TALL CONCRETE BUILDINGS SUBJECTED TO VERTICALLY MOVING FIRES: A CASE STUDY APPROACH

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Declaration

This thesis has been completed by Ian A Fletcher under the supervision of Dr Stephen Welch and Professor José L Torero and has not been submitted for any other degree or professional qualification. I declare that the work presented in this thesis is entirely my own except where indicated by full references.

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Abstract

Fire in buildings can have a severe impact in terms of both human safety and potential economic loss. This is especially true in the case of fires of such severity that the building structure is damaged.

Concrete buildings are traditionally regarded as safe in a fire situation as concrete is non-flammable and exhibits highly insulating material properties. The majority of current research relating to the impact of fire on structures examines other forms of construction. Research of concrete in fire is generally limited to investigation and testing of individual members in order to understand the often complex interactions exhibited by concrete as a material at high temperatures.

This research seeks to redress the balance by using a systematic approach to examine effects of fire on a holistic concrete structure in simplified but realistic temperature exposures. The research utilises evidence and structural information from the Windsor Tower in Madrid, which suffered a major fire in February 2005 with partial collapse in some areas of the structure. The fire spread throughout the building, travelling both upwards and downwards.

Computer modelling was used extensively. Computational Fluid Dynamics (CFD) analysis was used to explore likely fire temperature and duration in localised areas. Structural Finite Element Modelling (FEM) was used to develop a hierarchy of models, beginning with simple structural forms and progressing logically to more detailed structures. This produced a systematic and comprehensive analysis of the reaction of the structure to fire for comparison to the real, observable damage to the building and assessment of generic failure behaviours.

The structural model produced was used with a number of variations in support condition, fire spread rate and extent, and fire protection. It was found that for a structure of this type, structural stiffness of the concrete floors was insufficient to compensate for the loss of strength in heated steel members where there was no alternative load redistribution path.

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It was also found that in the case where an alternative load path exists, but involves steel members which have previously heated during the multiple-floor spread of the fire, the rate of fire spread has a critical effect on the structural stability.

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Contents

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List of Tables

List of Figures

List of Appendices

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CHAPTER 1: THESIS OUTLINE AND RESEARCH CONTRIBUTION

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1.1 The Problem: Concrete Structures in Fire

Fires in buildings have been a cause for concern since time immemorial. Some of the first fire tests were carried from the 1790s onwards (Babrauskas 1980a; Babrauskas 1980b), with the majority of testing concentrating mainly on the effects of fire on an individual member. It is only relatively recently that the structural interaction of buildings and fire has become a subject of academic study. Previously, the majority of work in this field focused on improving the fire resistance of elements by insulation and fire proofing.

This work is undertaken to gain a better understanding of the effect of a fire on the entire structure, specifically using reinforced concrete as the main structural material, and to examine the effects and limitations of fire protection of individual members on a building's overall structural stability.

The nature of concrete-based structures means that they generally perform very well in fire and are considered to be 'safe', as concrete is non-combustible and can act as a barrier preventing heat and fire spread. However, concrete is a complex non-homogeneous material and its properties can change dramatically when exposed to high temperatures. As a result modelling concrete structures can be extremely complicated. The principal effects of fire on concrete are loss of compressive strength and spalling, the forcible ejection of material from the member.

There is a need for a wider understanding of the response of concrete members to different heating regimes and the performance of whole-frame structures subjected to realistic fire exposures. Very few full-scale tests have been undertaken, and therefore some useful insights can be gained from the observations and assessment of concrete structures' performance in real fires. This thesis addresses this issue by means of a detailed analysis of the Windsor Tower fire.

1.2 Thesis Proposal, Aims and Objectives

This research proposes that real fire case studies can be used to advance the understanding of the mechanics and stability of concrete structure during a fire.

As it is impractical to gather precise data on a real fire and the structure which it affects, assumptions of characteristic variables must be made. The ‘Worst credible case’ method is developed and utilised to systematically select these variables.

The Windsor Tower in Madrid, which was subjected to a severe fire and underwent partial collapse in February 2005, will be used as the main case study for this research. Other case studies are discussed in Section 2.8.

Rather than conducting a forensic analysis, the variables selected via this method are used to perform an analysis of the fire and structural response of a ‘Windsor Tower-like’ structure. This will lead to a greater understanding of the holistic effects of fire on a generic structure, and the steps which must be taken to prevent collapse.

The aim of this research is an increasing understanding of structural stability during a fire. The application of this research could lead to improved fire safety, and ultimately saving lives. To this end, six research objectives can be defined.

1. Investigate and fully understand the extent, nature and impacts of the problem
2. Undertake a comprehensive literature review to further objective 1 and to establish potential viable research routes.
3. Investigate Windsor Tower structural data to establish the required model extent and member sizes.
4. Investigate possible modelling methods and select the most appropriate.
5. Develop appropriate fire and structural models that can be applied to a ‘Windsor Tower-like structure’. Test and compare the developed models with observations.
6. Consider further improvements and future research opportunities.

This thesis is structured into seven chapters. Tables 1.1 and 1.2 highlight the key elements within each chapter and demonstrate how these relate to the research objectives above.

Chapter	Objectives 1	2	3	4	5	6
Chapter 1: Thesis Outline and Research Contribution This chapter highlights: • Research problem, aims and objectives • Contribution to knowledge • Publication list	✓					
Chapter 2: Background This chapter highlights: • Forms of Construction • Fire Characterisation • Concrete in Fire • Modelling Concrete • Performance of Reinforcement in fire • Fire Tests • Examples of real fires in structures	✓	✓				
Chapter 3: The Windsor Tower Case Study This chapter highlights: • Structural form of the Windsor Tower • The Windsor Tower Fire • Post – Fire Structural Damage • Possible Collapse Mechanisms			✓	✓		

Table 1.1 Measuring chapters 1-3 against research objectives

Chapter	Objectives					
	1	2	3	4	5	6
Chapter 4: Research Method This chapter highlights: • Establishing a modelling framework • Modelling the fire • Modelling heat transfer • Modelling the structure			✓	✓		
Chapter 5: Testing Model Variables This chapter highlights: • Fire Modelling • Structural Modelling • Sensitivity Analysis					✓	
Chapter 6: The Full Sized Model This chapter highlights: • Full scale Modelling and Analysis • Developing a full scale structural model • Material values • Relating full scale analysis to sensitivity study • Heat Transfer • Multifloor Variables					✓	
Chapter 7: Conclusions and Further Research						✓

Table 1.2 Measuring chapters 4 -7 against research objectives

1.3 Thesis Contribution & Publications

The research presented in this thesis has several contributions to the fields of structural fire engineering. These contributions are briefly summarised as:

- An intuitive research method, ‘the worst credible case’ – a systematic approach to establishing unknown variables.
- Simplification of Computational Fluid Dynamics model output to a more generic but still applicable fire definition.
- Examination and characterisation of collapse mechanisms for a ‘Windsor Tower-like’ structure.
- Established the importance of major fire resistant structural elements to allow load redistribution.
- Demonstration of load redistribution limitations and the requirement for some degree of fire protection to structural steel elements.
- Structural response to evolving fires.

Table 1.3 gives details of the 8 publications by Ian A Fletcher. Pre–publication versions of the 5 conference papers ,2 journal papers and 1 book chapter are given in full as ***Appendix B***.

	Title	Conference /Journal
Fletcher, I. A. Welch, S. Torero, J.L. Carvel, R.O. Usmani, A.	Behaviour of Concrete structures in fire	Journal of Thermal Science Volume 11 (2) : 37-52 2007
Capote, J. A. Alvear, D. Lázaro, M. Crespo, J. **Fletcher, I. A.** Welch, S. Torero, J. L.	Modelado de las solicitaciones de los elementos estructurales de hormigón en Edificios de Gran Altura en Incendios Reales (Modelling the stresses of structural concrete elements in high rise buildings during actual fire) (In Spanish)	Materiales de Construcción Journal Paper Under Review
Fletcher, I. A. Welch, S. Borg, A. Hitchen, N.	Performance of Concrete in Fire: A Review of the State of the Art, with a case study of the Windsor Tower fire	SiF '06 - 4th International workshop on Structures in Fire Universidade de Aveiro, Portugal. 779-790
Capote, J. A. Alvear, D. Lázaro, M. Espina, P. **Fletcher, I. A.** Welch, S. Torero, J. L.	Analysis of Thermal Fields Generated by Natural Fires on the Structural Elements of Tall Buildings	International Congress Fire Safety in Tall Buildings, Santander, Spain. 93-109
Fletcher, I. A. Welch, S. Capote, J.A. Alvear, D. Lázaro, M.	Model-based analysis of a concrete building subjected to fire	Advance Research Workshop in Fire Computer Modelling, University of Cantabria (Spain). 213 – 226
Fletcher, I. A. Welch, S. Capote, J.A. Alvear, D. Lázaro, M.	Effects of fire on a concrete structure: Modelling the Windsor Tower	Fire Design of Concrete Structures - From Materials Modelling to Structural Performance, FiB 2007, University of Coimbra, Portugal. 571-582
Fletcher, I. A. Welch, S. Capote, J. A. Alvear, D. Lázaro, M	Effects of Fire on a Concrete Structure: Modelling the Windsor Tower.	SiF '08 - Fifth International Conference on Structures in Fire, Nanyang Technological University, Singapore. 344-354
Reszka, P. Abecassis Empis, C. Biteau, H. Cowlard, A. Steinhaus, T. **Fletcher, I.A.** Fuentes, A. Gillie, M. Welch, S.	Experimental Layout and Description of the Building	The Dalmarnock Fire Tests: Experiments and Modelling Published by: The School of Engineering and Electronics, University of Edinburgh 31-62

Table 1.3 Table of Publications

CHAPTER 2: BACKGROUND

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2.1 Introduction

In order to study the effects of fire on a concrete structure, it is necessary to examine the previous research and modelling which has been performed.

