University of Southern Queensland

Faculty of Engineering and Surveying

Design of Steel Structures under Fire

A dissertation submitted by

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in fulfilment of the requirements of

Courses ENG4111 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Civil)

Submitted: October, 2005

Abstract

The Australian Steel Design Code, AS4100 provides guidance for design of steel elements. The fire section of the code details the procedure necessary to determine the time and temperature of a steel member to fail under fire conditions. Researchers all over the world have conducted studies and have reported their findings and possible design criteria's to use in the prediction of the temperature rise of steel when subjected to elevated temperatures.

The material properties that affect the behaviour of structural steel members exposed to fire have been reviewed and they are the thermal, mechanical and deformation properties of steel which change at elevated temperatures.

This report describes the design methods used for design of steel building elements required to have a fire resistance level in accordance with the Australian Steel Code AS4100.1998. This report also contains sample design calculations to predict failure of simply supported unprotected beams subjected to elevated temperatures.

The review of AS4100 indicates that a simple method is used for estimating temperatures for unprotected steel elements, whilst standard fire test curves are to be used for estimating temperatures for protected steel elements.

Structural behaviour of steel structures in fire depends upon a number of variables such as material degradation at elevated temperature and restraint stiffness of members surrounded by fire. High temperatures and gradients across a structural element are the driving force behind large deflections and axial forces. In buildings exposed to fire they all interact, thus influencing the structural behaviour.

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Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Signature

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Date

Acknowledgement

I am very grateful for all the assistance given to me, without which this study would have been very difficult to complete. Dr. A. Khennane, my supervisor, suggested the research topic and the methods for analysing the problems. Thanks are due to staffs of University of Southern Queensland who have helped by providing assistance for this study.

My appreciation is also extended to my friends at University of Southern Queensland who directly or indirectly contributed to the success of this study. Deep appreciation is also extended to my mother and brother for their continuous moral support, encouragement and help.

Above all, I am very grateful to the almighty God for the blessing rendered upon me, which led to the successful completion of this study.

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NOMENCLATURE

The following notations listed below have been used in equations in this report.

α	convective heat transfer coefficient (kW/m^2K)
r_f	reduction factor for flexural buckling in fire design situations
ε	Emissivity
φ	configuration factor
k_{sm}	exposed surfaced area to mass ratio
σ	Stefan-Boltzmann constant
q	convective heat transfer rate
A	cross-sectional area of the steel (m^2)
An	net area of the ventilation openings (m^2)
c_p	specific heat of the steel (J/kg K)
H_p/A	Section Factor (m^{-1})
fy	yield stress (MPa)
H_p	heated perimeter of steel (m)
k	thermal conductivity (W/mK)
$k_o – k_6$	regression coefficients
w_c	uniformly distributed load (UDL)
M_f	design member capacity in bending at elevated temperatures (kN.m)
N_{sf}	design axial capacity at elevated temperatures (kN)
N_{tf}	design tension capacity at elevated temperatures (kN)
Q	live load (kN)
G	dead load (kN)
T	time (minutes)
T	temperature (°C)
T_l	limiting temperature (°C)
V_f	design shear resistance for elevated temperatures (kN)
V	design shear resistance for normal temperatures (kN)
Z_e	plastic section modulus (mm^3)

1 Introduction

1.1 General

Fire is destructive; it causes injury, death and loss of property followed by negative environmental consequences. Therefore, design of structures should incorporate measures to mitigate or prevent destruction of the structure whilst safeguarding safety issues related to human occupancy.

Steel elements are commonly utilised for structures in the building and construction industry given its strength properties. However, it has relatively low resistance to elevated temperatures thus causing failure of the overall structure. The expected behaviour is dependent upon the severity of the fire, material properties and the degree of protection provided. Therefore, studying the behaviour of steel structures under fire becomes an important issue.

1.2 Aim

The aim of this research project is to:

a) Carry out a literature review on the behaviour of structural steel at elevated temperatures,
b) Analyse the structural behaviour of steel beams and columns under fire and,
c) Review the provisions of design code AS4100 and design of simple steel elements such as beams and columns for a fictitious standard fire.

1.3 Dissertation Overview

Chapter 1 of the project highlights the aim of research; Chapter 2 details the properties of structural steel at elevated temperatures. Chapter 3 enlightens the behaviour of structural steel elements exposed to fire; Chapter 4 outlines the provisions of Australian Steel Design Code for fire resistance levels. Chapter 5 encompasses the design of single simply supported beams and columns that will fail when the load capacity is reached and exceeded at one critical point of the span, and causes failure of the steel member. Finally, Chapter 6 outlines the conclusions and recommendations

2 Properties of Steel at Elevated Temperatures

During a fire, steel whether in the form of beams, columns or any structural member for that matter is exposed to hot gases from the fire and the exposure will depend upon the configuration of the structural member. For example, an unprotected column is likely to be exposed on all four sides whereas a beam supporting a floor may only be exposed on the bottom flange and or sides depending upon whether it is buried in the supported floor system (Lemont 2001).

2.1 Forms of Heat Transfer

2.1.1 Conduction

Conduction is a form of heat transfer involving interactions between the free electrons within a solid material (Buchanan, 2001). In fire situations, heat transfer by conduction mainly occurs in protected steel members since convection and radiation heat transfer is not permitted due to the members having some fire protection. Conduction is considered not important for unprotected steel where radiation heat transfer is dominant.

2.1.2 Convection

Convection occurs from the movement of fluids, either liquid or gases that are at different temperatures resulting in different densities, thus allowing flame spread. Conductive heat transfer takes place when a higher temperature fluid transfers energy to lower temperature surface at point of contact whilst in motion (Buchanan, 2001). In fire situations, convection involves hot gases from flame spreading through or around a solid material that is initially cool and transferring heat energy to it. The rate of heating depends on the velocity of the fluid at the surface and the thermal properties of the fluid and the solid.

The general formula for convective heat transfer, q, is given by:

$$q = h \, \Delta T \qquad 2.1$$

where h is the convective heat transfer coefficient which depends on the geometry of the solid surface, the flow mature and boundary layer thickness. A typical value for h in standard fires is 25 W/m^2K (Buchanan, 2001).

2.1.3 Radiation

Radiation is the strongest mechanism of heat transfer in fires because the energy transferred between two bodies is related to the temperature of the emitting (fire) and receiving surfaces (steel members). Radiation is the transfer of energy through electromagnetic waves and objects exposed to the radiation source absorb these energy. Unlike conduction and convection, heat can travel by radiation through a vacuum, or transparent solid or liquid although on earth there is always a medium, air through which radiation must travel (Buchanan, 2001).

The general formula for radiation (resulting heat flow) between an emitting surface and a receiving surface is:

$$q = \varphi \, \varepsilon \, \sigma \, (T_e^4 - T_r^4) \qquad 2.2$$

where T_e is the absolute temperature of the emitting surface (K), and T_r is the absolute temperature (K) of the receiving surface.

The emissivity, ε, indicates the efficiency of the emitting surface and it ranges from zero to 1.0. In fire situations having luminous flame and hot surfaces, emissivity ranges from 0.7 to 1.0 but it can change depending on the receiving surfaces. Buchanan (2001), mentions that emissivity can change during fire because it depends on type of steel exposed i.e. a zinc coated steel member will have a very low emissivity unit. The temperature reaches about 400ºC where the zinc coating melts and exposes the bare steel surface to fire.

The configuration factor φ is a measure of the amount of emitter (fire) seen by the receiving surface. Buchanan (2001) states that the configuration factor depends on the fire situation and these values are obtained from textbooks on heat transfer.

