p>							

Table 6.1 Tabulated results of fire insulation check-
-Centre Column

Code	BS 5950: Part 8: Code of Practice for Fire Resistant Design			European Recommendations for Fire Safety of Steel Structures			Swedish Institute of Steel Constr.
Edge Column							
Cases of study	<i>Fire resistance period (min)</i>						
	<i>90</i>	<i>60</i>	<i>30</i>	<i>90</i>	<i>60</i>	<i>30</i>	
Top level							
Case 1	Yes	Yes	Yes	-	-	-	No
Case 2	-	-	-	-	-	-	No
Bottom level							
Case 1	Yes	Yes	Yes	-	-	-	No
Case 2	-	-	-	-	-	-	No
<p>where:</p> <p>Case 1: Column next to a wall</p> <p>Case 2: Column built into a wall</p> <p>Yes: Protection required</p> <p>No: No protection required</p>							

Table 6.2 Tabulated results of fire insulation check-
-Edge column

CHAPTER SEVEN Conclusions - -Suggestions for future work

Three basic components have been considered in order to predict field performance and ultimately design a structure for the impact of fire. The components are given as follows:

1. the fire
2. the heat transfer to the structural members
3. the effect of elevated temperatures on structural performance.

These three components are not independent, but interrelate in a very complex manner. Each involves many parameters. An understanding of these parameters is necessary in order to develop models which will adequately predict the phenomenon.

In order to illustrate the interrelationships between the three basic components affecting structural response and identify some of the key variables in each component, a logic flow chart has been developed for the response of structural steel to fire (Fig. 7.1). The chart identifies the critical components necessary to predict the performance of a structural system.

The inter-relations among the three major components are different for each material. In this study on steel columns, the fire was isolated from the other two basic components (heat transfer, structural response). It was implied that the steel structures exhibit a minimum effect on the relationship between the fire, the heat transfer to structural members and structural response. This assumption does not hold for structures constructed from different materials. With regard to wood structures, once wood reaches its ignition point the wood ignites and augments the original fire. The increased compartment combustion rate results in an increase in the radiant heat to the wood

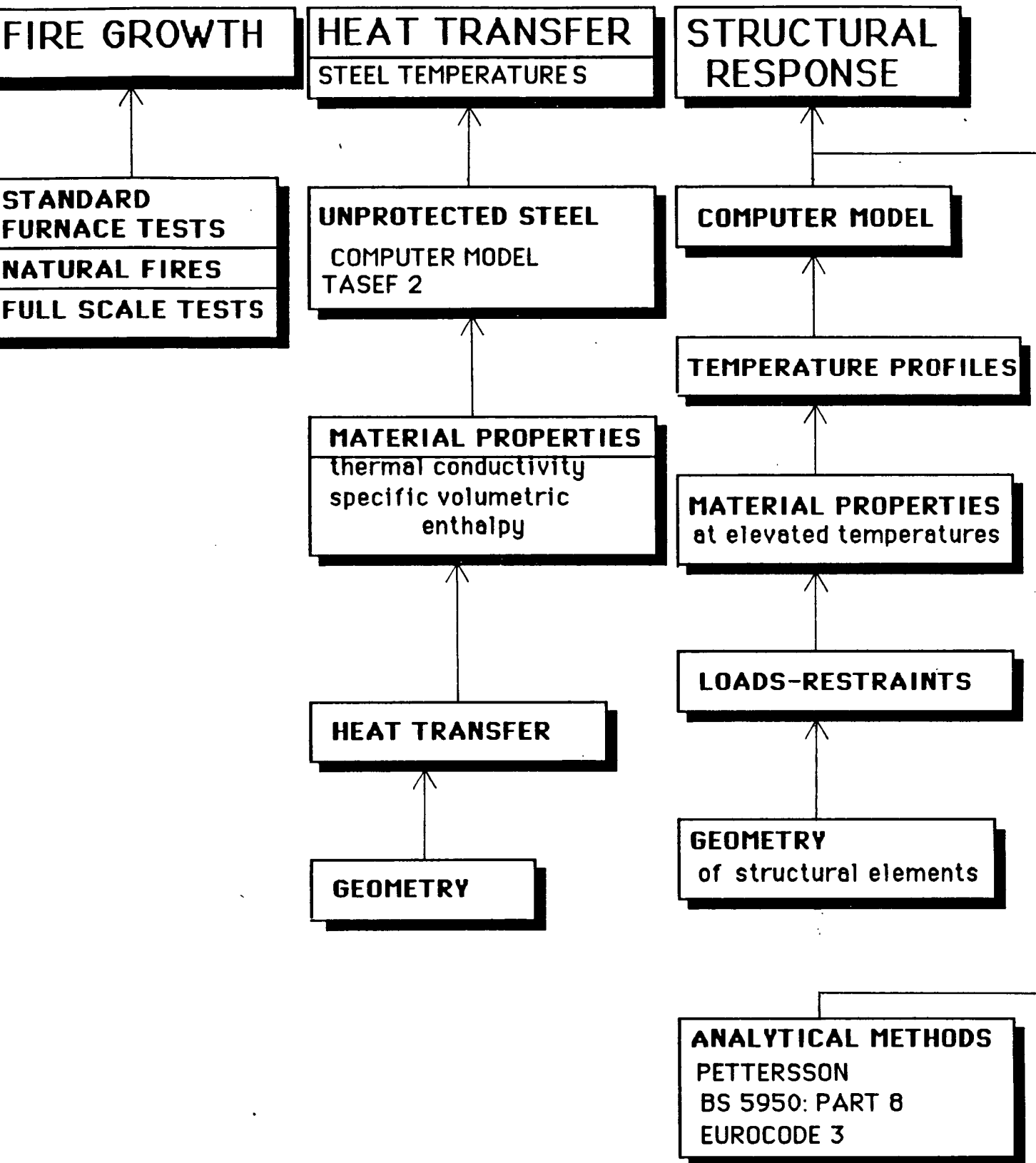


Fig. 7.1 Components necessary to predict the structural performance of a structural system.

structure and increases the wood charring rate. A reinforced concrete wall may have an important impact on the heat transfer to and from the compartment resulting in alterations to a compartment fire.

For the purpose of the present study, the fire growth was predicted either using the standard fire curve [BSI, 1972] or temperature-time curves for different fire loads and opening factors [Pettersson, 1970]. Full-scale test results have also been used as data to predict the structural response to fire.

The fire environment may also be predicted using computer models. Some computer models deal only with one phase of fire development, either the pre-flashover regime or the post-flashover regime. Modelling the pre-flashover regime is difficult because the details of fuel type and arrangement are unknown to the designer. To tackle the problem, an envelope of statistical data derived from a large number of full-scale tests or a series of computer simulations may be used. Problems regarding the modelling of the post-flashover fire are similar. If the pre-flashover fire is ignored, flashover is not predicted and the model may not be applicable to the given problem. The effect of the preflashover regime on openings into the compartment is unknown. Window glass breakage and door burn through can be taken into account. The usual approach to the problem is to assume that all potential openings into the compartment are open. The reliability of the above approach is unknown when predicting the development of the real fire.

The second component studied, once the fire is defined, is the heat transfer to the structure. This study is limited to structural steel and more specifically to unprotected steel. The problem at this stage is solved with the aid of the computer program TASEF-2 which uses a forward difference time approach (Chapter 3). The material properties used as data to the program are given

as a function of temperature (Chapter 2). For the cases studied in this research, the results obtained compare satisfactorily with those obtained from the experimental tests (standard or full-scale fire tests). It can therefore be claimed that the computer program can be used to simulate a standard test or full scale test conditions.

One of the future trends regarding this program in particular is its modification to handle three dimensions.

Regarding the present version of the program, the program could be transformed to model a structural member in a more user friendly way. At the moment, the user has to model a specific element as a rectangular one and define areas which are fictitious.

Once the heat transfer problem was solved and the time-dependent temperature gradient in a structural member was determined, the third component which is the structural response was studied. In Chapter Five, different design methods have been studied to establish their reliability in predicting the structural response in a fire. Different design methods were applied to four column cases and their relative applicability was examined. A computer model could be used to analyse the effect of fire on the structure.

The analysis should include the effects:

- change in loading
- change in the resistance to the load

The change in load occurs in a structural element because of its restraint to expansion when heated. It is a serious problem where there are significant transverse gradients which result in moments in excess of those caused by the eccentricity of the actual member. Increased temperature causes a reduction in the structural load resistance of a structural member because of changes in the steel properties.

In Chapter Four , the variation of the steel properties with temperature was studied as well as their modelling.

The target of future work would be the development of a design procedure based on computer programs which would predict the field performance of steel. In that case, the results of a computer model of one basic component can be used as input to model a subsequent component.