This chapter examines the behaviour of concrete as a material, fire testing of concrete structural elements and concrete structures, and fires in real concrete buildings.

This will be examined in the following sections:

- **Forms of Construction**
- **Fire Characterisation**
- **Concrete in Fire**
- **Modelling Concrete**
- **Performance of Reinforcement in Fire**
- **Fire Tests**
- **Examples of Real Fires in Structures**
- **Conclusions**

2.2 Forms of Construction

There are many forms of construction, using materials as diverse as masonry, timber, concrete and steel in a huge number of possible combinations and permutations. The method of construction will vary depending on the application, and obviously a tunnel or bridge will be built in a totally different manner from a high rise building. Some of the most common methods of construction for large structures are:

- Steel framed buildings
- Composite construction
- Concrete framed buildings

'Steel framed buildings' are formed from a steel skeleton with floor slabs, most often in concrete, sitting on top of the structural members. The floor slab has no structural effect on the building. If concrete is used for the floor slab, it can be either a precast system or cast in-situ.

In 'composite construction', a concrete slab is most commonly cast upon steel beams. The formwork for this slab is a profiled metal sheet, known as decking, which spans between the beams. 'Shear studs' are welded to the top of the steel beams, through this profiled decking. These studs allow a mechanical bond to be formed between the concrete and the steel member, and therefore allow the beam and the slab to act as a single member with an increased strength. The steel decking is left permanently in place after the concrete has been cast. Steel reinforcement is typically added above the profiled decking.

'Concrete framed construction' uses concrete columns and beams, either precast or cast in-situ, generally with concrete floor slabs between the beams. The floor slab can be formed in many ways, and one well known form of construction is the "waffle slab" where the formwork creates hollows within the slab to reduce its overall weight (Figure 2.1).

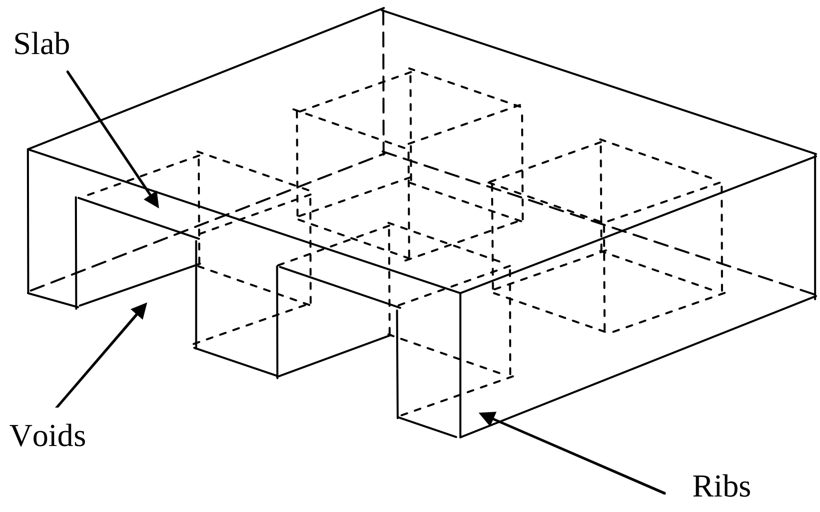

Figure 2.1 - Example Waffle Slab

2.3 Fire Characterisation

In order to understand the effect of fire on a structure, it must first be estimated what thermal exposure is actually being applied to the members.

When undertaking laboratory fire testing of a structural member, a furnace is used to bring the gases surrounding the structural components up to a given temperature. This heating is generally applied according to one of several ‘fire curves’.

Fire curves themselves are relatively simple, giving a rate of temperature increase which is initially fast but slows to a gradual increase. There is no cooling phase to the curves.

2.3.1 ISO 834 Standard Fire Curve

Most commonly used in design is the ISO 834 Standard Fire Curve (often abbreviated to the ‘ISO curve’ or the ‘Standard Fire’ curve), and its equivalents (BS

476, ASTM E-119, EN 1365) although other fire curves such as the 'Hydrocarbon Fire' are also used.

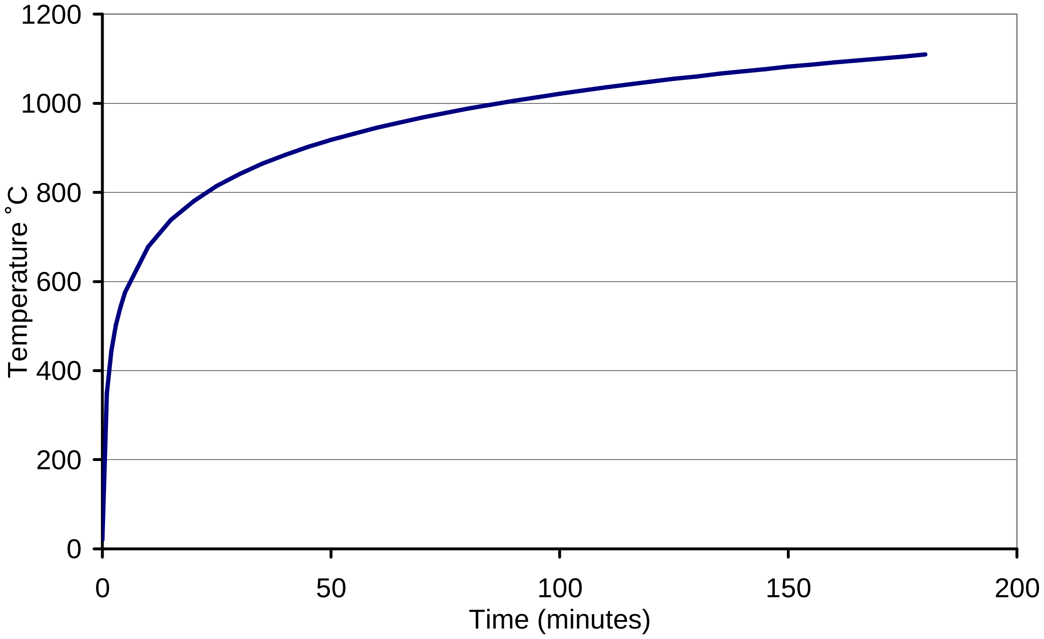

Figure 2.2 ISO Fire curve

The essence of the Standard Fire curve (Figure 2.2) was established in 1917, after acknowledging that specifying a peak temperature for a furnace would not accurately represent fire conditions as there would always be a 'warming up' period prior to this temperature being reached. A variety of early temperature-time curves were examined, leading to the idealised Standard. (Babrauskas 1980a; Babrauskas 1980b)

While these predetermined heating regimes such as the ISO curve have many disadvantages and may not accurately follow the temperature profile of a real fire, they are clearly understood and relatively easily applied as a temperature profile in a computer simulation.

Fire curves dictate the gas temperature within the furnace, rather than the temperature of any given area of the structural elements. The actual thermal exposure of an individual element will vary greatly depending on both the properties of the furnace and fuel (Drysdale 1998) and the element under testing. These variables can greatly influence the heat flux impacting upon the member. As an example, a steel element and a concrete element subjected to the same furnace test will expose the steel element to a significantly higher net heat flux than it does the concrete element (Welch 1997). The thermal conductivity of the steel is many times higher than that of the concrete, resulting in greater heat transfer from the furnace to the steel member.

2.3.2 Eurocode parametric curves

A more recent development is the use of Eurocode parametric curves. These curves take into account the fuel loading within a compartment and its ventilation conditions to give a temperature profile more accurately based upon the building conditions. These curves have the disadvantage that if the compartment dimensions, fuel and ventilation are unknown, many assumptions will still have to be made. (Eurocode 1-1-2 2002).