2.2 Thermal Properties of Steel

The thermal properties of steel indicate that steel changes with varying temperature. The strength or the load bearing capacity of steel decreases dramatically with an increase in temperature experienced in a fire. Although the thermal conductivity, specific heat, and density of steel vary with temperature, these differences do not have great effect on the strength of steel.

2.2.1 Specific Heat

Of the thermal properties of steel, the specific heat varies according to temperature and this variation is shown below. Figure 2.1 shows the peak specific heat value of steel to be about 730°C and this is due to metallurgical change of steel at this temperature.

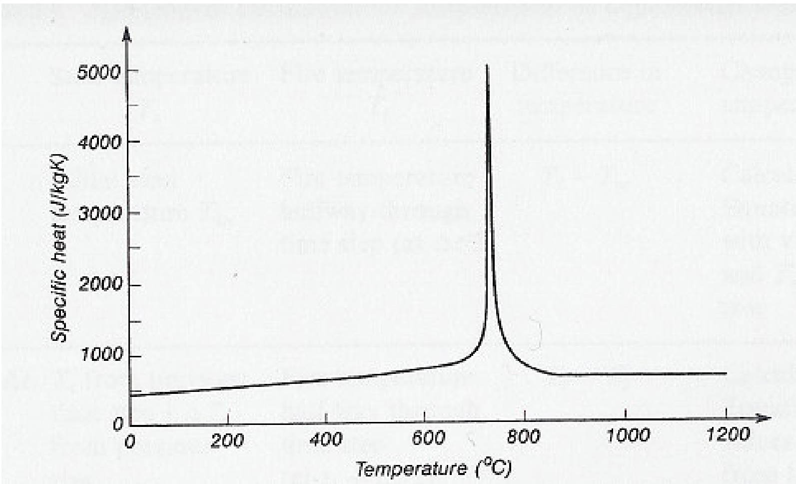

Figure 2.1 Specific heat of steel as a function of temperature
Adopted from Buchanan (2001)

Buchanan (2001) and Lie (1992) suggested that the specific heat c_p of steel can be taken as 600J/kgK for simple calculations but to obtain a more accurate value the equations below should be used. The reason being that in real fire situations steel temperature is greater than the peak value of 730°C.

The equations proposed by Buchanan (2001) are shown below and are applied to various ranges of steel temperatures T in °C.

$c_p = 425 + 0.773\,T - 1.69 \times 10^{-3} T^2 + 2.22 \times 10^{-6}\,T^3$	$20° \mathrm{C} \leq T < 600°\mathrm{C}$	2.3
$= 666 + 13002/(738 - T)$	$600° \mathrm{C} \leq T < 735°\mathrm{C}$	
$= 545 + 17820/(T - 731)$	$735° \mathrm{C} \leq T < 900°\mathrm{C}$	
$= 650$	$900° \mathrm{C} \leq T \leq 1200°\mathrm{C}$	

2.2.2 Thermal Conductivity

The thermal conductivity of steel varies according to temperature of the steel. There are slight variations between different grades of steel, but they are not significant. Buchanan (2001) states that for simple calculations the thermal conductivity can be taken as 45 W/mK, but for more accurate calculations the equations given below are recommended.

$k = 54 - 0.0333\,T$	$20°\mathrm{C} \leq T < 800°\mathrm{C}$	2.4
$= 27.3$	$800°\mathrm{C} \leq T \leq 1200°\mathrm{C}$	

Figure 2.2 below shows the variation of thermal conductivity of steel with temperature change. It can be noted from the figure that there is a linear reduction in thermal conductivity of 54 W/mK to 27.3 W/mK for a range of 0ºC to 800ºC and remains at a constant value of 27.3 W/mK after 800ºC (Buchanan 2001).

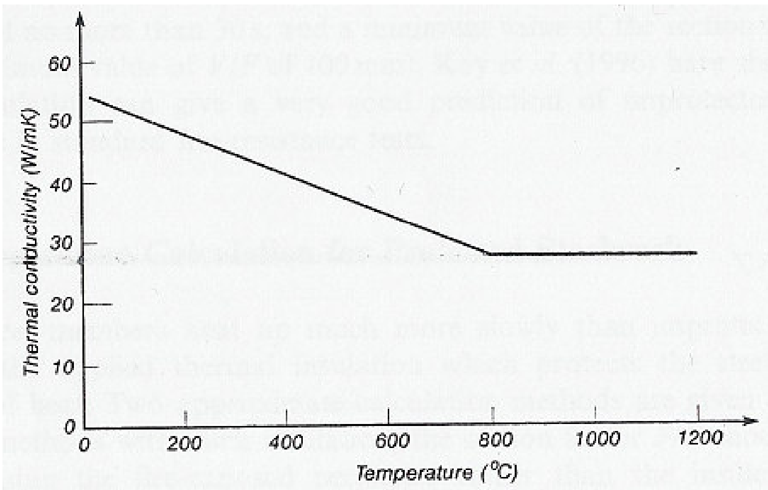

Figure 2.2 Thermal conductivity of steel as a function of temperature
Adopted from Buchanan (2001)

2.2.3 Density

As mentioned by (Buchanan 2001), the density of steel is to remain at a constant value of 7850 kg/m^3 for all temperatures during a fire.

2.3 Section Factor

The term section factor is the ratio of the heated perimeter to the cross sectional area of a steel member. The temperature variation on the heated perimeter of a steel member varies depending on the fire protection if any applied to the member. As mentioned by Buchanan (2001), this ratio is important in design calculations because it gives an indication of the effective sectional area of the steel member in relation to rate of heating since it is directly proportional.

The section factor is expressed as Hp/A, where H_p is the heated perimeter of the cross section (m) and A is the cross sectional area of the section (m^2). To obtain a ratio of heated surface area to volume, we use F/V, where F is the surface area of unit length of member (m^2) and V is the unit length steel volume of the member (Buchanan, 2001). Section factor tables are readily available from steel manufacturers and steel codes such as Eurocode3. Figure 2.3 below demonstrates the shape factor calculations for beams with different types of orientation and protection applied. For example if a member is exposed to fire on less than four sides, the ratio can be calculated according to the table. Buchanan (2001), also states that for surface area calculations, the protective material thickness should be deducted to obtain a more accurate value.