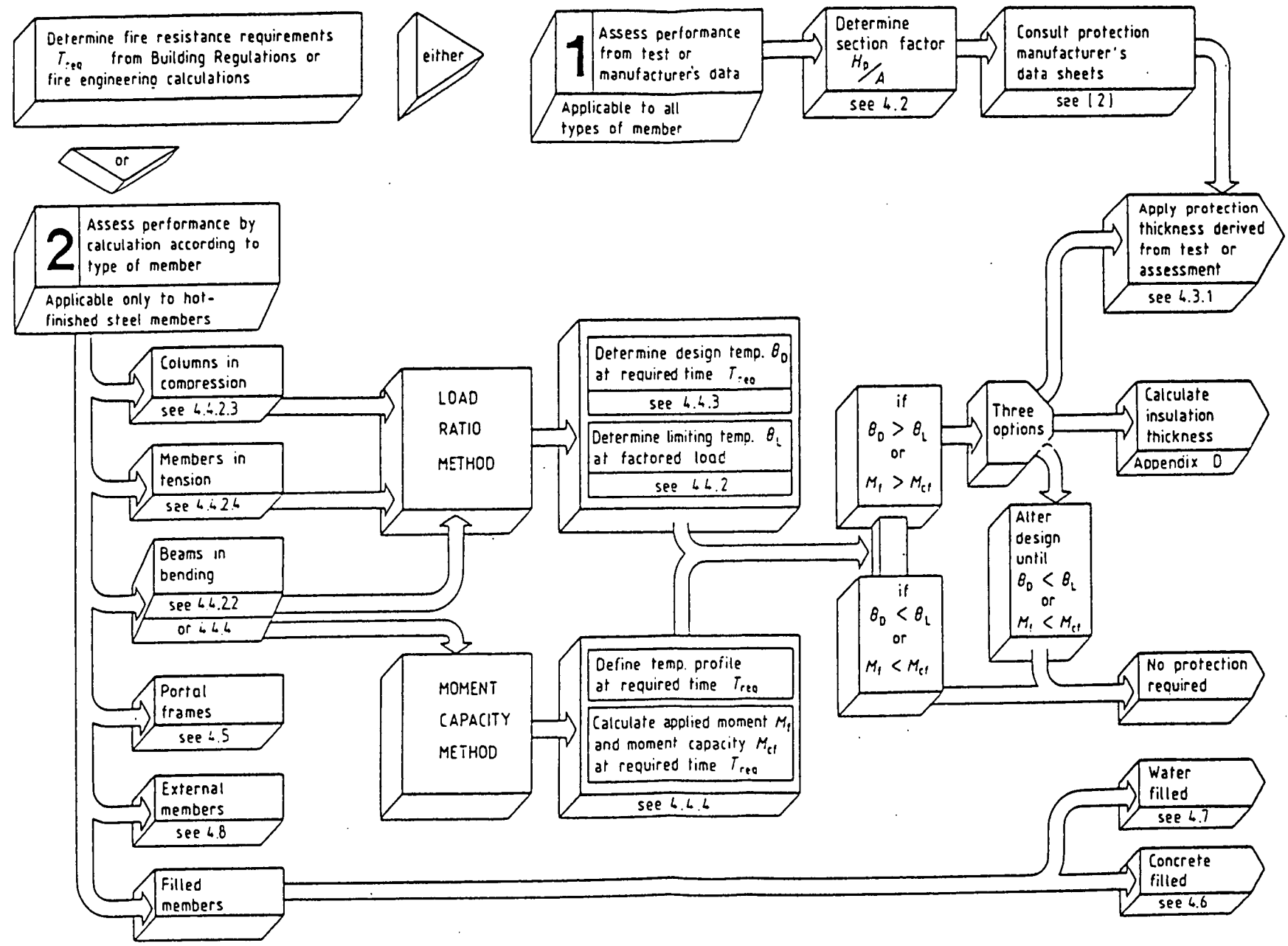


Fig. 1 Fire Design Procedures

Table 1. Strength reduction factors for steel complying with grades 43 to 50 of BS 4360

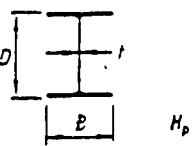


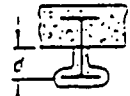







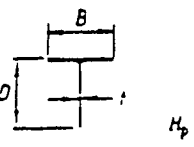


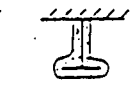


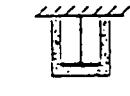
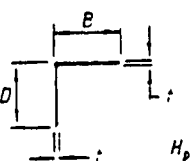


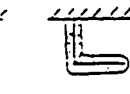



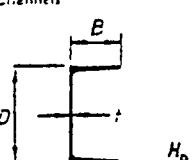





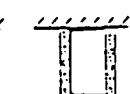
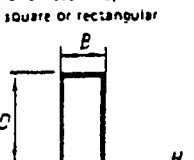




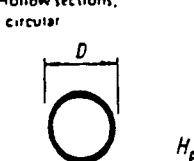


Temperature	Strength reduction factors at a strain (in %) of:		
	0.5	1.5	2.0
°C			
100	0.97	1.00	1.00
150	0.959	1.000	1.000
200	0.946	1.000	1.000
250	0.884	1.000	1.000
300	0.854	1.000	1.000
350	0.826	0.968	1.000
400	0.798	0.956	0.971
450	0.721	0.898	0.934
500	0.622	0.756	0.776
550	0.492	0.612	0.627
600	0.378	0.460	0.474
650	0.269	0.326	0.337
700	0.186	0.223	0.232
750	0.127	0.152	0.158
800	0.071	0.108	0.115
850	0.045	0.073	0.079
900	0.030	0.059	0.062
950	0.024	0.046	0.052

NOTE 1. Intermediate values may be obtained by linear interpolation.

NOTE 2. For temperatures higher than the values given, a linear reduction in strength to zero at 1300 °C may be assumed.

Table 2. Load factors for fire limit state

Load	γ_f
Dead load	1.00
Imposed loads:	
(a) permanent:	
(1) those specifically allowed for in design, e.g. plant, machinery and fixed partitions	1.00
(2) in storage buildings or areas used for storage in other buildings (including libraries and designated filing areas)	1.00
(b) non-permanent:	
(1) in escape stairs and lobbies	1.00
(2) all other areas (imposed snow loads on roofs may be ignored)	0.80
Wind loads	0.33

	4 sides	3 sides	3 sides	2 sides	1 side	4 sides	3 sides	3 sides	2 sides	1 side
Universal beams, universal columns and joists (plain and casted) 										
	$LB - 2D - 2t$	$3B - 2D - 2t$	$2B - 2d - t$	$2B - D - t$	B	$2B - 2D$	$B - 2D$	$B - 2d$	$B - D$	B
										
	$2B - 2D$	$B - 2D$	$2B - 2D - t$			$2B - 2D$	$B - 2D$	$B - 2D$		
Angles 										
	$2B - 2D$	$B - 2D$	$2B - 2D - t$			$2B - 2D$	$B - 2D$	$B - 2D$		
Channels 										
	$LB - 2D - 2t$	$LB - D - 2t$	$3B - 2D - 2t$			$2B - 2D$	$2B - D$	$B - 2D$		
Hollow sections, square or rectangular 										
	$2B - 2D$	$B - 2D$				$2B - 2D$	$B - 2D$			
Hollow sections, circular 										
	πD					πD				(see note 2)

NOTE 1. The general principle applied in calculating H_p/A for unprotected or profile protected sections is to use the actual profile of the steel section; fillet radii may be taken into account and are normally included in published tables. For box protection, the smallest enclosing rectangle of the steel section is used.

NOTE 2. The air space created in boxing a section improves the insulation and a value of H_p/A , and therefore H_p , higher than that for profile protection would be anomalous. Hence H_p is taken as the circumference of the tube and not $4D$.

Table 3

Calculation of H_p/A values

Table 5. Limiting temperatures for design of protected and unprotected hot finished members

Description of member	Limiting temperature at a load ratio of:					
	0.7	0.6	0.5	0.4	0.3	0.2
	°C	°C	°C	°C	°C	°C
Members in compression, for a slenderness λ (see note) < 70 > 70 but < 180	510	540	580	615	655	710
	460	510	545	590	635	635
Members in bending supporting concrete slabs or composite slabs: unprotected members, or protected members complying with item (a) or (b) of 2.3 other protected members	590	620	650	680	725	780
	540	585	625	655	700	745
Members in bending not supporting concrete slabs: unprotected members, or protected members complying with item (a) or (b) of 2.3 other protected members	520	555	585	620	660	715
	460	510	545	590	635	690
Members in tension: all cases	460	510	545	590	635	690

NOTE. λ is the slenderness, i.e. the effective length divided by the radius of gyration.

Table 6. Design temperature for columns and tension members

Flange thickness	Design temperature for fire resistance period of:			
	30 min	60 min	90 min	120 min
mm	°C	°C	°C	°C
< 6.8	841	945	1006	1049
9.4	801	911	950	1020
11.0	771	900	950	1020
12.5	747	891	950	1020
14.2	724	882	950	1020
15.4	709	877	950	1020
17.3	689	869	950	1020
18.7	676	864	950	1020
20.5	661	858	950	1020
21.7	652	854	950	1020
23.8	637	848	950	1020
25.0	630	844	950	1020
27.0	618	839	950	1020
30.2	601	832	950	1020
31.4	595	829	950	1020
36.5	574	820	950	1020
37.7	569	818	950	1020
42.9	552	810	950	1020
44.1	548	808	950	1020
49.2	533	801	950	1020
58.0	512	791	950	1020
67.5	494	782	950	1020
77.0	479	774	950	1020

NOTE. The values given in table 6 assume heating from four sides.

Table 4. Maximum section factor for unprotected members

Description	H_p/A
Members in bending, directly supporting concrete slabs or composite slabs	m^{-1} 90
Columns in simple construction (as described in BS 5950 : Part 1)	50
Columns comprising rolled sections filled with aerated concrete blockwork between the flanges in accordance with (1)	69

Table 7. Design temperature for beams

Flange thickness	Design temperature for fire resistance period of:			
	30 min	60 min	90 min	120 min
mm	°C	°C	°C	°C
< 6.8	810	940	1000	1045
8.6	790	939	1000	1045
9.7	776	938	1000	1045
10.9	767	938	1000	1045
11.8	755	936	1000	1045
12.7	750	936	1000	1045
13.2	746	936	1000	1045
14.8	741	936	1000	1045
17.0	739	935	1000	1045
17.7	736	933	1000	1045
18.8	730	931	1000	1045
19.7	722	929	1000	1045
20.2	719	929	1000	1045
22.1	716	928	1000	1045
23.6	694	920	1000	1045
25.4	688	919	1000	1045
26.8	676	914	1000	1045
27.9	665	908	1000	1045
32.0	625	885	1000	1045
36.6	586	849	1000	1045

NOTE. The values in table 7 assume heating from three sides.

Table 8. Design temperature reductions

Aspect ratio D_e/B_e	Design temperature reduction for fire resistance period of:			
	30 min	60 min	90 min	> 90 min
	°C	°C	°C	°C
< 0.6	80	40	20	0
> 0.6 < 0.8	40	20	0	0
> 0.8 < 1.1	20	0	0	0
> 1.1 < 1.5	10	0	0	0
> 1.5	0	0	0	0

NOTE. This table does not apply to channels or hollow sections.