Past research has sought to differentiate structural response to ‘short hot’ fires compared to ‘long cool’ ones. During a ‘short hot’ fire there is considerable heat release during the initial period of the fire followed by rapid cooling. Conversely a ‘long cool’ fire, does not reach these high temperatures but structural members may be subject to a greater depth of heating. Debate on which of these conditions is more harmful is ongoing (Lamont, Usmani et al. 2004), and it is sensible to consider on a case-by-case basis which type of heating regime will be more onerous.

2.3.3 Computational Modelling

A further option is the use of Computational Fluid Dynamics methods to predict the temperatures and heat fluxes within a building. This requires detailed knowledge of the building itself and the combustible materials within it, or reasonable assumptions about these details. It should be noted (Rein 2007a) that even in a situation where the fire loading, geometry and ventilation characteristics can be well defined, the results from CFD modelling can vary over a wide range depending upon the initial assumptions made. The real behaviour of a fire can also vary widely depending on initial variations. For this reason, extreme care should be used when interpreting any results from a CFD model.

It should be remembered that the occupancy or use of a building is very rarely fixed, and while calculations can be performed during the building's construction the fire loads are frequently subject to change over time.

While acknowledging these limitations, accurate computer modelling of real or full scale fire tests is very important to the future understanding of structures in fire. It is impossible to build an exact replica of every building and test every fire variation within it. Computer modelling of the building with a variety of possible fires can more easily be performed in order to build an "envelope" of fire conditions. In order for these models to be regarded as reasonable, they must be validated to the best degree possible against the observed results of a real fire.

2.4 Concrete in Fire

Concrete is a complex, non homogenous material composed of a cement gel matrix and aggregate. In reinforced concrete steel reinforcement is also present. Each of these components reacts differently when exposed to high temperatures. (Bazant 1996)

There are many different types of aggregate used in the production of concrete, and the ratios in which concrete components are varied depends on the structural properties desired. Thus definitive information about the performance of "concrete" in fire is difficult as there are many different materials, all referred to as concrete, which can react very differently.

The thermal conductivity of concrete is relatively low, resulting in large temperature gradients within a concrete member when heated. This is generally interpreted as inherent fire proofing, especially in reinforced concrete members where the majority of the structural strength is provided by the reinforcement. The core of the concrete will also be insulated and therefore retain its structural strength. Some degree of thermal analysis of a member is therefore necessary, and the critical time period for failure of a concrete member may be after the extinction of the fire, when the thermal wave has propagated through a significant proportion of the member.

The compressive strength of concrete is generally regarded as remaining reasonably constant until a critical temperature is achieved. A fast decrease generally follows with further increase in temperature. Typical values for the critical temperature are as follows, and arise from physical and chemical changes to the concrete material (Bazant 1996):

- Sand light-weight concrete: T_C ~ 650°C
- Calcareous: T_C ~ 660°C
- Siliceous: T_C ~ 430°C

2.4.1 Physical and Chemical Interactions

A number of physical and chemical changes occur in concrete subjected to heat (Bazant 1996 ; Beard 2005). Some of these are reversible upon cooling, such as loss of moisture content, but others are non-reversible and may significantly weaken the concrete after a fire.

Most concrete contains liquid water within its pores, which will begin to vaporise if the temperature reaches the boiling point of water. This may vary within a range from around 100°C to 140°C as high pressures increase the boiling point of liquids. This will cause a build-up of pressure (often referred to as Pore Water Pressure, PWP) within the concrete which forces liquid water from areas of high pressure to areas of low pressure. This is generally reversible on cooling, provided there is sufficient moisture in the atmosphere.

Calcium hydroxide in the cement will begin to dehydrate at temperatures exceeding 400°C, bringing about a significant reduction in the physical strength of the material and producing water vapour which may again lead to an increase in pore water pressure (Bazant 1996).

Other physical and chemical changes are generally related to the type of aggregate used in the concrete mixture. Quartz-based aggregates increase in volume, due to a mineral transformation at about 575°C. Limestone aggregates will begin to decompose at about 800°C. The thermal expansion response of the aggregate itself is usually predictable, but differential expansion between the aggregate and the cement matrix may cause cracking and spalling.

These physical and chemical changes in concrete will have the effect of reducing the compressive strength of the material. Due to the low conductivity of concrete the heat will not quickly penetrate very far into the member, meaning that the structure as a whole normally retains much of its strength. Steel suffers a significant reduction in strength at a similar critical temperature.

The severity and temperature of the fire can sometimes be gauged (Alarcon-Ruiz, Platret et al. 2005) because concrete does not fully returning to its original state following a fire. Colour changes, sometimes referred to as “pinking”, will provide some information on the maximum temperature which an area of concrete reached (Arioz 2007). Studies of the depth of cracking in a concrete building subjected to fire by Geogali & Tsakiridis (Georgali 2005) have found that this also relates to the temperature of the fire. The INTEMAC report into the fire in the Windsor Tower measured the ultrasonic pulse velocity of concrete to gauge the depth to which fire damage had penetrated, giving an indication of the durations and temperatures of the fire (Calavera 2005).

2.4.2 Spalling

One of the most commonly observed physical reactions of concrete when exposed to fire is spalling, the explosive ejection of pieces of concrete from the main body (Tenchev and Purnell 2005). This occurs when the tensile strength of the concrete becomes lower than the internal forces generated to expel the outermost layer of concrete, and can occur during both the heating and cooling phases.

There are several potential impacts to spalling:

- In a reinforced concrete structure, spalling may reduce the amount of concrete covering the reinforcement, allowing direct heating of the steel
- The spalling may reduce the cross-sectional area of the concrete member, resulting in equal loads being supported by a reduced area of concrete

- Spalling may expose deeper levels of concrete to the fire compartment temperatures, allowing the strength of the interior concrete to be reduced due to the effects of heat and potentially causing further spalling in a chain reaction.

Spalling is often linked to the pore water pressure within the concrete forcing the surface layer to be ejected from the member. It can also be caused by the stresses induced in the concrete itself by high levels of compression. It is likely that different mechanisms are involved in different circumstances (Jansson 2008). It is generally understood that the required moisture content for spalling conditions to occur is around 2%.

A high rate of temperature change may also be required. Note that while the rate of temperature change, and therefore the temperature gradients within the concrete, may be high the actual temperatures themselves need not be. Spalling has been seen at temperatures as low as 200°C (Both, van de Haar et al. 1999) and during sudden cooling, for example during fire extinguishing. This was demonstrated during a test of some concrete structural elements at Hagerbach test gallery, Switzerland (Wetzig 2001). During the test a concrete sample resisted temperatures of up to 1600°C for two hours without collapsing, but half an hour into the cooling phase the sample collapsed explosively.

In the majority of cases it is unknown how much spalling takes place as a result of cooling rather than heating. These factors makes it impossible to give a specific temperature when spalling will occur, though temperature gradients in the range of 7-8 K/mm depending on the material and strength of the concrete have been suggested.(Schneider and Lebeda 2007) Spalling is generally more commonly observed when using High Strength concrete, with strengths in excess of 60kN/m^2 and often as high as 100 kN/m^2. This concrete has higher compressive strength than normal strength concrete; however it is also considerably less porous and moisture absorbent. This makes it more difficult for water vapour to escape during heating, and increases the likelihood of high pressure developing within the structure. It is by no means guaranteed that a High Strength concrete will have a poorer performance in spalling, as its improved tensile properties can counteract the increase in internal forces.

The mitigation of spalling is a subject of great interest. One of the most promising methodologies is the addition of polypropylene fibres to the concrete mixture (Kalifa, Menneteau et al. 2000; Ali, O'Connor et al. 2001; Han, Hwang et al. 2005; Hertz and Sorensen 2005). While it has been demonstrated that the addition of fibres to a concrete mixture does help reduce spalling, the precise method by which this process works is debatable. It is often thought that the melting of the fibres allows pathways to form within the concrete, allowing the moisture to either escape from the surface of the member or travel deeper within it to areas of lower temperature. Some recent work (Khoury and Majorna 2007) has cast doubt upon this due to the melting point of the polypropylene fibres being higher than the boiling point of water. The suggested alternative is that the fibres soften when exposed to heating, allowing the water vapour to force itself past the fibres to areas of lower pressure.

Spalling is a largely localised phenomenon, and it would be unusual to find a concrete structure where an equal amount of spalling had occurred in all areas. While the effects of spalling on an individual member can often be very severe, it is reasonable to assume within the context of a whole building fire that many other members will not be subject to serious spalling and will therefore be able to withstand the extra load which will be transmitted to them from the spalled members.

2.4.3 Cracking

Thermal expansion and dehydration of concrete due to heating may lead to the formation of fissures in the concrete rather than, or in addition to, explosive spalling. These fissures may also provide pathways for direct heating of the reinforcement bringing about additional thermal stress and further cracking. In particularly severe circumstances the integrity failure due to these cracks may provide pathways for heat, combustion products and flames to spread through the barrier to the adjoining compartment.