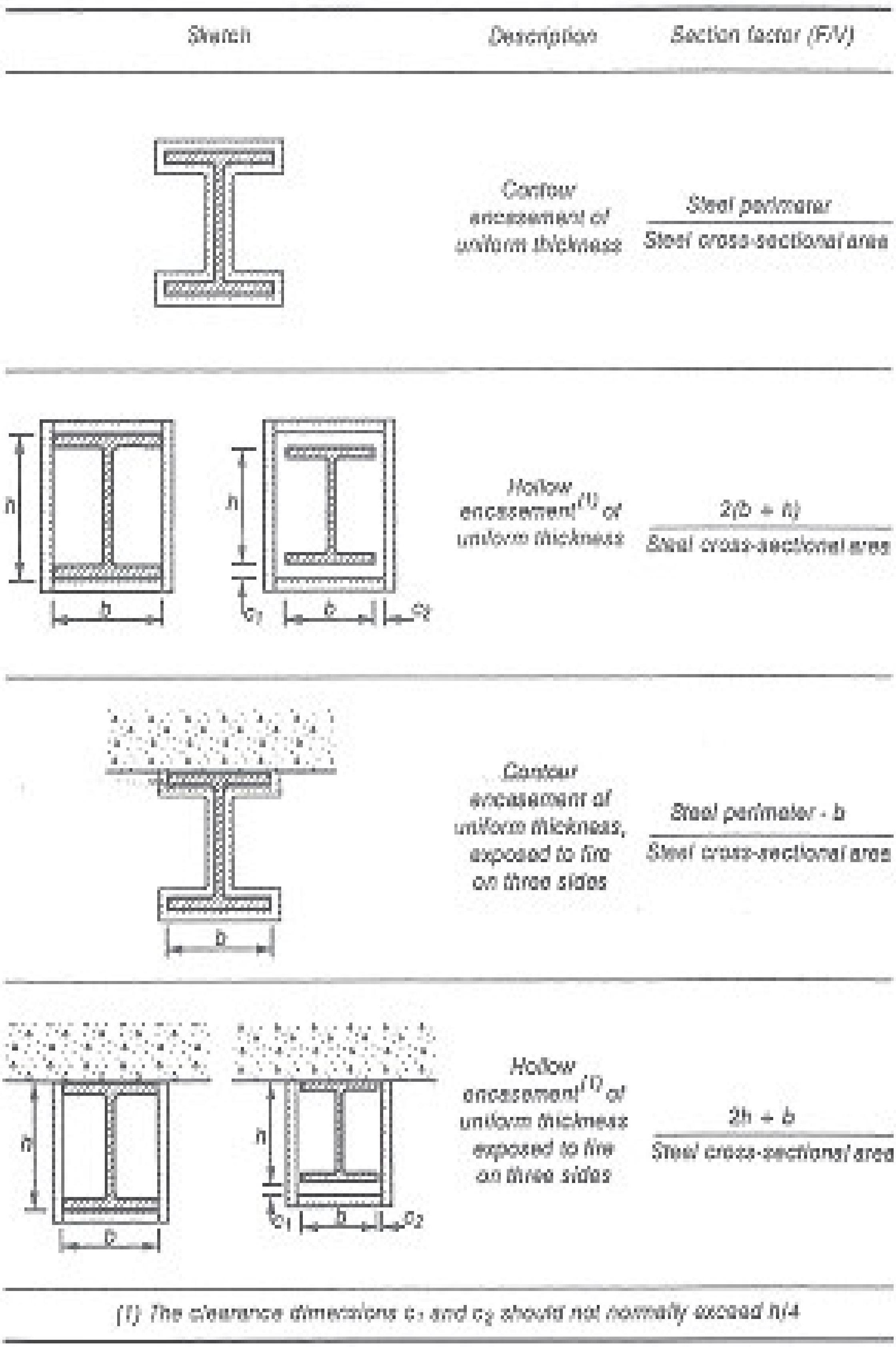

Figure 2.3 Vaiations of Hp/A for different methods of protection
Adopted from Buchanan (2001)

2.4 Mechanisms of Protection

There are many passive fire protection systems available to reduce the rise in temperature of steel members when exposed to elevated temperatures in a fire situation. Buchanan, (2001), states that fire resistance rating of a protected steel member although determined by calculations and depends on factors such as properties of protection material and fire temperature, there has to be some assurance of the fire resistance rating. This usually is achieved by full-scale testing of the structural system incorporating fire protection material, thus validating the effectiveness of the protection material used for specified fire duration in a real fire situation.

Protection systems commonly used to increase the fire resistance rating of steel members are listed below and briefly explained (Buchanan, 2001).

a) Concrete encasement,
b) Board systems,
c) Spray- on systems,
d) Intumescent paints,
e) Timber encasing,
f) Concrete filling,
g) Water filling, and
h) Flame shields.

Concrete encasement involves pouring of concrete in the formwork housing the steel members. Reinforcement is provided to hold concrete in place during a fire situation and the required thickness of the concrete is determined from the design codes. A certain disadvantage of this form of protection is that it results in increased construction costs and bulky structural members.

Board systems are mainly developed using calcium silicate or gypsum plaster. Calcium silicate boards are made of an inert material that is designed to remain in place during the duration of the fire. Gypsum boards have good insulating properties as well, and its resistance in fire is enhanced by the presence of water in the board which vaporise in elevated temperatures. This reaction provides a time delay when the board reaches about 100 °C, but reduces the strength of the board after exposure to fire. Advantages of this form of protection system are that it is easy installation and finishing enhancing the aesthetic aspects of design.

Spray-on protection system is usually the cheapest form of fire protection for steel members. Materials used for this method usually are cement-based with some form of glass or cellulosic fibrous reinforcing to hold the material together. The disadvantage of this method is that the application is a wet and messy one and the finished work is not aesthetically attractive. This form of fire protection is usually applied to beams rather than columns because it can be easily damaged due to soft material composition. Structural components such as bolts, steel brackets are likely to be protected with the spray-on protection system because other forms of protection might be difficult.

Intumescent paint is a special paint that swells into a thick char when it is exposed to elevated temperatures enhancing the fire rating of the steel member beneath. The advantage of this protecting system is that the application is a quick process, is less bulky and the member can be simply painted over thus not deteriorating the appearance of the steelwork. The disadvantage being that it is more expensive than other systems such as board and spray-on systems.

Using timber boards to encase structural members is another method of fire protecting system. The timber used has to be well seasoned and a thermosetting adhesive are usually used to firmly fix the boarding over the structural members.

Concrete filling is mainly used for hollow steel sections to improve their fire performance. An advantage of the system is that external protection is not required and can increase the load bearing capacity of that member. The infill concrete can be reinforced or be in the form of plain concrete.

Water filling system works in a similar principle to concrete filling where hollow steel sections are filled with water. The in filled water has some additives added in order to prevent corrosion. This form of protection requires plumbing systems to ensure water will flow in the members by convection and excessive pressure is not developed by heated water. It is only used in special structures and is considered expensive when compared with other systems.

Flame shields are used to protect external structural steelwork from radiation by flames exiting through the window openings. Usually architectural claddings are installed to form the shields.

2.5 Mechanical Properties of Steel at Elevated Temperatures

When a structural component is exposed to fire, it experiences high temperature gradients and stress gradients, which varies with time. Steel has a limited strength, meaning that at a certain temperature the strength of the member will decrease to virtually zero. The mechanical properties of steel vary with temperature, generally decreasing as the temperature of the steel increases. As the structural member is subjected to heat, the mechanical properties such as tensile and yield strength, and modulus of elasticity, decrease. If the yield stress decreases to the working stress, the element will fail. The steel temperature is the critical temperature at that point. The critical temperature of steel is often taken as approximately 540ºC, but varies depending upon the type and size of the steel member (Lemont 2001).

2.5.1 Components of Strains

The deformation of steel at elevated temperature is described by assuming that the change in strain consists of thermal strain, creep strain and mechanical or stress-related strain.

2.5.1.1 Thermal Strain

When a steel member is heated, it undergoes thermal expansion. This expansion is in a linear form and the equation to approximate this expansion is:

$$\Delta L/L = 14 \times 10^{-6} (T - 20) \qquad 2.5$$

where the temperature of the steel, T, is in °C. Buchanan (2001), states that the effects of thermal expansion is usually not necessary for design of simple members such as single beams and columns, mentioning that if thermal restraint forces evolve in beams it will be advantageous for the beam in terms of fire performance, however this expansion behaviour will cause an increase in the axial loading of the columns. He also mentions that thermal expansion must be considered for frame structures and complex structural systems where members are restrained by other structural components. The reason being that thermal expansions induce large internal forces which can speed up the rate of structural failure in a fire situation. Lie (1992), however reports that due to thermal expansion, the structural integrity of a structure exposed to elevated temperatures deteriorates and the expansion and contraction of members should be taken into consideration for all design cases.