Appendix B: Eurocode No. 3. Design of steel structures
Part 1: Draft 1990. Structural design.
ECCS- Technical Committee 3 - Fire Safety
of Steel Structures

Table 1 Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel to EN 10025.				
Nominal steel grade	Thickness t mm ^{*)}			
	$t \leq 40\text{mm}$		$40\text{mm} < t \leq 100\text{mm}$	
	f_y (N/mm ²)	f_u (N/mm ²)	f_y (N/mm ²)	f_u (N/mm ²)
Fe E 235	235	360	215	340
Fe E 275	275	430	255	410
Fe E 355	355	510	335	490

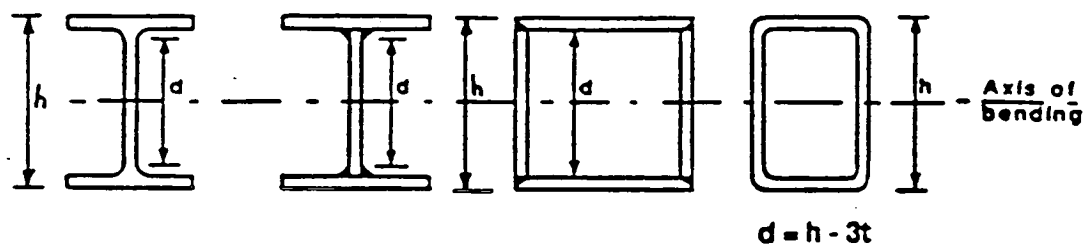
*) t is the nominal thickness of the element.

Table 2 Reduction factors

$\bar{\lambda}$	χ for buckling curve			
	a	b	c	d
0,2	1,0000	1,0000	1,0000	1,0000
0,3	0,9775	0,9641	0,9491	0,9235
0,4	0,9528	0,9261	0,8973	0,8504
0,5	0,9243	0,8842	0,8430	0,7793
0,6	0,8900	0,8371	0,7854	0,7100
0,7	0,8477	0,7837	0,7247	0,6431
0,8	0,7957	0,7245	0,6622	0,5797
0,9	0,7339	0,6612	0,5998	0,5208
1,0	0,6656	0,5970	0,5399	0,4671
1,1	0,5960	0,5352	0,4842	0,4189
1,2	0,5300	0,4781	0,4338	0,3762
1,3	0,4703	0,4269	0,3888	0,3385
1,4	0,4179	0,3817	0,3492	0,3055
1,5	0,3724	0,3422	0,3145	0,2766
1,6	0,3332	0,3079	0,2842	0,2512
1,7	0,2994	0,2781	0,2577	0,2289
1,8	0,2702	0,2521	0,2345	0,2093
1,9	0,2449	0,2294	0,2141	0,1920
2,0	0,2229	0,2095	0,1962	0,1766
2,1	0,2036	0,1920	0,1803	0,1630
2,2	0,1867	0,1765	0,1662	0,1508
2,3	0,1717	0,1628	0,1537	0,1399
2,4	0,1585	0,1506	0,1425	0,1302
2,5	0,1467	0,1397	0,1325	0,1214
2,6	0,1362	0,1299	0,1234	0,1134
2,7	0,1267	0,1211	0,1153	0,1062
2,8	0,1182	0,1132	0,1079	0,0997
2,9	0,1105	0,1060	0,1012	0,0937
3,0	0,1036	0,0994	0,0951	0,0882

Table 3
(Sheet 1) Maximum width-to-thickness ratios
 for compression elements

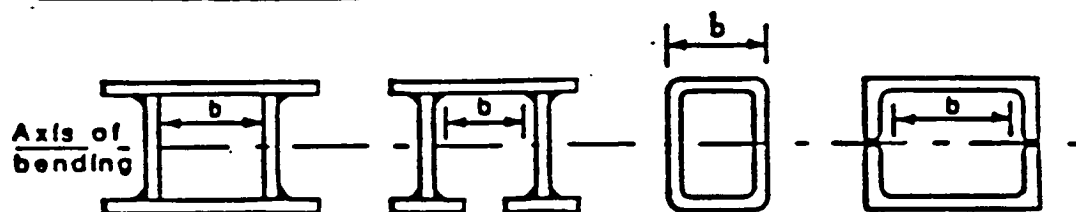
(a) Webs: (Internal elements perpendicular to axis of bending)



Class	Web subject to bending	Web subject to compression	Web subject to bending and compression	
Stress distribution (compression positive)				
1	$d/t \leq 72\epsilon$	$d/t \leq 33\epsilon$	when $\alpha > 0.5$: $d/t \leq 396\epsilon/(13\alpha - 1)$ when $\alpha < 0.5$: $d/t \leq 36\epsilon/\alpha$	
2	$d/t \leq 83\epsilon$	$d/t \leq 38\epsilon$	when $\alpha > 0.5$: $d/t \leq 456\epsilon/(13\alpha - 1)$ when $\alpha < 0.5$: $d/t \leq 41.5\epsilon/\alpha$	
Stress distribution (compression positive)				
3	$d/t \leq 124\epsilon$	$d/t \leq 42\epsilon$	when $\psi > -1$: $d/t \leq 42\epsilon/(0.67 + 0.33\psi)$ when $\psi \leq -1$: $d/t \leq 62\epsilon(1-\psi) \sqrt{-\psi}$	
$\bar{\epsilon} = \sqrt{235/f_y}$	f_y	235	275	355
	ϵ	1	0.92	0.81

Table 3
(Sheet 2) Maximum width-to-thickness ratios
for compression elements

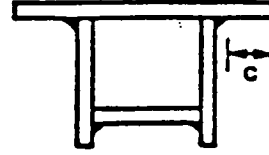
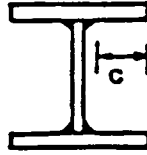
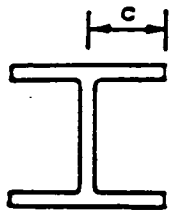
(b) Internal flange elements: (Internal elements parallel to axis of bending)



Class	Type	Section in bending	Section in compression		
1	Rolled Hollow Section Other	$(b-3t)/t \leq 33\epsilon$ $b/t \leq 33\epsilon$	$(b-3t)/t \leq 42\epsilon$ $b/t \leq 42\epsilon$		
2	Rolled Hollow Section Other	$(b-3t)/t \leq 38\epsilon$ $b/t \leq 38\epsilon$	$(b-3t)/t \leq 42\epsilon$ $b/t \leq 42\epsilon$		
3	Rolled Hollow Section Other	$(b-3t)/t \leq 42\epsilon$ $b/t \leq 42\epsilon$	$(b-3t)/t \leq 42\epsilon$ $b/t \leq 42\epsilon$		
$\epsilon = \sqrt{235/f_y}$		f_y	235	275	355
		ϵ	1	0,92	0,81

Table 3
(Sheet 3) Maximum width-to-thickness ratios for compression elements

(c) Outstand flanges:



Rolled sections

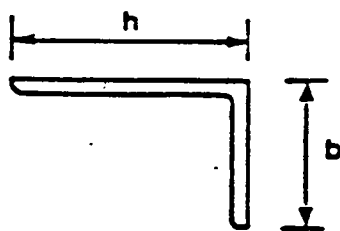
Welded sections

Class	Type of section	Flange subject to compression	Flange subject to compression and bending		
			Tip in compression	Tip in tension	
Stress distribution in element (compression positive)					
1	Rolled	$c/t \leq 10\epsilon$	$c/t \leq \frac{10\epsilon}{\alpha}$	$c/t \leq \frac{10\epsilon}{\alpha \sqrt{\alpha}}$	
	Welded	$c/t \leq 9\epsilon$	$c/t \leq \frac{9\epsilon}{\alpha}$	$c/t \leq \frac{9\epsilon}{\alpha \sqrt{\alpha}}$	
2	Rolled	$c/t \leq 11\epsilon$	$c/t \leq \frac{11\epsilon}{\alpha}$	$c/t \leq \frac{11\epsilon}{\alpha \sqrt{\alpha}}$	
	Welded	$c/t \leq 10\epsilon$	$c/t \leq \frac{10\epsilon}{\alpha}$	$c/t \leq \frac{10\epsilon}{\alpha \sqrt{\alpha}}$	
Stress distribution in element (compression positive)					
3	Rolled	$c/t \leq 15\epsilon$	$c/t \leq 23\epsilon \sqrt{k_\sigma}$		
	Welded	$c/t \leq 14\epsilon$	$c/t \leq 21\epsilon \sqrt{k_\sigma}$		
			For k_σ see table 5.3.3		
$\epsilon = \sqrt{235/f_y}$		f_y	235	275	355
		ϵ	1	0,92	0,81

Table 3
(Sheet 4) Maximum width-to-thickness ratios
for compression elements

(d) Angles:

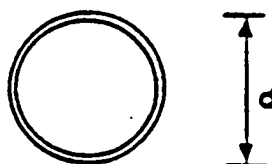
Refer also to (c)
"Outstand flanges"
(see Sheet 3).



(Does not apply to
angles in continuous
contact with other
components)

Class	Section in compression
Stress distribution across section (compression positive)	
3	$\frac{h}{t} \leq 15\epsilon \quad : \quad \frac{b+h}{t} \leq 23\epsilon$

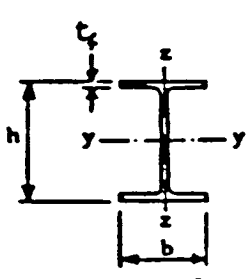
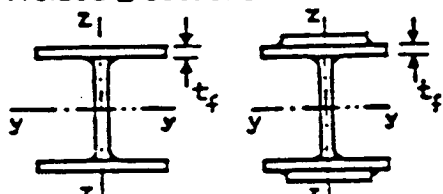
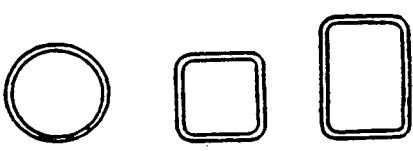
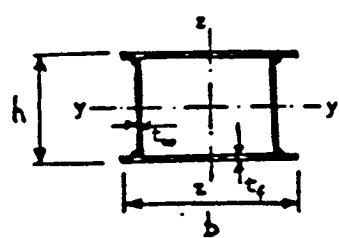
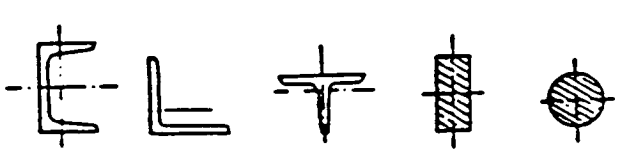
(e) Tubular sections:



Class	Section in bending and/or compression
1	$d/t \leq 50\epsilon^2$
2	$d/t \leq 70\epsilon^2$
3	$d/t \leq 90\epsilon^2$