2.5 Modelling Concrete

Many models are available to define the mechanical behaviour of concrete at elevated temperatures (Law and Gillie 2008). A number of these are reviewed by Li & Purkiss (Li and Purkiss 2005), including the model suggested by Schneider (Schneider 1986). It is noted that these models subdivide the strain imposed on the concrete into four different types. These are:

- Free thermal strain caused by the change in temperature
- Creep strain caused by the dislocation of microstructures within the material
- Transient strain caused by changes in chemical composition
- Stress-related strain caused by externally applied forces.

The models examined by Li & Purkiss handle these strains differently. In each case ‘free thermal strain’ is solely a function of the temperature of the concrete member, however creep, transient and stress-related strains are taken to be functions of stress, temperature and time. This makes it difficult to separate which strains are being influenced during an experiment. In order to reduce this level of complication, some of the models gather two or even all three of these strains together into one term. Typically, this is the ‘Transient Creep Strain’, incorporating the creep strain and transient strain together.

The Li & Purkiss model demonstrates the significance of transient strain and that models that do not include it may be unconservative for high temperatures, though at low temperatures transient strain appears to have less effect. Li and Purkiss also noted that “full stress-strain curves provided in the structural Eurocode for concrete design, EN 1992-1-2 (Eurocode 2 2003) for higher temperatures are ‘unconservative’ compared to the values from the models examined.”

2.6 Performance of Reinforcement in Fire

The performance of steel during a fire is understood to a higher degree than the performance of concrete, and the strength of steel at a given temperature can be predicted with reasonable reliability. It is generally acknowledged that steel reinforcement bars require protection from high temperature exposure as low Carbon content steel exhibits 'blue brittleness' between 200 and 300°C.

Concrete and steel exhibit similar thermal expansion at temperatures up to 400°C. Steel will expand more compared to the concrete at higher temperatures. If temperatures of the order of 700°C are attained, the load bearing capacity of the steel reinforcement will be reduced to about 20% of its design value.

Reinforcement can also have a significant effect on the transport of water within a heated concrete member, creating impermeable regions where water may become trapped. This forces the water to flow around the bars, increasing the pore pressure in some areas of the concrete and therefore potentially enhancing the risk of spalling. Conversely, these areas of trapped water can also alter the heat flow near the reinforcement, leading to reduced temperatures of the interior of the member (Chung and Consolazio 2005).

2.7 Fire Tests

Fire testing of concrete can be classified as either laboratory testing of small scale members or partial/full scale building tests.

2.7.1 Laboratory Scale testing

The majority of research tests carried out to examine the properties of concrete have been performed using individual members. Typically these tests are designed to examine one particular aspect of the behaviour of concrete members during a fire, for example spalling.

Various methods of load application can be used during tests e.g. sandbags placed on top of the member or hydraulic jacks. The fire exposure of the member undergoing testing varies depending on the type of member to be tested. In the case of a slab, typically only one side (generally the underside) of the slab will be exposed to elevated temperatures, with the other side of the slab allowed to remain at ambient temperature. This will be unrepresentative in the case where, for example, a fire breaks out both above and below a given floor (Drysdale 1998; Beitel 2002).

There are practical limits to testing members in a furnace due to the limited size of most furnaces:

- Short sections of members must often be used.
- It is very difficult to have more then one member being tested at once (Beitel 2002)

Therefore testing of structural members generally takes place with restraint conditions different from those which would accurately represent a real life situation.

It is virtually impossible to maintain the same temperature within all areas of the furnace. While the characteristic temperature may be said to follow the specified curve there will be areas of the furnace both hotter and colder than this temperature.

Thermal exposure of an individual element will vary greatly depending on both the properties of the furnace and the element under testing. Heat transfer to a specimen is often governed by radiation from the furnace walls. Variations in the wall emissivity and the emissivity of the sample itself will result in different heat fluxes acting on a member at various points. Therefore sections of the structural element being tested will potentially have temperature differentials across them (Welch 1997; Liang and Welch 2007). Results will also vary from furnace to furnace and between members under testing (Drysdale 1998).

The above points demonstrate the obvious limitations of testing upon individual members. Nevertheless, it must be remembered that individual member tests have a variety of advantages:

- Member tests are relatively simple, quick and cheap
- Numerous tests can be undertaken, allowing repeatability and variations to be documented, allowing conclusions about specific phenomena to be drawn.

2.7.2 Large Scale Testing

Given the limitations of testing individual members during a fire, the appeal of carrying out tests on larger structures is obvious. The principal advantage is that the member under consideration is contained within a network of other members which may either be at ambient temperature or at raised temperatures themselves.

However, large scale testing is extremely time-consuming and expensive to set up and obviously cannot be individually varied to take account of every possible form of building construction or fire exposure.

Past research has generally concentrated on the effects of fire on either purely steel framed structures, or structures of composite construction. Limited research has been conducted on concrete framed structures as it is widely assumed that these structures are ‘safe in fire’ due the high insulation properties and non-combustible nature of the material.

Renowned research by the BRE at Cardington has greatly advanced the understanding of the behaviour of steel framed buildings and the fire protection that is required for them. These large scale tests used a composite steel framed structure (Kirby 2000; Lamont, Usmani et al. 2004) largely under sponsorship by British Steel.

Analysis and modelling of the Cardington tests has identified ‘Tensile Membrane Action’(Bailey 2002) where a reinforced concrete floor slab, highly deformed due to fire, supports the steel framework of the building and prevents its collapse. The reinforcement within the concrete slab acts in tension to transmit the load from the unprotected steel to building members which are either not affected by the fire, or have been fire protected.

This demonstrated that in many cases, fire protection of every member in a steel structure may not be necessary as the localised failure of one member need not lead to the collapse of the structure. Instead the loads will be redistributed to undamaged members (Usmani and Rotter 2001).

Nonetheless, other studies have shown that a fire in a structure can have much more serious consequences than would be observed for an individual structural member in isolation. This is often due to thermal expansion of the members, which can lead to column misalignment and increased eccentricity of load causing buckling and potentially collapse.

A landmark test on a multi-storey concrete building was also performed at Cardington. This building was originally assembled to examine different methods of concrete construction. After completion of the original research, the opportunity was taken to conduct a fire test in the building (Bailey 2002). This full scale test involved large amounts of instrumentation in a compartment containing a column constructed from High Performance Concrete (HPC) with polypropylene fibres added to the mix. The ceiling of the compartment was a reinforced concrete slab, with no polypropylene fibres added.

During the fire test, problems occurred with the data recording instrumentation resulting in much of the hoped for data being lost during the end phase of the test. Nonetheless, a large amount of data was gathered, together with post-fire observation of the structure. It was clear that more spalling had taken place in the concrete floor /ceiling slab than would have been expected (Bailey 2002). The concrete slab did not collapse and a great deal of research has been undertaken to understand this. It is likely that the floor slabs were further supported by ‘compressive membrane action’ whereby a mechanical arching effect takes place due to expansion of the member, compressing it against its supports (Bailey 2002). Providing the downward deflection of the slab is not too great, and the concrete does not yield, this will allow the reinforced concrete structure to support loads even when the tension rebar has reached a temperature where it ceases to be effective. Thus compression can greatly increase the load capacity of a slab in what would otherwise be a failure situation. It should also be noted that an increase in compressive forces on a concrete member can often increase the risk of spalling, as mentioned in Section 2.4.2.

Another, more recent, test was performed by The University of Edinburgh in collaboration with the BBC “Horizon” show. Other collaborating organisations were

Strathclyde Fire and Rescue service, Arup, the Building Research Establishment, Vision Systems (now Xtralis) and Lion TV. A tower block in Dalmarnock, Glasgow had been scheduled for demolition allowing some limited fire testing to take place.

A room within the tower block was refurnished to have a fire load resembling that in a standard office (Rein 2007a). Several types of instrumentation were installed with the aim of:

- Characterising the fire
- Comparing results with predictive computer models of the fire spread.
- Comparing results with predictive computer models of structural response.

The instrumentation included thermocouples to allow the measurement of the fire temperature in a large number of positions, and velocity probes to allow measurement of gases in and out of the fire compartment via both the windows and doors.

The Dalmarnock tower block consisted of a mixture of precast and in-situ concrete. While it was not predicted that there would be a great deal of structural damage to the building, the concrete floor slab in the room above was instrumented with deflection meters, thermocouples and strain gauges.

The results of the Dalmarnock tests are not discussed in detail here, though it was immediately noticeable after the test that hairline cracks had appeared in the concrete floor slab above the fire compartment. These cracks were generally seen to occur at the ends of the curtailed reinforcement at the edge of the slab. While these cracks were not severe in this scenario it was noted that for thinner slabs with longer spans, such as is seen in many modern buildings, a breach in fire compartmentation could be possible (Deeny, Empis et al. 2007).

The importance of these tests is obvious, as without examining the behaviour of concrete structural elements within a whole structure a variety of effects cannot be understood clearly, such as the role of the expansion of concrete members, and therefore the increase in compressive force within them.