2.5.1.2 Creep Strain

The term creep describes long-term deformation of materials under constant load. Under most conditions, creep is only a problem for members with very high permanent loads. Creep is relatively insignificant in structural steel at normal temperature. However, it becomes very significant at temperatures over 400 or 500°C and is highly dependant on temperature and stress level as shown below (Buchanan, 2001). Figure 1.4 shows the creep properties of steel tested in tension. It can be seen that as temperature increases the creep deformations in steel increases which can accelerate rapidly leading to plastic behaviour. Buchanan, (2001) mentions that creep strain is not usually included explicitly in design calculations because of lack of data and difficulties in calculations. The effect of creep is usually allowed for by using stress-strain relationships that include an allowance for creep that might be expected in a fire-exposed member.

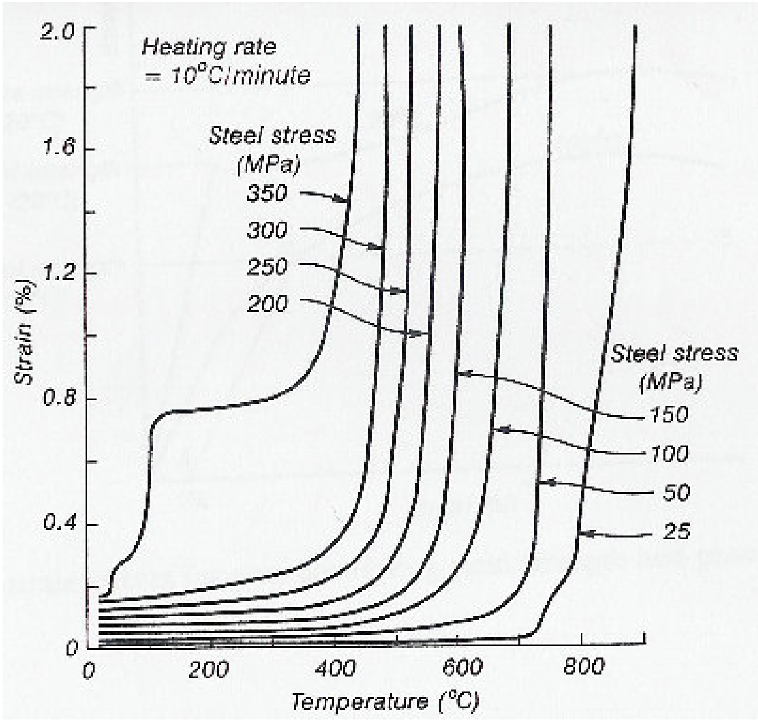

Figure 2.4 Creep of steel in tension
Adapted from Buchanan, (2001)

2.5.1.3 Stress-Related Strain

This form of strain is developed from stresses in a structural member under normal conditions or when exposed to fire. Figure 2.5 shows the stress-strain curves for typical- hot rolled steel at elevated temperatures. It can be noted that the steel suffers a progressive loss of strength and stiffness as temperature increases. The yield strength of structural steel members at normal temperatures is well defined, but this disappears at elevated temperatures. Figure 2.6 shows the stress-strain curves for typical steel illustrating yield strength at normal temperature and a softer curve at elevated temperatures. Buchanan (2001), states that since a value of yield strength is required for design at elevated temperatures, some researchers recommend using 1% proof stress as the effective yield strength. The modulus of elasticity is needed for buckling calculations and for elastic deflection calculation, but these are rarely used under fire conditions because elevated temperatures lead rapidly to plastic deformations (Buchanan, 2001).

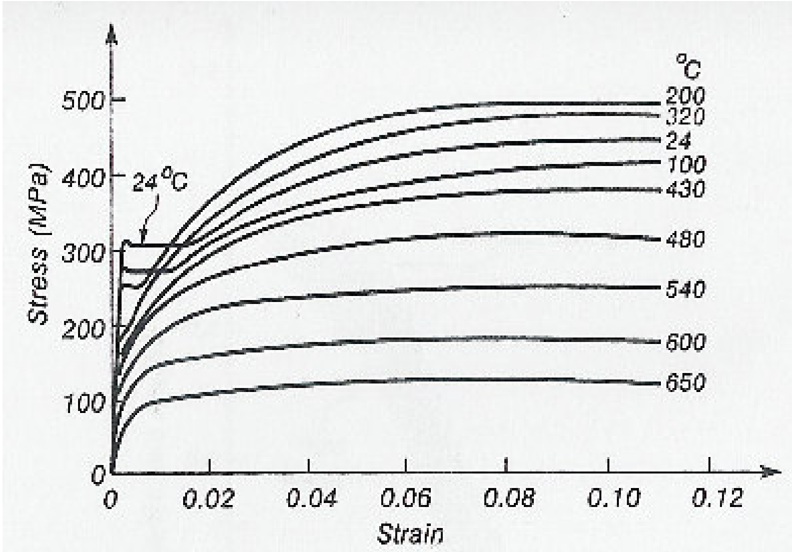

Figure 2.5 Stress-strain curves for typical hot rolled steel at elevated temperatures
Adapted from Buchanan, (2001)

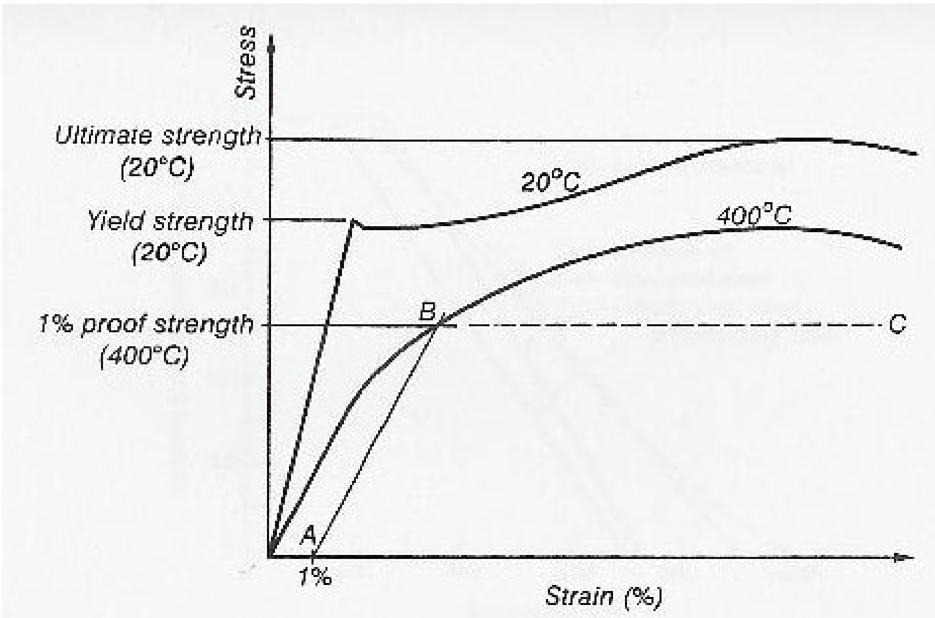

Figure 2.6 Stress-strain curves for steel illustrating yield strength and proof strength
Adapted from Buchanan, (2001)

Buchanan (2001), states that for the design of individual structural members such as simply supported beams that are free to expand during heating, the stress-related strain is the only component that needs to be considered. He mentions that if the strength reduction with temperature is known, member strength at elevated temperature can easily be calculated using simple formulae. The stress-related strains in fire exposed structures may be well above yield levels, resulting in extensive plastification, especially in buildings with redundancy or restraint to thermal expansion.