$\epsilon = \sqrt{235/f_y}$	f_y	235	275	355
	ϵ	1	0,92	0,81
	ϵ^2	1	0,85	0,66

Table 4 Selection of buckling curve for a cross-section

Cross section	Limits	Buckling about axis	Buckling curve
Rolled I-sections 	$h/b > 1,2:$ $t_f \leq 40\text{mm}$	y - y z - z	a b
	$40\text{mm} < t_f \leq 100\text{mm}$	y - y z - z	b c
	$h/b \leq 1,2:$ $t_f \leq 100\text{mm}$ $t_f > 100\text{mm}$	y - y z - z y - y z - z	b c d d
Welded I-sections 	$t_f \leq 40\text{mm}$ $t_f > 40\text{mm}$	y - y z - z y - y z - z	b c c d
Hollow sections 	hot rolled	any	a
	cold formed - using f_{yb})	any	b
	cold formed - using f_{ya})	any	c
Welded box sections 	generally (except as below)	any	b
	thick welds and $b/t_f < 30$ $h/t_w < 30$	y - y z - z	c c
U-, L-, T- and solid sections 		any	c

) See 5.5.1.4(4) and figure 5.5.2

$\epsilon_r = 0.5$		section factor F/V [m^{-1}]								
time t [min]	gastemp. ϑ_i [°C]	10	20	30	50	100	150	200	250	300
0	20	20	20	20	20	20	20	20	20	20
15	739	95	164	226	333	514	612	664	691	705
30	842	215	367	484	637	781	814	824	829	831
45	902	343	552	682	808	877	888	893	895	896
60	945	467	700	814	897	930	936	939	940	941
75	979	580	809	898	949	967	972	974	975	976
90	1006	678	889	954	985	997	1000	1002	1003	1003

Table 5 Calculated temperature of non-insulated steel members as a function of the section factor F/V and time t ($\epsilon_r=0.5$, $\Delta t=30$ secs)

Fire Engineering Design of Steel Structures [Pettersson, 1976]

LOADBEARING STRUCTURE

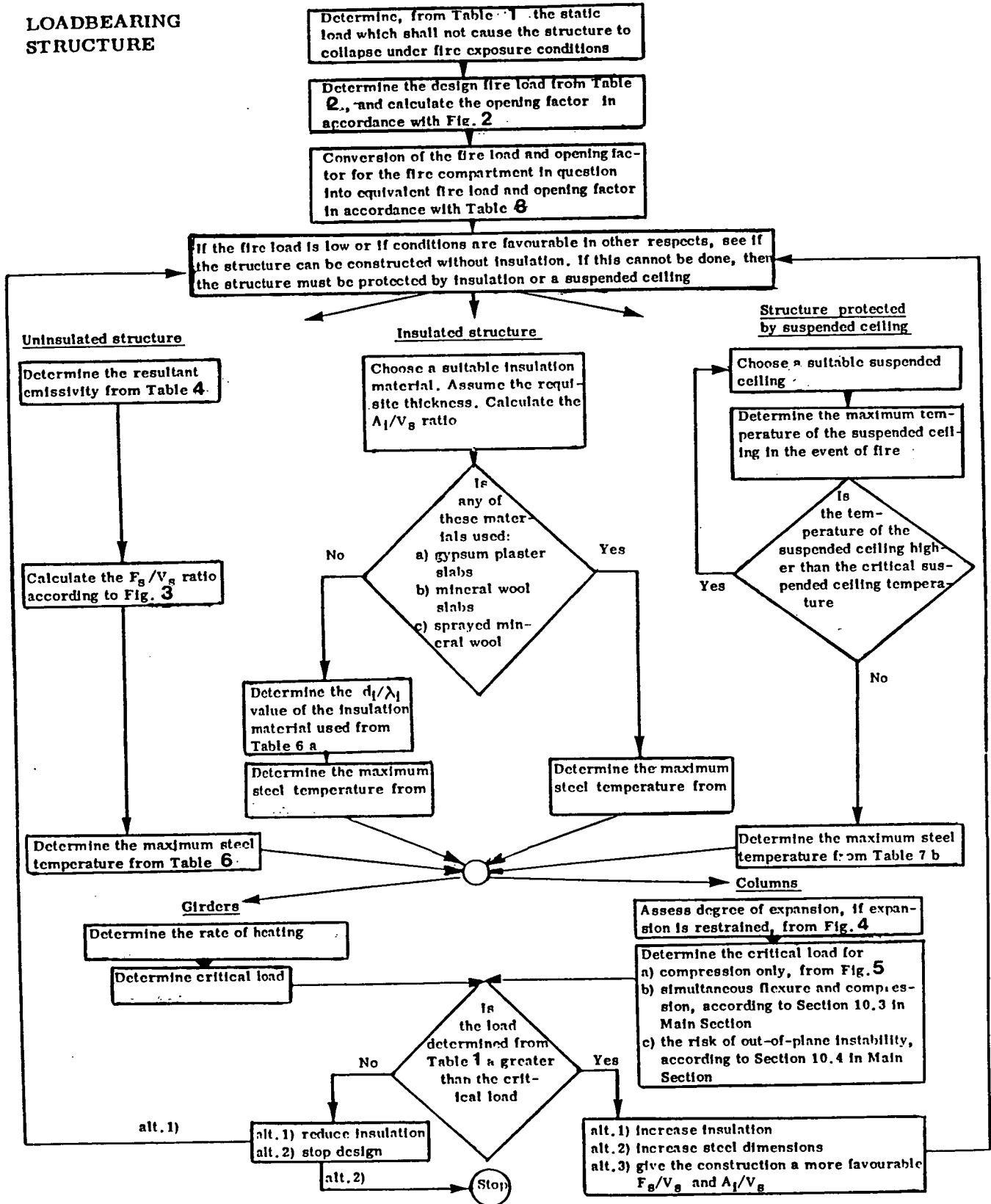


Fig. 1 Flow chart which illustrates the design procedure.

DETERMINATION OF THE DESIGN STATIC LOAD IN THE EVENT OF FIRE

Table 1 Static load which shall not cause a loadbearing structure to collapse under fire exposure conditions

It shall be shown that, during a complete fire process, the structure will not collapse due to the most dangerous combination of

1. dead load
2. snow load, multiplied by the load factor 1.2
3. live load, multiplied by the load factor 1.4

Calculation:

1. The dead load is to be calculated in the conventional way
2. For the snow load, the values to be applied for the static and mobile constituents are to be 80% of the values according to current building regulations
3. The following values are to be applied for the live load

Type of premises	Static load		Mobile load	
	kgf/m ²	{kN/m ² }	kgf/m ²	{kN/m ² }
<u>Buildings in which complete evacuation of people in the event of fire cannot be assumed with absolute certainty</u>				
Dwelling and hotel rooms, hospital wards, etc	35	{0.35}	70	{0.70}
Offices and schools(classrooms and group study rooms)	35	{0.35}	100	{1.00}
Shops, department stores, assembly halls (excl. records rooms and warehouses containing compact stacked loading)	35	{0.35}	250	{2.50}
<u>Buildings in which complete evacuation of people in the event of fire can be assumed with absolute certainty</u>				
Dwelling and hotel rooms, hospital wards, etc	35	{0.35}	35	{0.35}
Offices and schools (classrooms and group study rooms)	35	{0.35}	55	{0.55}
Shops, department stores, assembly halls (excl. records rooms and warehouses with compact stacked loading)	35	{0.35}	70	{0.70}

Table 2. Fire loads in various types of buildings and premises determined by means of statistical investigations. The values are referred to the total internal surface area of the fire compartments.

Type of building or premises	Fire load mean value		Fire load standard deviation		Design fire load. Fire load denoting the 80% level	
	Mcal/m ²	MJ/m ²	Mcal/m ²	MJ/m ²	Mcal/m ²	MJ/m ²
1 Dwellings^a						
2 rooms + kitchen	35.8	149.9	5.9	24.7	40.0	167.5
3 rooms + kitchen	33.1	138.6	4.8	20.1	35.5	148.6
2 Office buildings^{b, c}						
Technical offices (architects' offices etc)	29.7	124.4	7.5	31.4	34.5	144.5
Economic and administrative offices (banks, insurance companies, etc)	24.3	101.7	7.7	32.2	31.5	131.9
All the investigated offices taken together	27.3	114.3	9.4	39.4	33.0	138.2
3 Schools^b						
Junior level schools	20.1	84.2	3.4	14.2	23.5	98.4
Intermediate level schools	23.1	96.7	4.9	20.5	28.0	117.2
Senior level schools	14.6	61.1	4.4	18.4	17.0	71.2
All the investigated schools taken together	19.2	80.4	5.6	23.4	23.0	96.3
4 Hospitals	27.6	115.6	8.6	36.0	35.0	146.5
5 Hotels^b	16.0	67.0	4.6	19.3	19.5	81.6

^a The fire load due to floor coverings is not included in the values quoted.

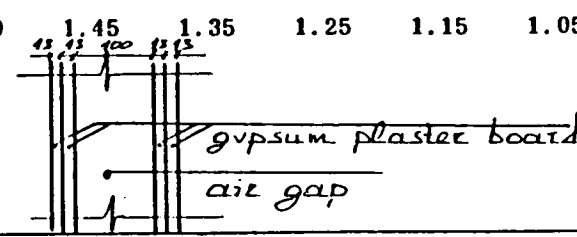
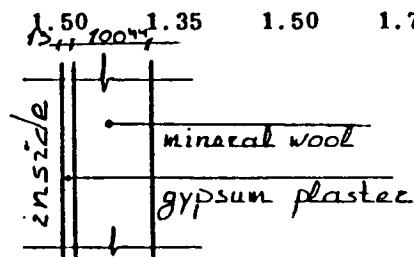
^b The values quoted apply only to the fire load due to furniture and fittings. Any additional fire load is to be calculated according to Equation (3.1 a) in the Main Section.

^c According to Swedish regulations, an entire office apartment is defined as a fire compartment. Since there were difficulties during the statistical investigation in determining the sizes of the fire compartments, the quoted values of the fire load apply to each office room. Furthermore, it is to be noted that office buildings are often constructed in such a way that each office room can be designated as an individual fire compartment. In Subsection 3.2.2 in the Main Section, distribution curves are also given for the fire load with reference to the floor area. These values can be used in determining the fire load per m² of the total internal surface area when division into fire compartments is arbitrary.