2.8 Examples of Real Fires in Structures

It is important to examine the effects of fire on a variety of structures. This need not be restricted to fires in buildings; civil engineering structures such as bridges can also be involved in fires, and consideration must be given to this during their design. Some of the most severe fires, particularly in terms of loss of life, take place within tunnels. The Mont Blanc tunnel fire resulted in the deaths of 38 people, and extensive structural damage (Beard 2005).

This section examines the effects of fires on four different structures of varying construction and type.

2.8.1 The World Trade Centre

A major event leading to the current interest in the effects of fire on structures was the September 11th attack on the World Trade Centre in 2001. While it is well known that the Twin Towers suffered a catastrophic level of damage, there were two other buildings damaged during the attack, buildings 5 and 7.

Building 7 suffered no direct damage from the terrorist attack, but suffered extensive damage caused by the fires which spread through it, causing collapse. This is the first time that a steel framed structure is known to have collapsed purely due to the action of fire (Gilsanz 2007).

There is some debate on how much effect the impact of the airplanes had on the Twin Towers. It has been argued that the impact removed much of the fire protection from the steel members of the building, resulting in increased temperatures acting in the steel members. It has also been argued that even without the removal of the fire protection, the heating effect of the fire would have lead to collapse of the buildings .(Quintiere 2002; Usmani, Chung et al. 2003; Flint 2005; NIST 2005). This later interpretation is based on the buildings collapsing not due to weakening of the material due to imposed temperatures, but the effects of thermal elongation.

Thermal elongation could lead to the steel floor members of the WTC towers expanding and then buckling under compression against the core of the building and the external columns, The buckled floor members can would then no longer provide lateral restraint to the (still relatively cool) exterior columns of the WTC towers reducing their ability to withstand axial load.

It is important to realise that the failure of a building due to a fire may not be initiated by material weakening but by other thermal effects.(Usmani and Rotter 2001; Usmani, Chung et al. 2003)

2.8.2 The CESP fire, São Paulo, Brazil

The two building headquarters of the Sao Paulo Power Company (CESP) in São Paulo, Brazil, was involved in a major fire on 21st May 1987. Both buildings, CESP 1 and CESP 2 were constructed of reinforced concrete frames with ribbed slab floors. The plywood formwork was left in place after construction, and the internal partition walls were also constructed from plywood. The buildings were connected by six footbridges at various levels (Beitel 2002).

The fire broke out on the 5th floor of CESP 1 as a result of an electrical failure, and quickly spread up the building due to the large load of combustible material in the plywood formwork and partition walls. The fire also spread to CESP 2, igniting on several levels at once due to the severity of the radiated heat originating from CESP1. The fire then spread quickly through Building 2.

Approximately two hours after the outbreak of fire within CESP 2, the concrete core of the building collapsed, though the entire building structure was not destroyed. The CESP 1 building did not undergo major structural collapse.

The collapse of CESP 2 was later attributed to the expansion of concrete beams. This pushed the vertical load-bearing members out of alignment, and the resulting eccentricity caused the loss of load-carrying capacity (Beitel 2002).

This case study clearly illustrates that the effect of a fire on two buildings with generally similar structure can be very different. This may be due to the fire acting upon the two structures developing differently – in the case of CESP 2, breaking out in multiple areas of the building at the same time.

While individual members may perform well in a fire, consideration must be given to their interaction. It was the expansion of concrete beams, which in CESP 1 survived the fire, which led to the collapse of the core of CESP 2. This shows that testing of structural members in isolation, and their design, must be combined with an overall appreciation of the behaviour of structures.

2.8.3 The Windsor Tower, Madrid

The Windsor Tower was built in 1978 and was at one time the tallest building in Madrid. On 12-13 February 2005, the Windsor Tower was involved in a major fire, of duration 18-20 hours. It was a largely concrete building, consisting of a concrete core and floors, with external steel columns. The building was divided into upper and lower sections by a strong concrete transfer floor (Calavera 2005; Ikeda and Sekizawa 2005; Kono 2005).

The fire started within an office on the 21st floor, causing extensive structural damage to the upper floors of the building. Damage to the lower floors was considerably less.

This fire is of particular interest as a large body of data is available on the structure, facilitating modelling to understand the mechanisms of this partial collapse. The Windsor Tower and its fire will be discussed in much greater detail in Chapter 3.

2.8.4 Channel Tunnel Fire 1996

The Channel Tunnel, a rail tunnel connecting England to France, has sustained 3 major fires since its construction:

- A major fire in November 1996
- A less severe fire in August 2006
- Another major fire on 11^{th} September 2008

The Channel Tunnel is a bored rock construction lined with rings of high strength precast concrete, between 400mm and 800mm thick. It consists of two parallel 'running tunnels' to allow passage of trains, and one internal 'service' tunnel which allows specialised road vehicle access (Bailey 2005).

Both the 1996 and 2007 fires were aboard a train transporting HGVs. In the 1996 fire the number of passengers was restricted to a few HGV drivers in one passenger compartment (Beard 2005). Reports from security guards outside the tunnel led to relatively quick detection that the train was alight prior to entry into the tunnel. This was confirmed by the tunnel fire alarm systems. The train was brought to a halt alongside a cross tunnel, connecting the running tunnel with the service tunnel. After the train came to a halt, thick smoke from the fire enveloped the passenger compartment and the driver's compartment, preventing the passengers from exiting the train. The tunnel's ventilation system was activated to blow the smoke away from the front of the train, allowing the passengers to exit.

After the train stopped, the fire began to grow to a higher intensity – estimated to be around 50MW. With the activation of the ventilation system, the fire grew to a higher level still, possibly around 350MW; an extremely large fire.

The result of this fire was a high degree of spalling from the tunnel lining, in some areas leaving only 51mm of concrete intact with severe damage to the steel reinforcement. Over a 50m length of the tunnel spalling left an average concrete thickness of 170mm.

The spalling was not only life threatening during the fire, due to the explosive nature of concrete being ejected from the structure, but could have potentially severely weakened the structure. Due to the tunnel being within a rock bed, and the relatively short length of the most severe damage, no structural instability occurred and the concrete was successfully repaired.

It should be noted that the severity of the tunnel fire was caused, in part, by the countermeasures put in place to prevent it from causing any loss of human life, i.e. the ventilation. While prevention of loss of life is the most important aspect of fire safety engineering, it should be remembered that this does not necessarily allow for continued structural stability. A structure should be designed remembering that the worst fire loading possible may not be that imposed during normal operating conditions, but the fire which results from some exceptional circumstances which may be brought about deliberately as a result of the fire.

The 2008 fire is still under investigation at the time of writing, though the majority of repair work has been completed and the tunnel re-opened. (Wynne 2009).

2.8.5 Other notable fires

There have been many fires in buildings with a wide variety of structural forms, some of which lead to collapse (Beitel 2002). Other notable fires are the collapse of the concrete slabs in a carpark in Gretzenbach, Switzerland which resulted in the death of seven firefighters (Firehouse.com 2004).

Another recent fire in a concrete framed building occurred in the Architectural department of the Technical University of Delft in the Neatherlands and resulted in collapse (Nu.nl 2008).

These fires demonstrate that while some materials are considered ‘safe’ in a fire, care must be taken examine the effects of fire on all structural forms.

2.9 Conclusions

This chapter has highlighted the advantages and limitations of current research understanding of concrete in fire. A wealth of data is available on the performance of concrete as a material in fire, and the performance of individual structural members. However the performance of a largely concrete structure as a whole is less well understood.

Given the relative lack of information available for large scale fire tests on concrete structures, the current research direction naturally falls towards examining the effects of fire on a real concrete structure as a case study.

It is proposed that the application of a real-life case study can be used to determine the following:

- Creation of a computer model to simulate
    - the structure of a building,
    - fire movement within the building,
    - behaviour of the structure during the fire.
- An intuitive research method, 'the worst credible case' to enable unknown concrete material property variables to be determined.
- Application of the model and material properties to other hypothetical concrete buildings.

Research contact with the GIDAI group at the University of Cantabria (Spain) has enabled access to a significant quantity of data relating to the construction details, post fire photographs, investigative and eye-witness reports of the Windsor Tower fire and the safe demolition of the building (Calavera 2005; Ikeda and Sekizawa 2005; Kono 2005).

The use of the Windsor Tower as a case study is therefore proposed. The next chapter highlights the construction details of this building and response to the fire with an aim of identifying plausible structural configurations and material properties for modelling.

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CHAPTER 3: THE WINDSOR TOWER CASE STUDY

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3.1 Introduction

As briefly discussed in Chapter 2, the Windsor Tower was involved in a major fire on 12-13 February 2005. The fire, of duration 18-20 hours, started in an office on the 21st floor of the building, causing extensive structural damage to the upper floors of the building. As a large fire in a major European capital, there is a significant amount of documentary evidence, both in the form of newspaper reports and television news footage. However much of this appears to be contradictory.