3 Behaviour of Structural Elements

Mechanical, physical, chemical and thermal properties of materials can all be affected by fire. Structural behaviour of steel structures when subjected to fire depends upon a number of variables such as material degradation at elevated temperature and restraint stiffness of the structure around the fire. High temperatures gradients in structural elements are the driving force behind large deflections and axial forces. For buildings when exposed to fire, they all interact, thus influencing the stability of a building. This leads to failure of structural elements and ultimately failure of the building.

The behaviour of beams and columns under the influence of fire has been investigated over many years but with increased intensity in the 80s. At this time, extensive testing was conducted in the UK, Germany, Netherlands, France and Belgium. The test results obtained have been extensively used by researchers for comparison with numerical models.

Lamont (2001) reported that heating rate of a steel section when exposed to fire is dependent on the size, shape and the location of the member in the building and the thickness and nature of any protection applied. The location of members in relation to the spread of heat is very important. Typical locations that can give widely varying heating rates include; a column placed inside a room and exposed to flames on four sides, a column placed outside a building, a beam that either is protected by a suspended ceiling or is high enough above the fire so that the upper limit of the flames is below the bottom flange, a beam supporting a floor slab in which the flames reach the underside of the slab, embedded columns and beams, cross sectional shapes e.g. circular or H-section.

3.1 Beam Analysis

Bennetts and Thomas (2002) reported that lateral buckling of beams at elevated temperatures is very rare since most beams are braced by a floor system. Their research discussed the findings from the fire tests at the BRE Cardington test facility involving an eight story steel framed building subjected to real full scale fire. The conclusions were as follows; the tests demonstrated that under certain situations, unprotected steel beams designed to be composite with a composite floor slab will perform much better in fire than what would be expected from individual isolated member behaviour in fire. If this is the case, then the implication is that in certain situations, no protection of structural steel beams may be necessary.

Lewinger. et al.(1999) conducted some preliminary investigations on a model three story structural steel building and reported that at elevated temperatures, the moment capacity need to be evaluated as the reserve moment capacity, that is the difference between design moment strength and ultimate moment strength. The results were analysed for the reverse moment strength with the temperature moment for various steel grades. The results indicated that after two hour fire duration the reserve moment strength of steel members with a yield stress of 250 MPa or 345 MPa is less than the temperature moment strength. This indicated that steel with higher yield stresses such as 485 MPa (high performance steel) or greater should be used if reverse strength capacity is required, thus enabling steel members to resist loadings caused by a fire event. Their findings indicate that high performance steel performs extremely well at both ambient and elevated temperatures. Therefore fire resistance level of steel beams of a concrete-steel composite deck construction exposed to fire can be improved by using higher strength steels in order to maintain the serviceability deflection requirements in fire situations.

Buchanan (2001) reported that for continuous beams which spread over several supports or is part of a moment resisting frame structure is more stable than a simply supported beam in a fire situation. This is due to moment redistribution during the fire therefore increasing the fire resistance level of the structure. Figure 3.1, shows that a simply supported beam will fail as soon as one plastic hinge forms in the beam. At this point the flexural capacity of the beam is same as the applied moments. For a continuous beam the failure will not occur until three hinges are formed, thus increasing the fire resistance level of the structure.

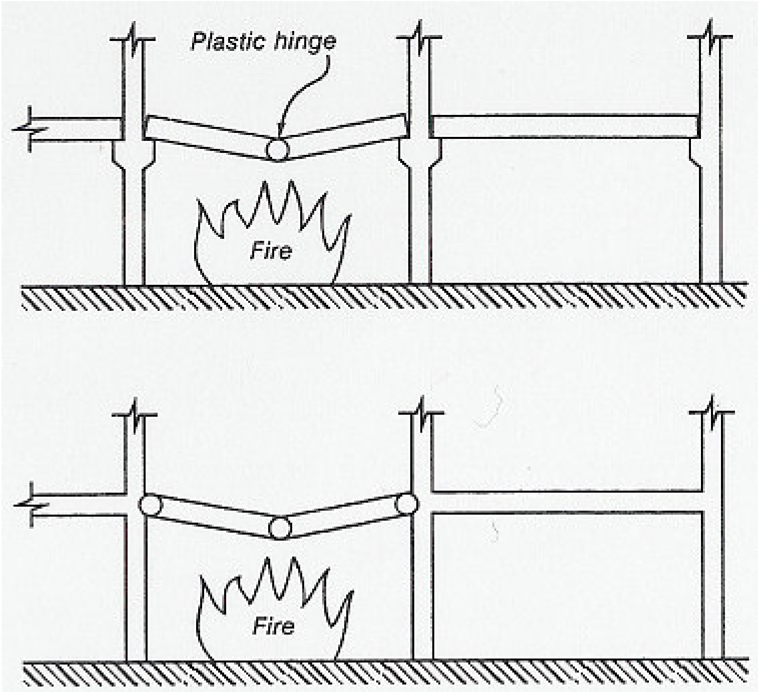

Figure 3.1 Failure mechanism for simply supported and continuous beam
Adapted from Buchanan (2001).

3.2 Column Analysis

Bailey (1999) reviewed the Cardington fire tests conducted by Building Research Establishment (BRE) and concluded that the internal and external columns are subjected to high moments caused by expansion of the connecting beams during a fire. He further stated that if these moments were simply included within the member during the design process, the calculations would show that the column would fail during the fire owing to local plasticity. The investigation revealed that column instability was significantly affected by; beam to column heating rates, beam cross section size, span of beams, end rigidity of the heated column and column axial load. They also revealed that column cross section size, beam to column connection rigidity and horizontal restraint to the heated beams (provided realistic values are chosen) had nominal effect on the behaviour of the column.

Bennetts and Thomas (2002) reported that the failure temperature of a steel column is independent of the stiffness of the axial restraint, given expected eccentricities of load and a minimum level of axial restraint. In simple words, the failure temperature of a practical column will be the same for all values of restraint above some value. This means that the reduction in failure temperature due to restraint varies with the slenderness of the column. For example, the failure temperature of a stocky column (slenderness ratio of 40) will be reduced by about 100ºC, whilst that of a slender member (slenderness ratio of 100) by about 250ºC. One of the key issues raised by their research is the effect of the expanding beams on the external columns. Bennetts and Thomas (2002) stated that measured strains taken during the fire tests indicate that significant bending moments are developed within the columns due to the expansion of the attached floors. Other factors to consider when analysing the severity of column failure is the end condition of the columns, the relative size of column to beam and the temperature of the column. If thermal gradient is present in the section this would reduce the failure temperature whilst continuity at the ends of the columns (rotational restraint) is very beneficial, thus columns should be treated more cautiously than beams.

Plank (2000) reported that due to buckling of columns, the analysis of columns in fire is potentially more difficult than for beams. Very stocky columns have significantly higher failure temperature than columns with intermediate slenderness ratio which results in non-uniform heating causing thermal bowing and hence additional bending due to load-deflection effect. This however is compensated by the strength retained in the cooler parts of the column and also if columns are non-uniformly heated they survive to higher temperatures then those which are fully exposed to fire. Behaviour of columns is also influenced by initial imperfections and eccentricity of loads, magnitude and direction of different bending influences and restraint to free thermal expansion.

Buchanan (2001) advised that design of columns subjected to temperature gradients is done best with the aid computer programs because thermal bowing and instability govern their behaviour. This is largely due to lateral buckling which has to be considered in the design process and predicting their behaviour is unreliable.