Table 3 . Factors for the conversion of the actual fire load and opening factor for different types of fire compartment^a to equivalent fire load and opening factor applicable to fire compartment Type A (standard fire compartment)

Equivalent fire load = $k_f \cdot$ actual fire load
 Equivalent opening factor = $k_f \cdot$ actual opening factor

Fire compartment		Factor k_f					
		Actual opening factor ($m^{1/2}$)					
Type	Description of enclosing construction	0.02	0.04	0.06	0.08	0.10	0.12
A	Thermal properties corresponding to average values for concrete, brick and lightweight concrete (standard fire compartment)	1.0	1.0	1.0	1.0	1.0	1.0
B	Concrete (100%)	0.85	0.85	0.85	0.85	0.85	0.85
C	Lightweight concrete (100%)	3.0	3.0	3.0	3.0	3.0	2.5
D	50 % concrete 50 % lightweight concrete	1.35	1.35	1.35	1.50	1.55	1.65
E	50 % lightweight concrete 33 % concrete 17 % { from the inside outwards, 13 mm gypsum plaster-board 100 mm mineral wool brickwork	1.65	1.50 <i>100%</i>	1.35	1.50	1.75	2.00
F ^b	80 % uninsulated steel sheeting 20 % concrete	1.0-0.5	1.0-0.5	0.8-0.5	0.7-0.5	0.7-0.5	0.7-0.5
G	20 % concrete 80 % { 2x13 mm gypsum plaster-board 100 mm air gap 2x13 mm gypsum plaster-board	1.50	1.45 <i>100%</i>	1.35	1.25	1.15	1.05
H	100 % { steel sheeting 100 mm mineral wool steel sheeting	3.0	3.0	3.0	3.0	3.0	2.5



Calculation procedure:

Determine the resultant emissivity ϵ_r

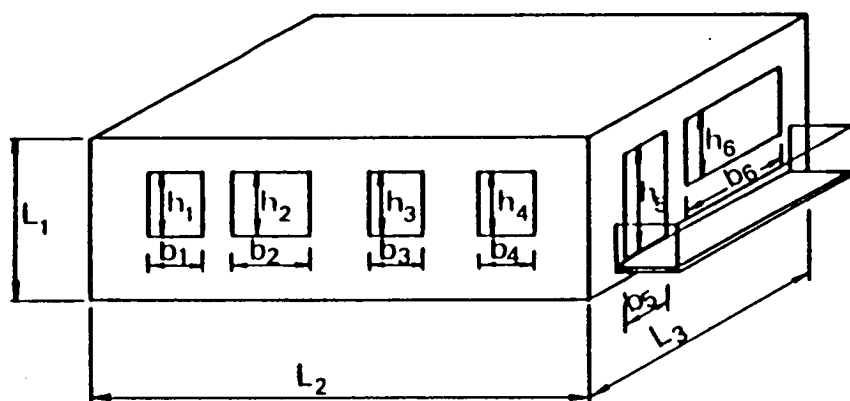
Determine the F_g/V_g ratio

Determine the maximum temperature θ_{max}

Table 4. Resultant emissivity ϵ_r for different constructions. The values yield results on the safe side (Main Section, Subsection 5.2.4)

Type of construction	Resultant emissivity ϵ_r
1 Column exposed to fire on all sides	0.7
2 Column outside facade	0.3
3 Floor girder with floor slab of concrete, only the underside of the bottom flange being directly exposed to fire	0.5
4 Floor girder with floor slab on the top flange ^a	0.5
Girder of I section for which the width-depth ratio is not less than 0.5	0.5
Girder of I section for which the width-depth ratio is less than 0.5	0.7
Box girder and lattice girder	0.7

^a More accurate values of ϵ_r which take into account the width-depth ratio b/h and spacing-depth ratio c/h of the girders are given in Fig. 5.2.4 b in the Main Section



$$A = A_1 + A_2 + \dots + A_6 - b_1 h_1 + b_2 h_2 + \dots + b_6 h_6$$

$$h = \frac{1}{A} [A_1 h_1 + A_2 h_2 + \dots + A_6 h_6]$$

$$A_r = 2 [L_1 L_2 + L_1 L_3 + L_2 L_3]$$

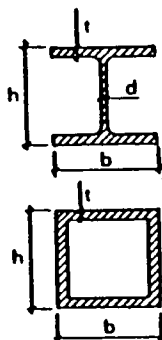
OPENING
FACTOR
 $\frac{A\sqrt{h}}{A_r}$

Fig. 2 Calculation of the opening factor for a fire compartment with vertical openings. L_1 , L_2 and L_3 denote the internal dimensions of the fire compartment

Table 5 The ratio F_g/V_g (m^{-1}) for rolled I girders for two different cases of exposure to radiation

Steel section	All surfaces of the steel section exposed to radiation	One side of one flange not exposed to radiation
HEA		
100	275	227
120	277	229
140	260	215
160	240	199
180	232	192
200	217	180
220	200	166
240	183	152
260	175	146
280	169	140
300	157	130
320	145	121
340	138	115
360	132	110
400	123	104
HEB (IPB)		
100	226	188
120	208	173
140	192	160
160	174	144
180	163	135
200	151	126
220	143	119
240	134	111
260	130	108
280	126	105
300	119	99
320	112	94
340	109	91
360	105	88
400	100	85
IPE		
80	440	380
100	400	346
120	369	321
140	343	299
160	317	277
180	298	260
200	277	242
220	260	227
240	242	212
270	231	203
300	220	193
330	205	180
360	190	167
400	178	157

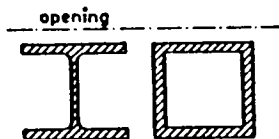
Column within a fire compartment



$$\frac{F_s}{V_s} = \frac{2h + 4b - 2d}{\text{cross section area}}$$

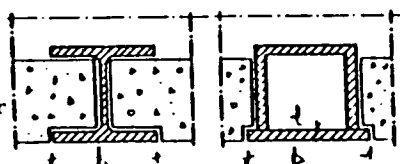
$$\frac{F_s}{V_s} = \frac{(2h + 2b)}{(\text{cross section area})}$$

Column, immediately outside a window opening



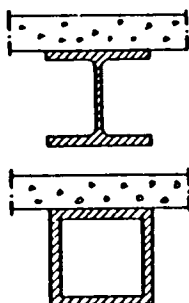
$$\frac{F_s}{V_s} = \frac{2h + b}{\text{cross section area}}$$

Floor structure, composed of steel beams with a concrete slab, supported on the lower flange of the beams



$$\frac{F_s}{V_s} = \frac{b}{bt} = \frac{1}{t}$$

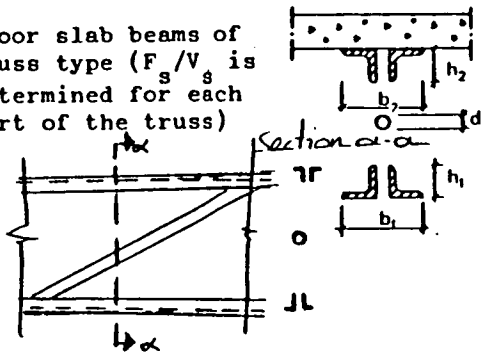
Beams with a floor slab, supported on the upper flange of the beams



$$\frac{F_s}{V_s} = \frac{2h + 3b - 2d}{\text{cross section area}}$$

$$\frac{F_s}{V_s} = \frac{2h + b}{\text{cross section area}}$$

Floor slab beams of truss type (F_s/V_s is determined for each part of the truss)



$$\frac{F_s}{V_s} \text{ (lower flange)} = \frac{2b_1 + 2h_1}{\text{cross section area of lower flange}}$$

$$\frac{F_s}{V_s} \text{ (upper flange)} = \frac{b_2 + 2h_2}{\text{cross section area of upper flange}}$$

$$\frac{F_s}{V_s} \text{ (diagonal)} = \frac{4}{d}$$

Fig. 3 Examples of calculating F_s/V_s (m^{-1}), the ratio of the area per unit length exposed to fire (m^2/m) to the enclosed steel volume per unit length (m^3/m) for different types of construction. See also Table 5 b (Main Section, Subsection 5.2.5)