A large amount of other data was available, including the structural plans, media reports and several reports detailing the method of safe demolition post-fire. (Calavera 2005; Ikeda and Sekizawa 2005; Kono 2005)

Much of the data concerning the Windsor Tower was, naturally, in Spanish, though there was also one major report written in Japanese. This often led to problems in data extraction due to language difficulties.

The available information will be examined in the following sections:

- **Structural form of the Windsor Tower**
- **The Windsor Tower Fire**
- **Post – Fire Structural Damage**
- **Possible Collapse Mechanisms**
- **Conclusions**

3.2 Structural form of the Windsor Tower

The Windsor Tower was largely composed of a multi storey office block above an entertainment and shopping complex. The entire building was 32 storeys tall, 29 above ground, with an occupied height of 97m.

The structural data available on the Windsor Tower will be examined in the following sections:

- General layout
- Waffle slab
- Columns
- Fire Protection
- Material Strength
- Loading

3.2.1 General Layout

The upper section of the building, above floor three, was a tower block containing offices and consisted of the following (see Figure 3.1):

- concrete core
- several interior concrete columns
- exterior steel columns
- concrete waffle slab floor with permanent clay formwork
- fire protected steel beams spanning between the pairs of concrete columns
- a concrete edge beam around the perimeter of the building

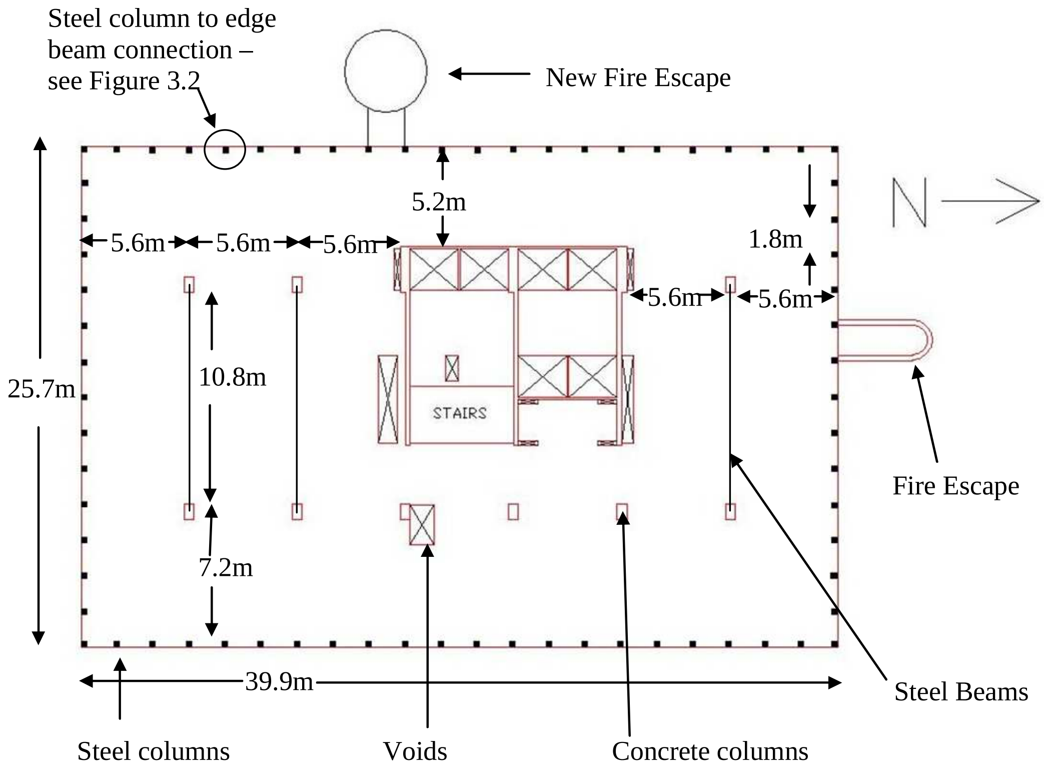

Figure 3.1 Windsor Tower Floorplan above second transfer floor (Calavera 2005)

Two 'transfer floors' in the tower block, located between floors 3 and 4, and floors 16 and 17, supported the tower. These were significantly stronger than the average floor, consisting of a series of concrete walls over the entire floorplan of the tower and a solid floor slab. The two topmost floors, 28 and 29 (the roof), were significantly smaller in floorplan than the rest of the tower and do not appear to have been intended for occupation. The underground levels were used as a multi-storey car park. An external elevation of the tower is shown in Figure 3.3.

The upper transfer floor between the 16th and 17th floors had the same horizontal dimensions as the other floors throughout the building at floor level, and the columns supporting the upper section of the tower rested upon it. It consisted of eight evenly spaced large 'deep' concrete beams, 3.75m deep by 0.5m wide spanning east to west. A smaller beam 1m deep by 0.6m wide spanned between the 'deep' beams on the east and west faces of the building in order to support the steel columns on these sides.

The transfer floor is shown as recessed in Figure 3.3 as the exterior cladding of the building was not continuous between the upper and lower sections of the tower, with the gap occurring at the transfer floor level.

The building had a different structural layout below the 3rd floor with an entertainment and shopping complex along with underground parking. These areas of the building were largely unaffected by the fire and are not recorded as sustaining any major structural damage.

The layout in the section of the tower between the transfer floors was slightly different, with additional concrete columns at the exterior of the building on the north and south sides. The concrete column sections varied from floor to floor, as did the reinforcement.

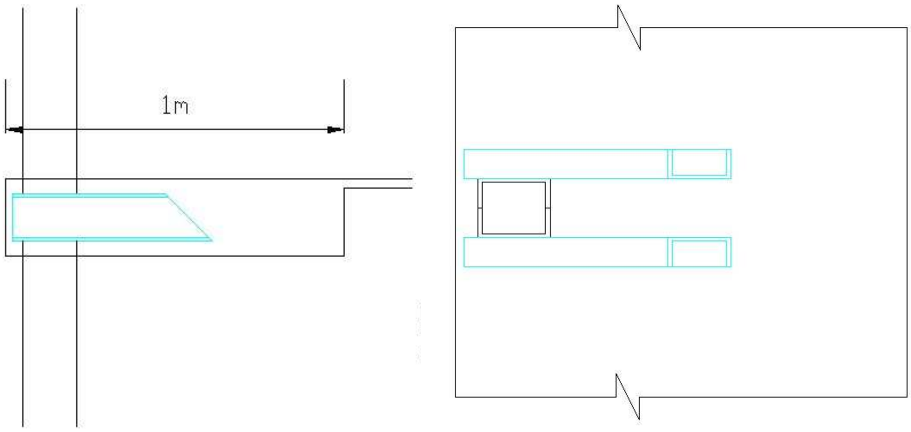

Figure 3.2 Edge Beam Section (Left) and Plan (Right)

As can be seen from Figure 3.2, the exterior steel columns were fixed to the edge beam by a substantial steel section welded at right angles to the column and encased in the concrete slab.