4 Fire Section of the Steel Code - AS4100:1998

4.1 Determination of Period of Structural Adequacy

The period of structural adequacy (PSA) of a member is the time in minutes for a member to reach the limit state of structural adequacy when exposed to the standard fire test, thus it is the amount of time a structural member has in order to support the applied loads when subjected to a standard fire test until failure.

AS 4100-1998: Section 12 provides the requirements of fire resistance level (FRL) of steel building elements and they are as follows:

The Period of Structural Adequacy (PSA) shall be determined using one of the following methods:

(a) by calculation:

(i) by determining the limiting temperature of the steel (T_l) in accordance with Clause 12.5; and then

(ii) by determining the PSA as the time from the start of the test (t) to the time at which the limiting steel temperature is attained in accordance with Clause 12.6 for protected members and Clause 12.7 for unprotected members;

(b) by direct application of a single test in accordance with Clause 12.8; or

(c) by structural analysis in accordance with Section 4, using mechanical properties, which vary with temperature in accordance with Clause 12.4. Calculation of the temperature of the steel shall be by using a rational method of analysis confirmed by test data.

4.2 Variation of Mechanical Properties of Steel with Temperature

4.2.1 Variation of Yield Stress with Temperature

In accordance to AS4100:1998, the influence of temperature on the yield stress of steel is determined by using equation 4.1a-b.

$$\frac{f_y(T)}{f_y(20)} = 1.0 \qquad \text{when } 0°\text{C} < T \leq 215°\text{C} \qquad 4.1\text{a}$$

$$= \frac{905 - T}{690} \qquad \text{when } 215°\text{C} < T \leq 905°\text{C} \qquad 4.1\text{b}$$

where

$f_y(T)$ = yield stress of steel at T °C

$f_y(20)$ = yield stress of steel at 20°C

T = temperature of the steel in °C

The temperature at which the proportion of the yield stress at elevated temperature is considered to have dropped to zero differs from that of the modulus of elasticity. Figure 4.1 shows the variation of the proportion of yield stress with temperature as given by AS4100.

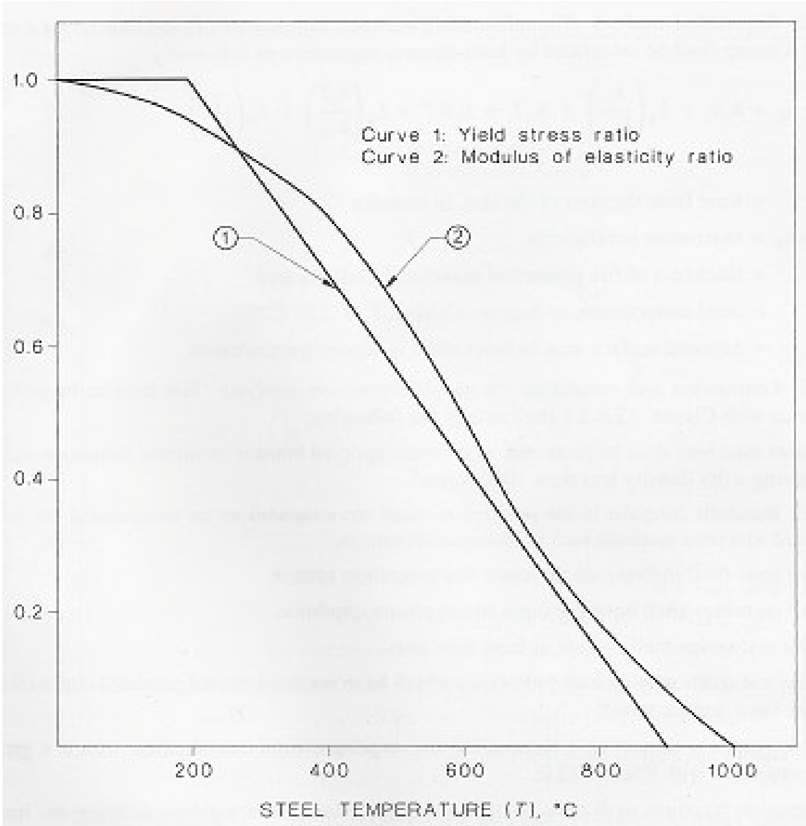

Figure 4.1 Variation of mechanical properties of steel with temperature
Adopted from AS4100:1998

4.2.2 Variation of Modulus of Elasticity with Temperature

The mechanical properties of steel vary with temperature, generally reducing in strength as the temperature of the steel increases. Steel has a limited strength, meaning that at a certain temperature the strength of the member will eventually reduce to zero as illustrated in Figure 4.1.

In accordance to AS4100, the influence of temperature on the modulus of elasticity of steel is determined using equation 4.2a-b. Figure 4.1 below shows the variation of the modulus of elasticity with temperatures as given by AS4100.

$$\frac{E(T)}{E(20)} = 1.0 + \left[\frac{T}{2000\left[\ln\left(\frac{T}{1100}\right)\right]}\right] \qquad \text{when } 0°\text{C} < T \leq 600°\text{C} \qquad 4.2a$$

$$= \frac{690\left(1-\frac{T}{1000}\right)}{T-53.5} \qquad \text{when } 600°\text{C} < T \leq 1000°\text{C} \qquad 4.2b$$

where

E(T) = modulus of elasticity of steel at T °C

$fy(20)$ = modulus of elasticity of steel at 20°C

T = temperature of the steel in °C

4.3 Determination of Limiting Steel Temperature

The temperature at which the member being analysed will fail is determined using the formula for the variation of the yield stress of steel for temperatures above 215 °C. This implies that the only factor affecting the steel strength is the yield stress with temperature.

$$T_l = 905 - 690r_f \qquad 4.3$$

where r_f is the ratio of the design action on the member under the design load for fire specified in AS1170.1, to the design capacity of the member at room temperature. Equation 4.3 can be used for three or four-sided exposure to fire, and for steel beams and columns.

The design capacity of the steel section is based on the yield stress and the cross sectional area of the beam. Assuming the cross section of the beam remains constant and a uniform temperature is maintained throughout the steel, then this formula is valid. This only occurs with four-sided exposure, as with three-sided exposure to a fire, there will be significant temperature differences across the cross section of the steel. When attempting to use this formula for three-sided exposure a finite element approach is used to obtain a limiting temperature that accounts for the temperature gradient in the steel.

4.4 Determination of Time at which Limiting Temperature is Attained for Protected Members

The determination of time at which the limiting temperature (T_l) is attained is based on results of tests on members with the appropriate protection applied. According to AS4100:1998, to assess the behaviour of a protected member, temperature data can be obtained either from regression analysis for a series of tests or from a single test.

For all members with four-sided exposure condition, the limiting temperature (T_l) is taken as the average temperature of the results at thermocouple locations during the test. The locations of the thermocouples are detailed in AS1530.4. For columns with three sided exposure condition, the limiting temperature (T_l) is taken as the average temperature of the thermocouples located on the face furthest from the wall, or alternatively the temperatures from members with four-sided exposure condition and same surface area to mass ratio can be used.

The two methods of determining the time at which the limiting temperature (T_l) is attained based on results of tests on members with the appropriate protection applied is detailed below.