q	AV _h /A _t	F _g /V _g	θ _{max}			q	AV _h /A _t	F _g /V _g	θ _{max}			q	AV _h /A _t	F _g /V _g	θ _{max}																																								
			ε _r	ε _r	ε _r				ε _r	ε _r	ε _r				ε _r	ε _r	ε _r	ε _r	ε _r																																				
			0.3	0.5	0.7				0.3	0.5	0.7				0.3	0.5	0.7	0.3	0.5	0.7																																			
10 {12}	0.01	50	325	315	370	15 {13}	0.01	50	400	420	440	20 {14}	0.01	25	390	425	445	25 {10}	0.01	25	155	490	500																																
		75	365	395	405			75	435	445	460			75	165	480	490			75	510	525	530	75	525	530	535																												
		100	395	410	425			100	450	460	470			100	185	500	500			100	530	535	535	100	530	535	535																												
		125	410	425	435			125	460	470	475			125	195	505	505			125	530	535	510	125	530	535	510																												
		150	425	435	440			150	470	475	480			150	200	505	510			150	535	510	510	150	535	510	510																												
		200	435	445	445			200	475	480	480			200	205	510	515			200	535	510	510	200	535	510	510																												
	12.5 {52.5}	0.01	50	335	390		410	17.5 {73.5}	0.01	50	425		490	615	22.5 {91.5}	0.01	25		500	550	575	30 {12}	0.01	25	555	600	625																												
			75	365	415		475			75	500		510	565			75		560	600	620			75	610	610	650	75	610	610	650																								
			100	415	490		520			100	510		575	595			100		565	620	630			100	610	650	655	100	610	650	655																								
			125	480	520		515			125	565		600	610			125		595	620	630			125	615	630	610	125	650	655	660																								
			150	500	510		555			150	585		605	615			150		605	620	625			150	625	610	645	150	650	615	700																								
			200	510	560		675			200	605		620	625			200		625	630	630			200	635	615	650	200	650	615	700																								
		15 {63}	0.02	50	335		320		365	15 {63}	0.02		50	400		455	510		20 {84}	0.02	25		410	455	510	25 {10}	0.02	25	355	420	510																								
				75	350		400		450				75	490		550	600				75		495	565	625			75	525	600	700	75	525	600	700																				
				100	405		460		510				100	550		610	655				100		585	650	700			100	650	700	740	100	650	700	740																				
				125	450		515		555				125	600		655	690				125		650	700	740			125	700	740	780	125	700	740	780																				
				150	495		555		595				150	635		680	710				150		700	740	780			150	750	790	830	150	750	790	830																				
				200	550		605		645				200	700		740	775				200		750	790	830			200	800	840	880	200	800	840	880																				
			17.5 {73.5}	0.01	50		285		320		365		17.5 {73.5}	0.01		50	310			400	475		22.5 {91.5}	0.01	25		310	400	475	30 {12}	0.01	25	310	400	475																				
					75		350		400		450					75	425			490	575				75		410	505	600			75	410	505	600	75	410	505	600																
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							75		350		400			450						75	425			490	575				75		410	505	600			75	410	505	600	75	410	505	600												
							100		405		460			510						100	500			550	630				100		510	610	700			100	510	610	700	100	510	610	700												
							125		450		515			555						125	550			600	680				125		550	650	740			125	550	650	740	125	550	650	740												
							150		495		555			595						150	600			650	730				150		600	700	790			150	600	700	790	150	600	700	790												
							200		550		605			645						200	650			700	755				200		650	750	840			200	650	750	840	200	650	750	840												
					22.5 {91.5}		0.01		50		200			250			300			22.5 {91.5}	0.01			50	260			290	400		22.5 {91.5}	0.01	25		430	460	480	30 {12}	0.01	25	505	525	510												
									75		235			275			330							75	290			340	430				75		450	505	515			75	515	535	560	75	515	535	560								
									100		305			370			425							100	350			375	430				100		490	560	670			100	545	560	560	100	545	560	560								
									125		365			410			485							125	400			465	535				125		530	375	500			125	560	565	565	125	560	565	565								
									150		415			450			515							150	450			515	605				150		580	480	610			150	590	595	595	150	590	595	595								
									200		485			485			580							200	525			525	785				200		620	620	775			200	620	620	775	200	620	620	775								
							25 {10}		0.01		50			200			250				300			25 {10}	0.01			50	260			290	400		25 {10}	0.01	25		430	460	480	30 {12}	0.01	25	505	525	510								
											75			235			275				330							75	290			340	430				75		450	505	515			75	515	535	560	75	515	535	560				
											100			305			370				425							100	350			375	430				100		490	560	670			100	545	560	560	100	545	560	560				
											125			365			410				485							125	400			465	535				125		530	375	500			125	560	565	565	125	560	565	565				
											150			415			450				515							150	450			515	605				150		580	480	610			150	590	595	595	150	590	595	595				
											200			485			485				580							200	525			525	785				200		620	620	775			200	620	620	775	200	620	620	775				
									25 {10}		0.02			50			200				250				300			25 {10}	0.02			50	260			290	400		25 {10}	0.02	25		430	460	480	30 {12}	0.02	25	505	525	510				
														75			235				275				330							75	290			340	430				75		450	505	515			75	515	535	560	75	515	535	560
														100			305				370				425							100	350			375	430				100		490	560	670			100	545	560	560	100	545	560	560
														125			365				410				485							125	400			465	535				125		530	375	500			125	560	565	565	125	560	565	565
														150			415				450				515							150	450			515	605				150		580	480	610			150	590	595	595	150	590	595	595
														200			485				485				580							200	525			525	785				200		620	620	775			200	620	620	775	200	620	620	775
30 {12}						0.01					50	170		200			260	30 {12}			0.01				50				210			260	350			30 {12}	0.01			25	430		460	480	30 {12}		0.01	25	505	525	510				
											75	220		260			350								75				240			280	390							75	460		505	515				75	515	535	560	75	515	535	560
											100	240		310			400								100				260			340	430							100	480		560	670				100	545	560	560	100	545	560	560
											125	260		340			510								125				280			360	450							125	500		600	710				125	560	565	565	125	560	565	565
											150	310		430			620								150				330			450	640							150	550		660	775				150	590	595	595	150	590	595	595
											200	390		500			760								200				430			550	810							200	620		740	855				200	650	660	665	200	650	660	665
	30 {12}					0.02		50			170	200		260	30 {12}		0.02				50	210			260				350			30 {12}	0.02				25			430	460		480	30 {12}			0.02	25	505	525	510				
								75			220	260		350							75	240			280				390								75			460	505		515					75	515	535	560	75	515	535	560
								100			240	310		400							100	260			340				430								100			480	560		670					100	545	560	560	100	545	560	560
								125			260	340		510							125	280			360				450								125			500	600		710					125							

DETERMINATION OF THE CRITICAL LOAD FOR STEEL COLUMNS UNDER FIRE EXPOSURE CONDITIONS

Calculation procedure:

Determine the degree of expansion if longitudinal expansion is restrained

Determine the critical load

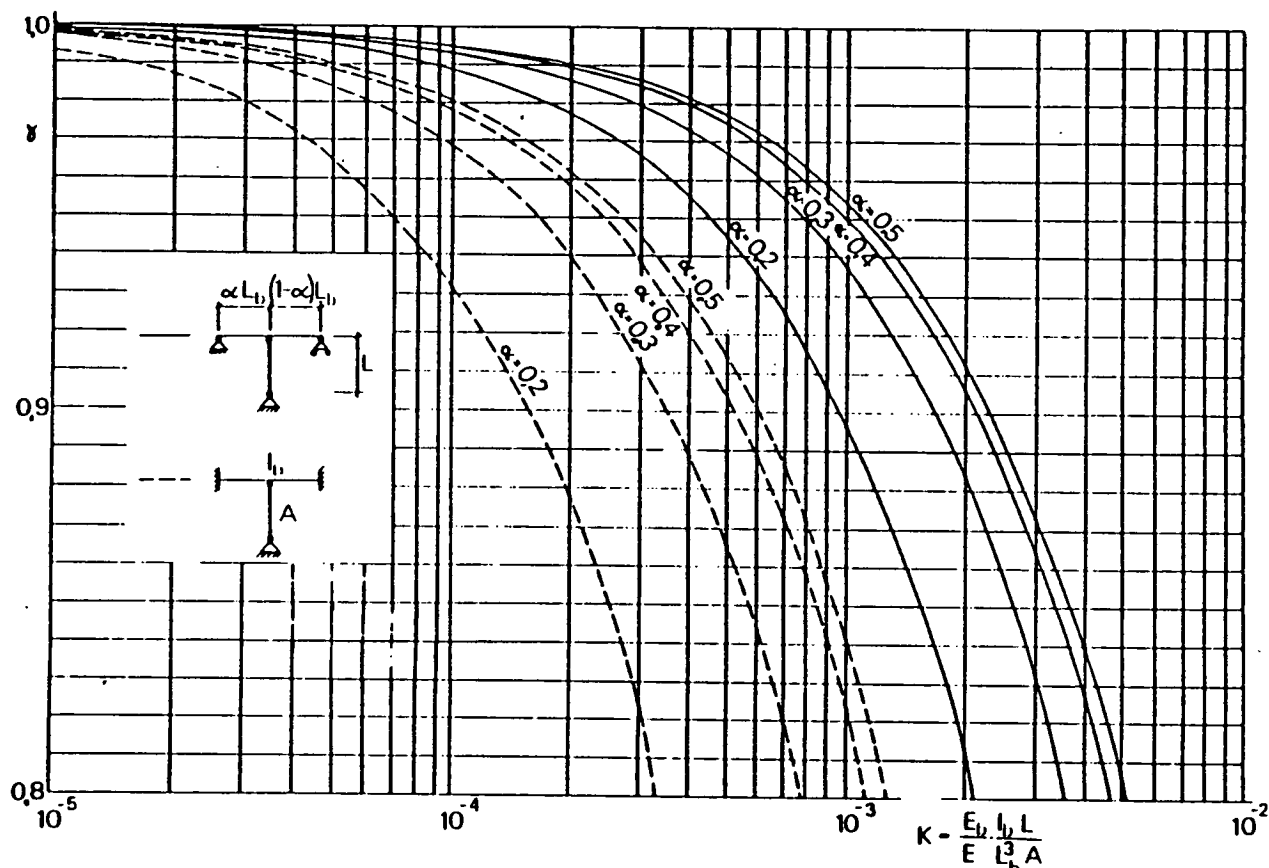
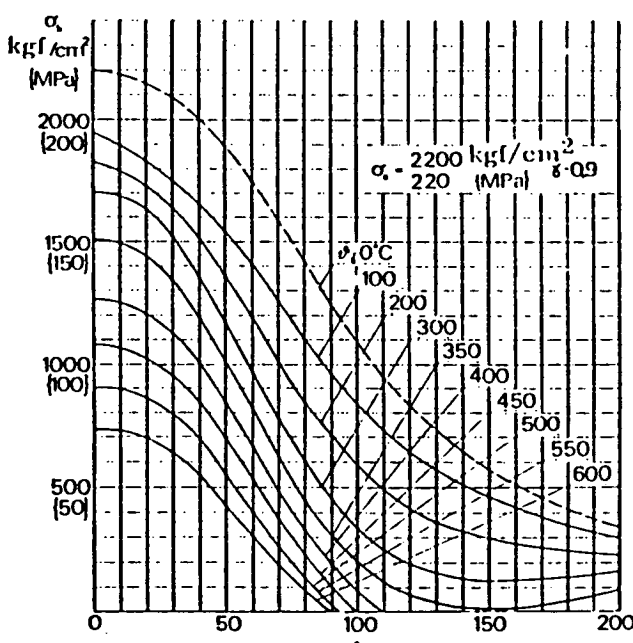
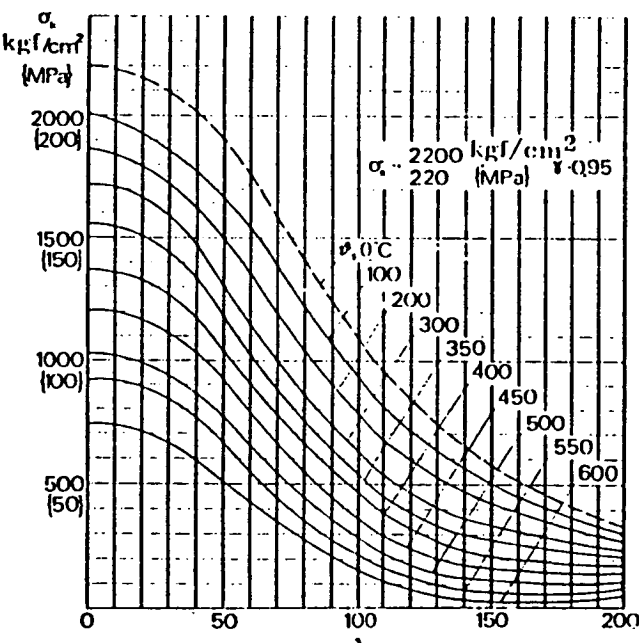
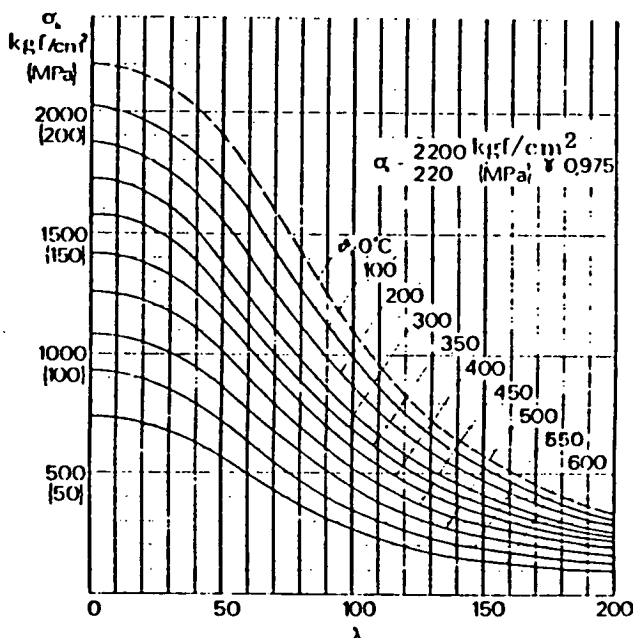
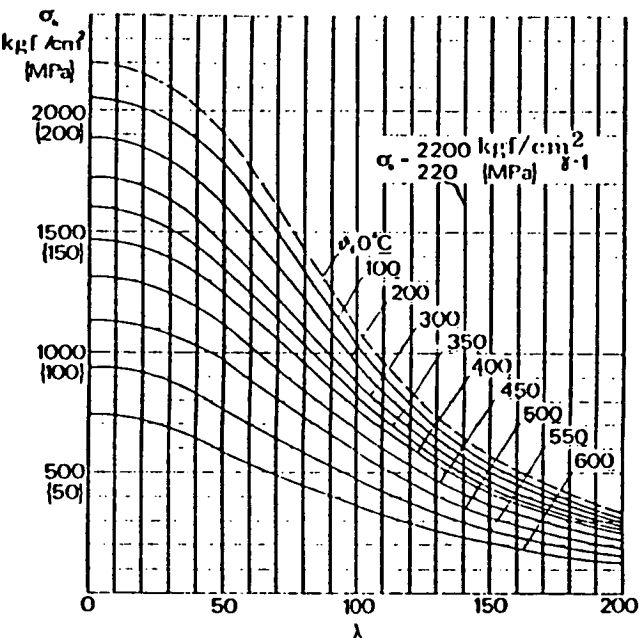
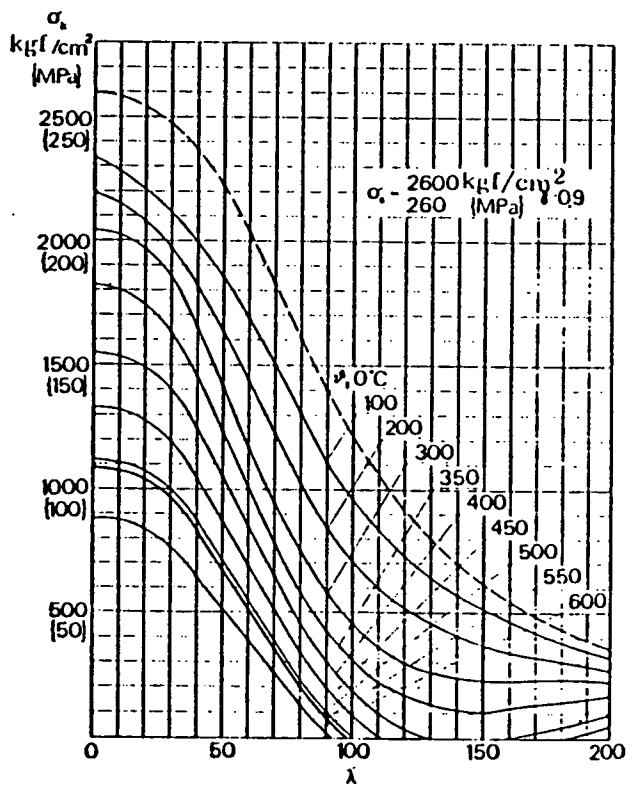
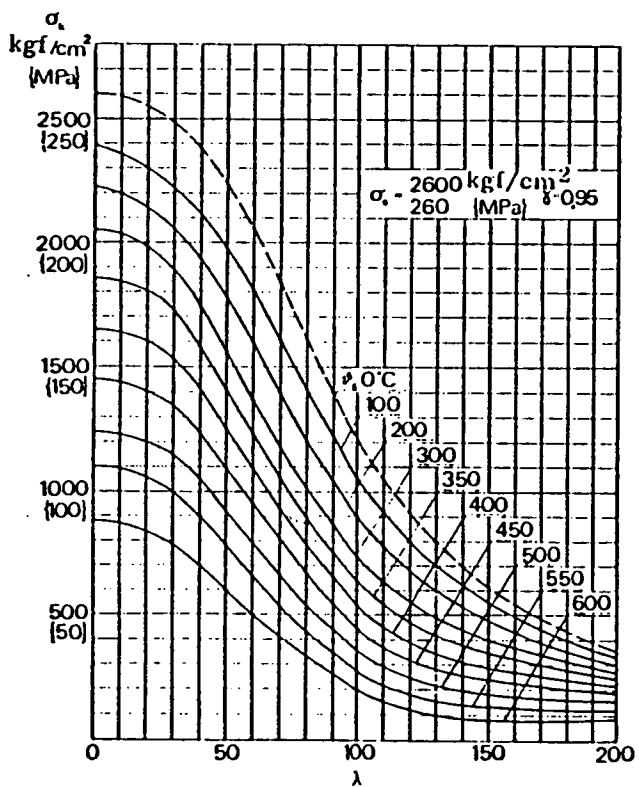
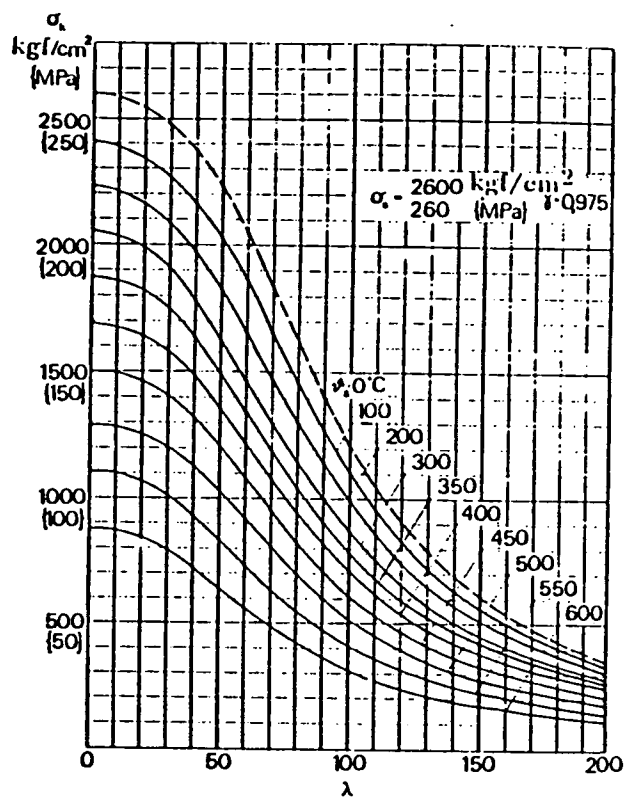
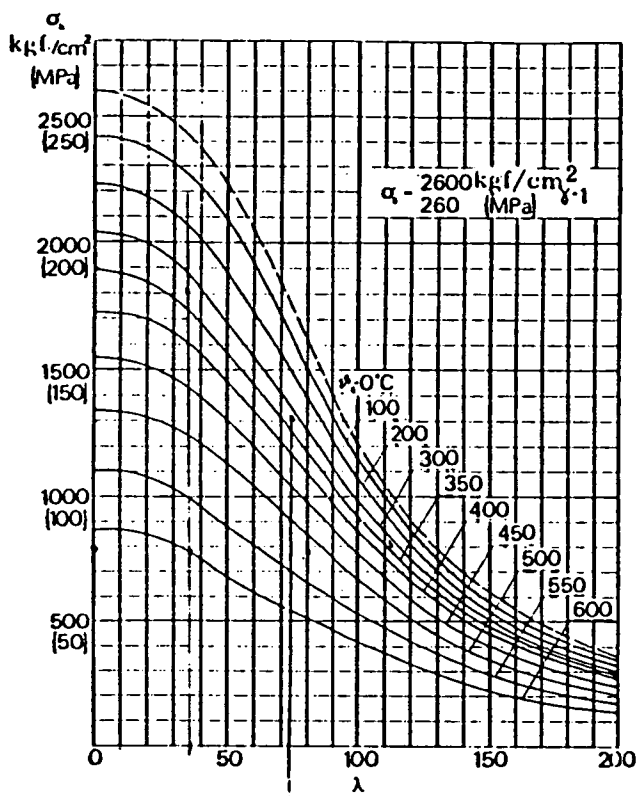
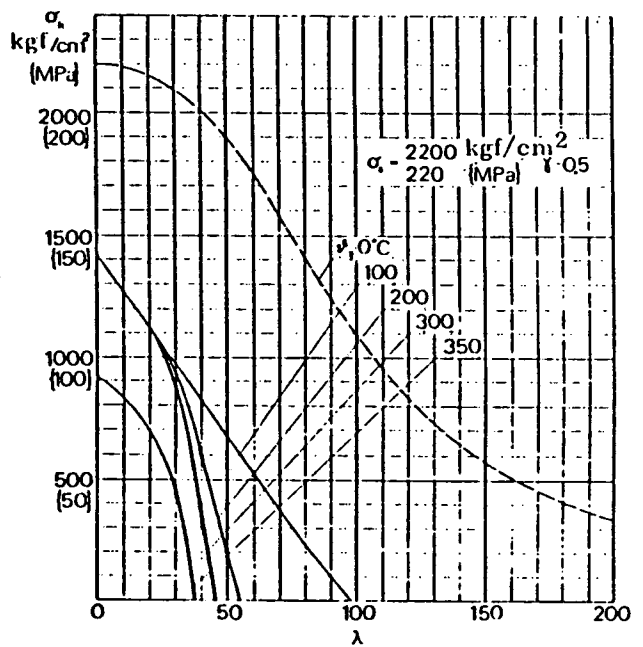
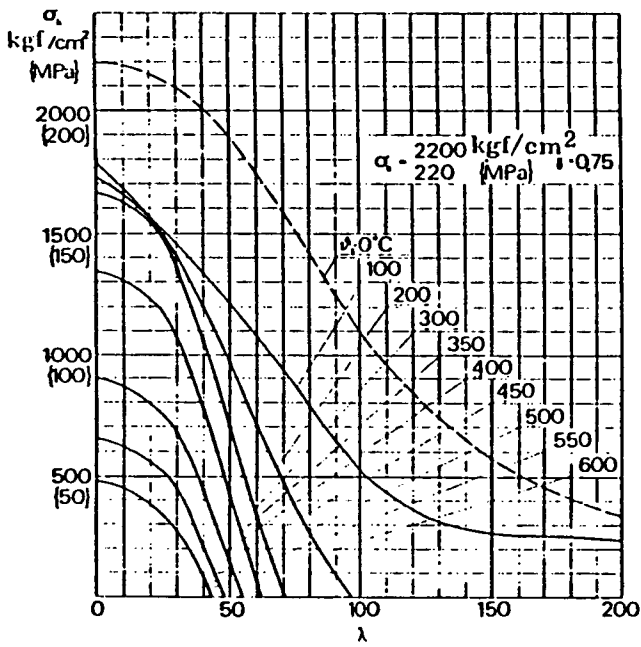
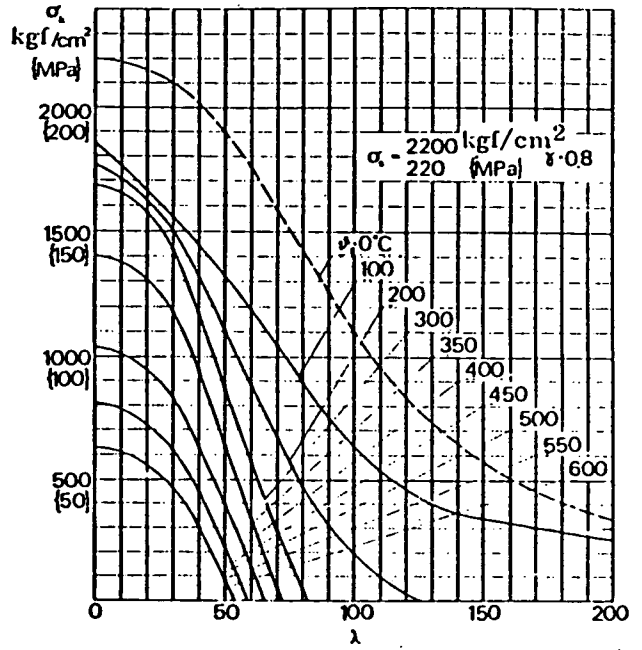
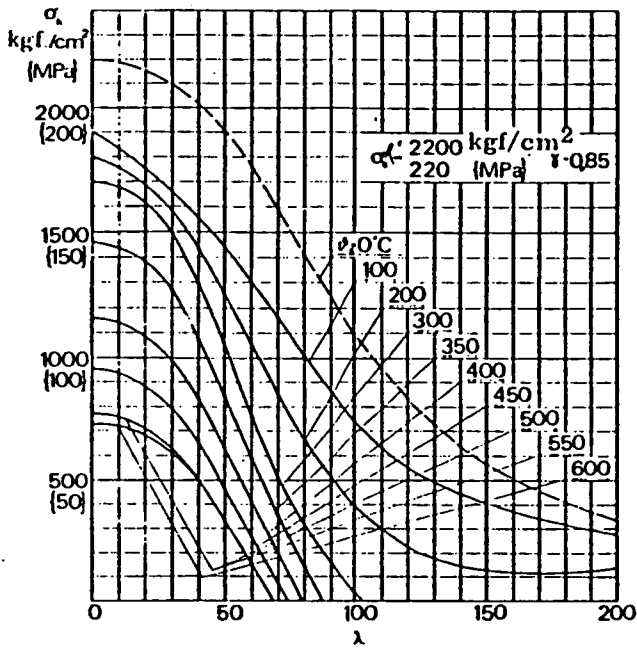