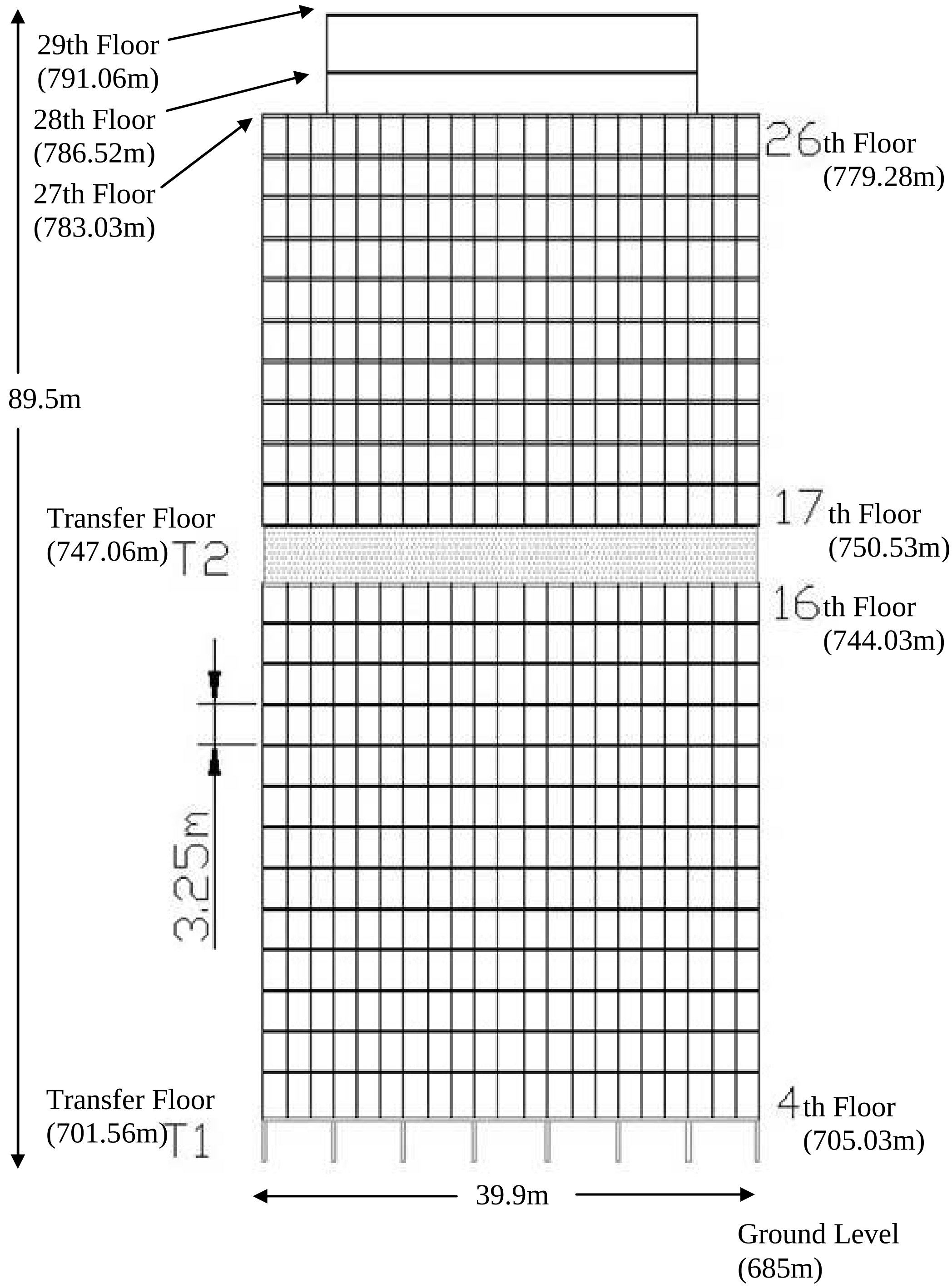

Figure 3.3 Elevation of Windsor Tower

The exterior steel columns were not fire protected at construction. At the time of the incident, a programme of upgrading of fire protection was being undertaken. The steel columns up to the second transfer floor had been largely 'protected'. On the 9th floor two sides of the building remained unprotected due to the sequential nature of the upgrades. An additional fire escape was also added to the west side of the building during this upgrade and was complete at the time of the fire.

The Windsor Tower had undergone a substantial upgrade to its façade. However construction details for this upgrade were not available and it is assumed that no major changes to the structure took place at this time.

3.2.2 Waffle Slab

The floor slab was a waffle slab with permanent clay formwork. The same waffle slab profile is used throughout the tower, though the reinforcement used within the ribs of the waffle varies.

The bottom reinforcement of the ribs running North-South is set onto a channel within the clay formwork. The cover to the bottom reinforcement can be approximated as 20mm thick, consisting of 10mm clay formwork layer and a further 10mm layer of concrete. (Calavera 2005). The bottom reinforcement bars in the beams running East to West are placed on top of this, and therefore have a minimum level of cover of 28-30mm. The bottom reinforcement consisted of 8-12mm bars.

The top reinforcement consisted of 8mm bars, with around 20mm cover in the beams running North-South and 12mm in the beams running East-West.

The ribs of the waffle slab were 0.23m deep including the concrete slab and 0.1m wide at 0.6m centres in both directions (including the concrete floor screed). The concrete floor between the ribs of the waffle was 0.03m deep (Figure 3.4).

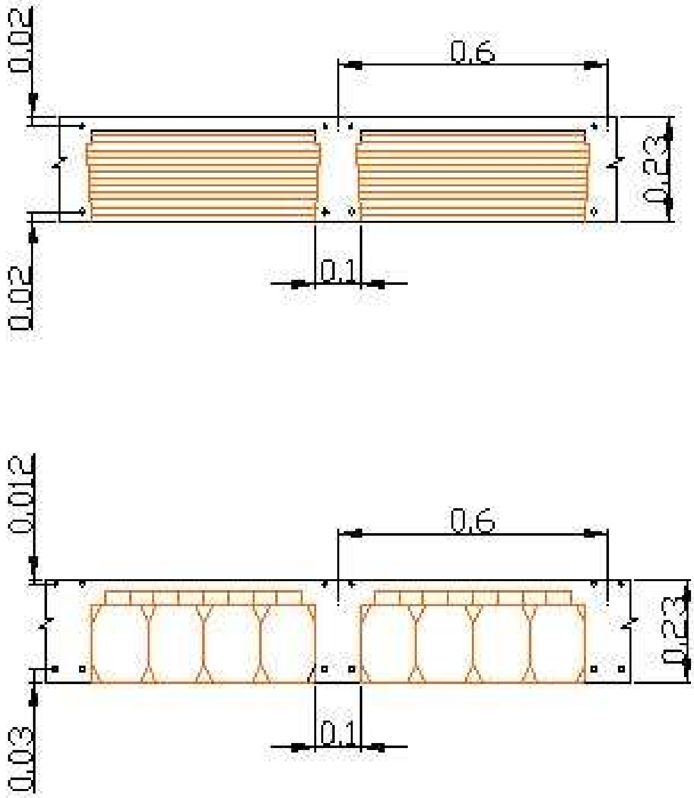

Figure 3.4 Waffle Slab construction (Clay formwork brown, cover approximate) (Calavera 2005)

3.2.3 Columns

The cross-section and reinforcement of the concrete columns varied on a floor by floor basis, as did the cross-sectional size of the exterior steel columns. These steel columns were manufactured from two channel sections facing each other to form a box section, see Figure 3.5. These columns are all classified as Class 1 according to the Eurocode design method (Eurocode 3 2005) or a Plastic cross section in the British Standard design method (BS 5950-1 2000), i.e. sections which will behave plastically rather than buckling.

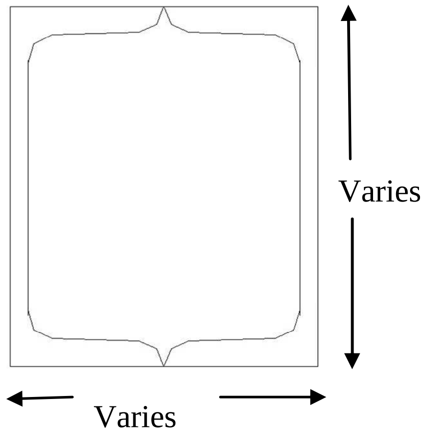

Figure 3.5 Example column cross section

3.2.4 Fire Protection

The exterior steel columns were not, when the building was first constructed, fire protected as this was not required by local building regulations at the time. A programme of fire protection upgrading was being undertaken prior to the fire to bring the building up to newer standards, and the steel columns between the two transfer floors had been protected, with the exception of those on the South and West sides of the 9th floor. The specification of this new fire protection to the steelwork are not known, however the report by Kono et al. (Kono 2005) indicates that 3 hour fire protection was required. It is also noted that fire protection may have been missing from some areas of the 15th floor columns.

There is also some circumstantial evidence that active fire protection measures, such as sprinkler systems, were being installed at the time of the fire. Although this is unverified, it is believed that voids were cut through the floor slabs throughout the building to facilitate this work.

Due to the upgrade of the façade, additional firestopping was also being installed between the edge of the floor slab and the back of the façade (Kono 2005). Although unverified, it is likely that gaps in the fire compartmentation of the building may have existed where this was incomplete.

3.2.5 Material Strength

The actual concrete strength varied from member to member, however the slabs' specified strength is 175 kg/cm^2 (17.5 MPa) for all floors. The aggregate used in the concrete is unknown, however the INTEMAC results show a correlation between test results and values expected from a Eurocode Siliceous concrete (Calavera 2005).

The Steel strength specified for the reinforcement is 5000 kg/cm^2 (500MPa).

3.2.6 Loading

Design live loads are also specified in the structural plans for the building. In the majority of the floors, this was a live load of 430kg/m^2 (4.2 kN/m^2), with a façade load of 200kg/m (1.96 kN/m) prior to installation of the new façade.

3.3 The Windsor Tower Fire

The fire, originating high up in the building, spread upward to the top floor. This is likely to have occurred due to a combination of external fire spread, whereby windows of the building break and allow the fire to re-enter the building on higher floors, and internal fire spread through the service voids (Fletcher 2006; Hitchen 2006). The fire also, somewhat unusually, spread down the building.

The entire tower section of the building was involved in the fire, with localised consumption of flammable materials resulting in the fire progressing in stages through the building.

In the areas above the second transfer floor, large areas of the floor slab exterior to the core collapsed and several of the concrete columns were also damaged.

In the lower levels of the building, significantly less damage occurred. On the 9th floor, which had large areas of unprotected steelwork, the exterior steel columns were later found to have buckled severely. However, this part of the building did not collapse.