4.4.1 Temperature Based on Test Series

The calculation of the variation of steel temperature with time is measured by interpolating results of a series of fire tests using regression analysis. The following equation which is based on least square regression can be used provided the following limitations and conditions are met.

$$t = k_0 + k_1 h_i + k_2 \left(\frac{h_i}{k_{sm}}\right) + k_3 T + k_4 h_i T + k_5 \left(\frac{h_i T}{k_{sm}}\right) + k_6 \left(\frac{T}{k_{sm}}\right) \qquad 4.4$$

where

t = time from start of the test, in minutes

k_0 to k_6 = regression coefficients determined from tests

h_i = thickness of fire protection material (mm)

T = steel temperature (°C), T>250 °C

k_{sm} = exposed surface area to mass ratio, in square metres/tonne

Limitations and conditions on the use of the regression analysis have to satisfy the following:

a) Steel members have to be protected with board, sprayed blanket or similar insulation materials with a dry density less than 1000 kg/m^3;

b) All tests to incorporate the same fire protection system;

c) All members to have the same fire exposure condition;

d) At least nine tests to be conducted for the test series;

e) The test series can include prototypes which have not been loaded provided stickability is achieved; and

f) Members subjected to three sided exposure condition have to comply with section 4.7.

4.4.2 Temperature Based on Single Test

The variation of steel temperature with time measured in a standard fire test may be used without modification provided:

a) The fire protection system is the same as the prototype;

b) The fire exposure condition is the same as the prototype;

c) The fire protection material thickness is equal to or greater than that of the prototype;

d) The surface area to mass ratio is equal to or less than that of the prototype; and

e) Where the prototype has been submitted to a standard fire test in an unloaded condition, stickability has been separately demonstrated.

4.5 Temperature Rise of Unprotected Steel

In accordance to AS4100:1998, the time (t) at which the limiting temperature is attained shall be calculated for:

(a) three-sided exposure condition as follows:

$$t = -5.2 + 0.0221T + \left(\frac{0.433T}{ksm} \right) \qquad 4.4$$

(b) four-sided fire exposure condition as follows:

$$t = -4.7 + 0.0263T + \left(\frac{0.213T}{k_{sm}}\right) \qquad 4.5$$

where

t = time from the start of the test, in minutes

T = steel temperature, in degrees Celsius, 500°C $\leq T \leq$ 750°C

k_{sm} = exposed surface area to mass ratio, 2 m^2/tonne $\leq k_{sm} \leq$ 35 m^2/tonne

To obtain times for temperatures below 500 °C, linear interpolation can be used with initial temperature of 20 °C at t equals 0.

4.6 Determination of PSA from a Single Test

The period of structural adequacy (PSA) of an element is determined from the results of a single standard fire test as detailed in AS1530.4 provided:

a) The fire protection system is the same as the prototype;
b) The fire exposure condition is the same as the prototype;
c) The fire protection material thickness is equal to or greater than that of the prototype;
d) The surface area to mass ratio is equal to or less than that of the prototype;
e) The conditions of support are the same as the prototype and the restraints are not less favourable than those of the prototype; and
f) The ratio of the design load for fire to the design capacity of the member is less than or equal to that of the prototype.

The above conditions indicate that the results of a single fire tests can only be used when the prototype gives comparable or more severe results than those of the member being analysed, particularly in relation to the effective length of a member, the loading and support conditions, the exposed surface area to mass ratio and the thickness of the insulation.

4.7 Three Sided Fire Exposure Condition

Where members are subjected to three sided fire exposure condition, they should be considered in separate groups unless the following conditions are satisfied:

(a) The variation of the characteristics of the members of a group from one another can not be more than:

(i) concrete density: $\left(\dfrac{\text{highest in group}}{\text{lowest in group}}\right) \leq 1.25$; and 4.6

(ii) effective thickness (h_e): $\left(\dfrac{\text{largest in group}}{\text{smallest in group}}\right) \leq 1.25$; and 4.7

where

h_e = cross sectional area excluding voids per unit width as illustrated in AS4100.

(b) Rib voids to be either all open or all blocked as illustrated in section 12 AS4100.

This clause caters for circumstances where there is three sided exposure with concrete densities which differ by more than 25 %, or for members with an effective thickness varying by more than 25 %. This makes an allowance for the consequential effect on the steel temperature that these variations create.

4.8 Special considerations

4.8.1 Connections

AS4100 recommends that the connections for protected members must have fire protection material applied with the same thickness as the maximum thickness required for any of the members framing into the connection. This thickness should be maintained over the entire connection components including bolt heads, welds and splice plates. This is a conservative approach to connection fire resistance level.

4.8.2 Web Penetrations

According to section 12 of AS4100 the thickness of fire protection material at and adjacent to web penetrations shall be the greater than that required for:

(a) The area above the penetration considered as a three-sided fire exposure condition k_{sm1};

(b) The area below the penetration considered as a four-sided fire exposure condition k_{sm2};

(c) The section as a whole considered as a three-sided fire exposure condition k_{sm3}.

The k_{smi} locations are identified in section 12 of AS4100. The thickness shall be applied over the full beam depth and shall extend each side of the penetration for a distance at least equal to the beam depth, and not less than 300 mm.

5 Design of Steel Members Exposed to Fire

Design of structural steel members exposed to fire is similar to design of structural members at normal room temperature. Design methods employed in AS4100 is based on limit state design of steel structures. In limit state design of structural members we determine the capacity of a member based on strength limit state and serviceability limit state. The strength limit state design process involves determining the ultimate strength of a member, thus avoiding collapse or failure once a member is subjected to the design loads. The serviceability limit state design criteria is concerned with control of deflections and vibrations which may affect the structure once in service. In a situation of fire, structural design of members and structures in general place emphasis on strength limit state because it is the strength and not deflection or vibration which is important to prevent or mitigate collapse of buildings.

When a steel structure is exposed to fire, the elevated temperatures reduce the strength capacity of members. Therefore, when designing a structure it is necessary to design the members in such a manner that the applied loads are less than the load capacity of the structure. The following inequality must then be satisfied:

Design action effect $\leq$ Design resistance

This is usually expressed as:

$$S^{*} \leq \phi R_u \qquad 5.1$$

where the design action effect, S^*, can be axial force, shear force or bending moment which may act singly or combined. The strength reduction factor, ϕ , is taken as 1.0 since design of structures under fire is primarily concerned with most likely expected strength.

5.1 Beams

For simply supported beams the design equation for flexure is:

$$M_f^* \leq M_f \qquad 5.2$$

$$M_f = k_{yT}\, Z_e\, f_y \qquad 5.3$$

where

M_f^* = design bending moment under fire conditions

M_f = member flexural capacity under fire conditions

k_{yT} = yield strength reduction factor from Equation 4.1a-b

The member capacity of a flexural member is governed by the nature of spread of heat across the section. If the member has a temperature gradient across its section, the limiting temperature is taken as the average of the temperatures measured at thermocouple locations as detailed in section 4.4. The limiting temperature for unprotected member is determined as detailed in section 4.5.