Fig. 4 The coefficient γ which indicates the degree of expansion under fire exposure conditions in columns connected to a simply supported beam and a beam rigidly restrained at both ends, respectively, as a function of the nondimensional parameter K . In the expression for K , E_b denotes the modulus of elasticity of the beam at the temperature concerned, (see Fig. 9.1 a in the Main Section), and E denotes the modulus of elasticity (secant modulus) of the column at the temperature and stress concerned, inclusive of the additional stress due to partial restraint on longitudinal expansion of the column (see Fig. 10.1 b in the Main Section). L_b and L denote the lengths of the beam and column, and I_b and A the moment of inertia of the beam and the area of column cross section, respectively

Fig. 5 Critical buckling stress σ_k as a function of the steel temperature θ_s and slenderness ratio λ for a column under fire exposure conditions, made of structural steel with a nominal yield stress at room temperature of $\sigma_s = 2200$ [220], 2600 [260] and 3200 kgf/cm² [320 MPa]. γ = coefficient indicating degree of expansion. When $\gamma \neq 1$, the cross sectional factor i/d influences the shape of the curves, but this influence is comparatively limited. The curves given have therefore been generally determined for $i/d = 0.5$, which for normal types of section results in design on the safe side. For determination of the critical load in a structure subject to simultaneous flexure and compression, see Section 10.3 in the Main Section. For determination of the critical load in a structure where there is a risk of out-of-plane buckling, see Section 10.4 in the Main Section







VULTEX

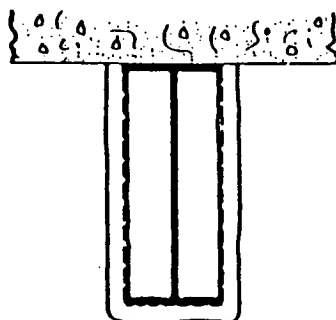
1. Product description	Sprayed Vermiculite	
2. Manufacturer	Hawk Business Services Limited Fire Protection & Insulation Division, Kings House, Kings Road, Newbury, Berks, RG14 5RA.	Tel: 0635 524272 Telex: 848507 IJULPII G Fax: 0635 35053
3. Availability	From approved applicators. Contact manufacturer.	
4. Protection technique	Profile, box and solid.	
5. Application technique	Factory prepared mix, mixed with water on site then spray or hand applied.	
6. Steel preparation requirements	Steel must be degreased and wire-brushed to remove loose scale. If steel is primed advice should be sought from the manufacturer.	
7. Additional mechanical fixing	None up to 65mm and 4 hour's fire resistance for 3 and 4 sided application. Mesh or equivalent may be needed in other cases.	
8. Nominal density	715 kg/m ³ applied.	
9. Thickness Range	10 – 80mm	
10. Fire resistance range	a) Up to 4 hours. b) Hp/A 26 – 310.	
11. Constraints for fire resistance	a) Minimum thickness 10mm. b) Maximum thickness unreinforced 65mm.	
12. Appearance	Light pink textured surface.	
13. On site use	For interior, semi-exposed (eg not subject to direct weather) exposed during construction period but may be required to be overcoated with Vultex Weather Shield.	
14. Durability	Resistant to frost, vermin and mould growth and limited attack by water.	
15. Performance in other BS fire tests	Non combustible in accordance with BS 476: Part 4, so complies with class O as defined in the Building Regulations, 1985.	
16. Other applications	a) Thermal insulation b) Sound insulation c) Fire protection of ductwork d) Fire protection of concrete e) General fire protection – internal linings to external walls, cavity barriers etc.	

H _p /A up to	Dry thickness in mm to provide fire resistance of					
	½ hour	1 hour	1½ hour	2 hour	3 hour	4 hour
30	10	10	14	18	26	35
50	10	12	17	22	33	43
70	10	13	19	25	37	48
90	10	14	21	27	39	52
110	10	15	22	28	41	54
130	10	16	22	29	42	56
150	10	16	23	30	44	57
170	10	16	23	30	44	59
190	10	16	24	31	45	59
210	10	17	24	31	46	60
230	10	17	24	32	46	61
250	10	17	24	32	47	61
270	10	17	25	32	47	62
290	10	17	25	32	47	62
310	10	17	25	32	48	63

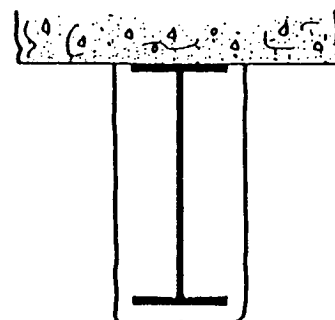
Linear interpolation is permissible between values of H_p/A



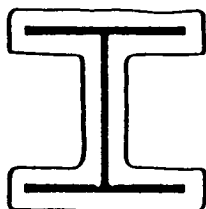
Profile



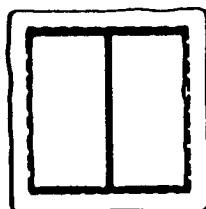
Box



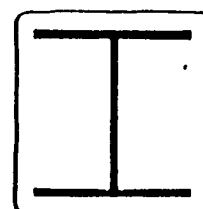
Solid



Profile



Box



Solid

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