The design equation for calculation of shear forces during fire condition is:

$$V_f^* \leq V_f \qquad 5.4$$

$$V_f = k_{yT}\, V \qquad 5.5$$

where

V_f^* = design shear forces under fire conditions

V = member shear resistance under normal conditions

k_{yT} = yield strength reduction factor from Equation 4.1a-b

5.1.1 Simply Supported Beam

5.1.1.1 Beam Subjected to Three-Sided Exposure to Fire: Unprotected steel (I)

(a) For a simply supported beam, the time of failure when exposed to a standard fire is calculated to be:

Data available:

Dead load	G = 6.0 kN/m (self weight included)
Live load	Q = 9.5 kN/m
Beam span	L = 10.0 m (assume full lateral restraint)

Beam size	410UB60 (406 mm deep steel beam, 59.7 kg/m)
Effective modulus	$Z_e = 1060 \times 10^3 \, \text{mm}^3$
Exposed surface area to mass ratio	$k_{sm} = 22 \, \text{m}^2\text{/tonne}$

Member capacity at room temperature:

Design load	$w_c = 1.25\,G + 1.5Q = 21.7 \text{ kN/m}$
Yield strength	$f_y = 300 \text{ MPa}$
Strength reduction factor	$\phi = 0.9$
Design bending moment	$M^* = \dfrac{w_c L^2}{8} = 272 \text{ kNm}$
Member bending capacity	$M_n = Z_e \, f_y = 318 \text{ kNm}$
Design flexural strength	$\phi \, M_n = 286 \text{ kNm}$
Design check	$M^* < \phi \, M_n$ K OK

Member capacity when exposed to fire:

Design load	$w_f = G + 0.4\,Q = 9.8 \text{ kN/m}$
Design bending moment	$M_f^* = \dfrac{w_f L^2}{8} = 123 \text{ kNm}$
Load ratio	$r_f = \dfrac{M_f^*}{\phi \, M_n} = 0.43$
Limiting steel temperature	$T_l = 905 - 690\, r_f = 608^{\circ}\text{C}$
Time to reach limiting	$t = -5.2 + 0.0221\, T_l + \left(\dfrac{0.433 T_l}{k_{sm}} \right)$
Temperature	$= 20.2 \text{ minutes}$

The calculations indicate that the member will attain its limiting temperature of 608ºC when exposed to the standard fire for 20.2 minutes. This beam will fail in strength after the limiting temperature is reached.

5.1.1.2 Beam Subjected to Three-Sided Exposure to Fire: Unprotected steel (II)

(a) For a simply supported beam, the flexural strength after 20 minutes exposure to a standard fire is calculated to be:

Data available:

Dead load	$G = 5.0$ kN/m (self weight included)
Live load	$Q = 8.0$ kN/m
Beam span	$L = 7.0$ m (assume full lateral restraint)
Beam size	460UB67 (454 mm deep steel beam, 67.1 kg/m)
Effective modulus	$Z_e = 1300 \times 10^3 \text{ mm}^3$
Exposed surface area to mass ratio	$k_{sm} = 21.5 \text{ m}^2/\text{tonne}$

Member capacity at room temperature:

Design load	$w_c = 1.25\,G + 1.5Q = 18.3 \text{ kN/m}$
Yield strength	$f_y = 300 \text{ MPa}$
Strength reduction factor	$\phi = 0.9$
Design bending moment	$M^* = \dfrac{w_c L^2}{8} = 112 \text{ kNm}$
Member bending capacity	$M_n = Z_e\, f_y = 390 \text{ kNm}$
Design flexural strength	$\phi\, M_n = 351 \text{ kNm}$
Design check	$M^* < \phi\, M_n$ K *OK*

Member capacity when exposed to fire:

Design load	$w_f = G + 0.4\,Q = 8.2 \text{ kN/m}$
Design bending moment	$M_f^* = \dfrac{w_f L^2}{8} = 50 \text{ kN/m}$
Temperature after 20 minutes	$T = 555^{\circ}\text{C}$ (from Equation 4.4)
Yield strength reduction	$\dfrac{f_y(T)}{f_y(20)} = \dfrac{905 - T}{690} = 0.51$
Flexural capacity	$M_f = 0.51\, Z_e\, f_y = 199 \text{ kN/m}$
Design check	$M_f^* < M_f$ K *OK*

The above calculations indicate that the member will have sufficient flexural strength at 20 minutes. Failure of the member will occur as member capacity falls below design loading. If the section size is reduced, the flexural capacity will also reduce, thus failure of beam will occur sooner than the member currently used. This beam will fail in strength once design bending moment (M^*_f) exceeds the member capacity (M_f).

5.2 Columns

The design equation for calculation of axial load subjected on a column during fire condition is:

$$Ncf^* \leq Ncf \qquad 5.6$$

$$Ncf = k_{yT}\ An\ f_y \qquad 5.7$$

where

Ncf^* = design axial load under fire conditions

Ncf = member axial capacity under fire conditions as determined from AS4100 and AS1170.1

k_{yT} = yield strength reduction factor from Equation 4.1a-b

An = net area of section

5.3 Tension members

The design equation for calculation of axial load subjected on a column during fire condition is:

$$Ntf^* \leq Ntf \qquad 5.8$$

$$Ntf = k_{yT}\ An\ f_y \qquad 5.9$$

where

Nf^* = design tension force under fire conditions

Nf = member tension capacity under fire conditions

k_{yT} = yield strength reduction factor from Equation 4.1a-b

An = net area of section

It is noted that tension members usually are uniformly stressed and buckling forces are non existent for single tension members.

6 Conclusions and Recommendations

6.1 Conclusions

Reviews of relevant literature and past research coupled with application of code of practice (AS4100), indicates that structural behaviour of steel structures when exposed to fire depends on the material properties and behaviour of structural elements. The variables that affect properties of steel at elevated temperatures are the thermal and mechanical properties. Although the thermal conductivity, specific heat, and density of steel vary with temperature, these factors do not have a significant effect on the strength of steel. Basically, deformation of structures results from deteriorating mechanical properties at elevated temperatures. These properties can be improved to enhance the fire resistance levels by protecting steel elements with the use of fire protection techniques during design phase.

The behaviour of structural elements is largely dependent on the size, shape and location of a member in relation to the intensity of fire. In the case of beams that support floor systems on the upper flange tend to perform better than a beam exposed to fire on all sides because the latter has a greater cross-sectional area which is subjected to heat transfer. In addition, if a beam is designed as continuous member over intermediate supports it will have a higher failure temperature due to momentary distribution within the element, whereas simply supported beam will have a lower failure temperature. However, columns are mainly subjected to moments induced from expanding beams. Stocky columns have higher failure temperature in comparison to slender columns.

6.2 Recommendations

This research project is based on the analysis and behaviour of simple steel elements only when exposed to fire conditions. Further research could be done to include complex structures such as multistorey frame structures which are widely used. Research on this topic will certainly aid engineers and provide tools for designing steel structures that would have a good level of fire resistance at reduced costs. In addition, research needs to be done on preparing a universal design code for engineers which should provide the design parameters to be taken into consideration when designing steel structures subjected to fire conditions.

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Appendix A: Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/4112 Research Project
PROJECT SPECIFICATION

FOR:	**SANJEEVAM GOUNDER**
TOPIC:	Design of steel structures under fire
SUPERVISOR:	Dr. Amar Khennane
SPONSORSHIP:	Faculty of Engineering & Surveying
PROJECT AIM:	Do a literature survey on: the behaviour of structural steel at high temperatures, the structural behaviour of steel beams and columns under fire, the provision of design codes (Australian) and design simple steel elements such as beams and columns for a fictitious standard fire
PROGRAMME:	Issue A, 20 October 2005

1: Review the background information on properties of steel as a structural material at high temperatures.

2. Research the behaviour of structural elements such as beams and columns under fire, emphasis on large displacements and instability.

3. Review the code of practice (AS4100) as to what they suggest.

4. Design of simple structural steel elements in case of a fire.

TIMELINES:

● March - April 2004	Review background information.
● May 2004	Research the behaviour of structural steel elements.
● June 2004	Review the codes of practice.
● July - August 2004	Design of simple structural steel elements and loading.
● September 2005	Conclusions and preparation of draft dissertation.
● October 2005	Preparation of final dissertation.

AGREED:

_______________ (Student)   _______________ (Supervisor)

___/___/___   ___/___/___