3.3.1 Ignition on the 21st Floor

It is well-established that the fire broke out on the 21st floor, in office 2109 at approximately 23:05hrs (Calavera 2005; Ikeda and Sekizawa 2005; Kono 2005). Detection occurred at 23:08hrs; and a '50cm flame' was reported to have been seen there at 23:18hrs by night staff investigating the alarm. This is consistent with a waste-paper basket fire or similar source. The fire service was called at 23:21hrs.

A great deal of analysis of the spread of this fire around the 21st floor has been carried out by the GIDAI group (Capote 2006). A key factor affecting the fire spread through the floor is the changes in ventilation characteristic caused by window breakage. This is difficult to document for many reasons:

- Lack of 360˚ footage of the building
- Fire took place at night
- Percentage of window breakage difficult to observe – whole or partial

Similar factors also make it difficult to judge the progress of the fire around the floor from outwith the building, as a visible 'glow' within the floor may be from a fire some distance away. External flaming is an obvious indication of a locally flashed-over fire, however this will not necessarily be visible around the entire floor perimeter at once and the fire may not have developed further within the floor.

3.3.2 Fire Spread Up and Down the Building

A key feature of the Windsor Tower fire is that the fire spread both up and down the building. The rate of upward spread is reported differently in a number of sources. The INTEMAC report mentions an average spread rate of 6.5 minutes from floor to floor. Meanwhile, the report by Kono et al. (Kono 2005) provides a more detailed timeline compiled from several sources, indicating the rate of fire spread between each floor. This timeline also shows a downward spread rate of 20 to 30 minutes. A version of this is replicated below (Figure 3.6).

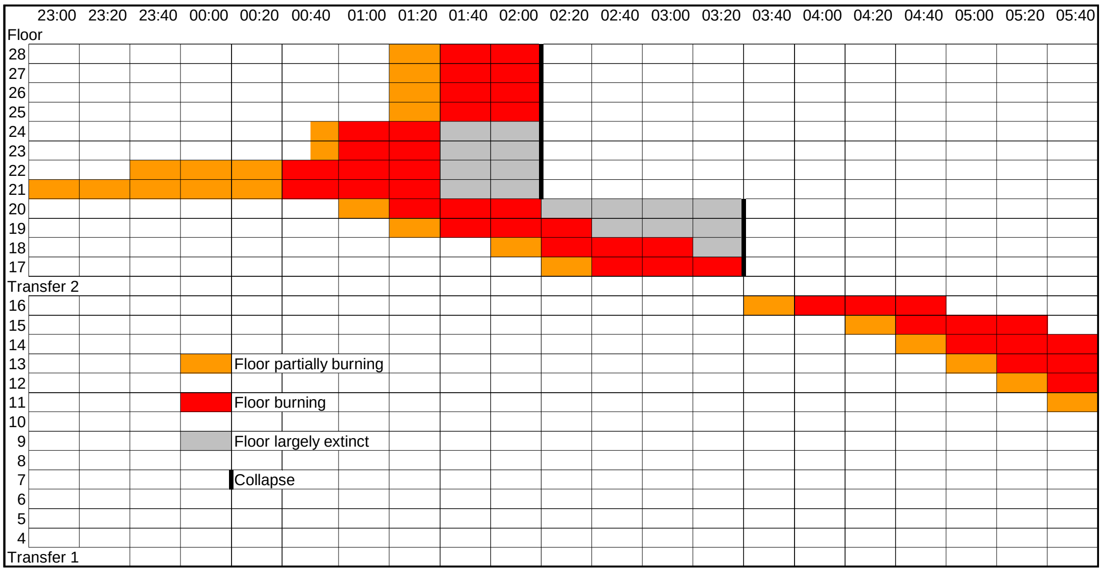

Figure 3.6 - Windsor Tower Fire Timeline (Kono 2005)

Again it is difficult to tell if a floor is fully involved in the fire. If there is internal fire spread, it is impossible to see any evidence of this until it reaches the perimeter of the building – conversely if the fire is spreading from the perimeter of the building inward, it is impossible to know if or when it reaches the core. As all the sides of a floor will not show evidence of fire at the same time, any timeline must be regarded as approximate.

It is unclear what mechanisms were involved in the vertical fire spread. There is evidence that external fire spread was involved to some extent. The windows on a burning floor broke allowing flames to exit the fire compartment, with the heat from this ‘spill plume’ breaking the windows on the floor above and allowing ignition of the material within the new floor (Hitchen 2006). Fire may also have spread through voids in the floor slabs, along with gaps in the firestopping between the floor slab and the façade.

While external fire spread is convincing in the case of upward spread, it is harder to justify in the case of downward fire spread. It is my speculation that burning materials may have fallen through voids and gaps in firestopping to lower floors. The relatively regular timescale of the downward spread suggests that some combustible fire compartmentation barriers may have been present with a fixed time until failure allowed burning materials to fall into the compartment below.

The upper transfer floor would still be a significant barrier to downward fire spread regardless of its mechanism. However it is suggested by Kono et al. (Kono 2005) and post fire damage observation (Calavera 2005) that the impact of the upper floors collapsing may have cracked the floor slabs of the Transfer Floor allowing burning material to enter the lower half of the building.

3.4 Post-Fire Structural Damage

The level of structural damage varied greatly within the tower. In the lower section of the tower, between the two transfer floors, the structure appeared to have survived largely intact. This includes the 9th floor, where there was no fire protection on large amounts of steelwork. As can be seen from Figure 3.7, Figure 3.8 and Figure 3.9 these unprotected steel columns buckled severely.

Figure 3.7 Post fire damage, Buckled Columns (Daniel Alvear Portilla, Group GIDAI, University of Cantabria)

Figure 3.8 Post fire damage, 9th Floor Highlighted (Daniel Alvear Portilla, Group GIDAI, University of Cantabria)

Figure 3.9 Post fire damage with demolition partially complete, 9th Floor Highlighted (Daniel Alvear Portilla, Group GIDAI, University of Cantabria)

Above the upper transfer floor, the level of structural damage was much more severe. Figure 3.10 shows the area which collapsed through much of this section of the building. Within this section of the building, fire protection had not been installed on the structural members.

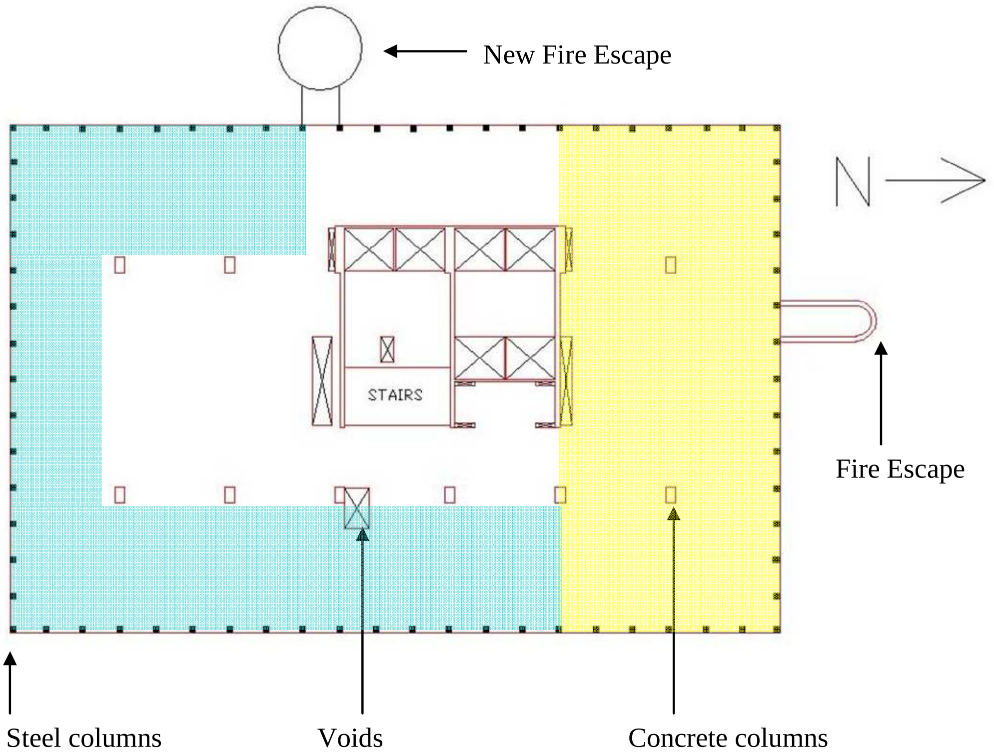

Figure 3.10 Diagram of collapsed areas (Calavera 2005)

3.5 Possible Collapse Mechanisms

The first area of interest is the external area of floor slab spanning between the concrete core of the building and the steel columns, coloured blue in Figure 3.10. It is proposed that with the reduction of steel strength due to heating of the unprotected columns, this area acted as an unsupported cantilever. At this point the slab would be unable to sustain its own weight in cantilever action (Figure 3.11).

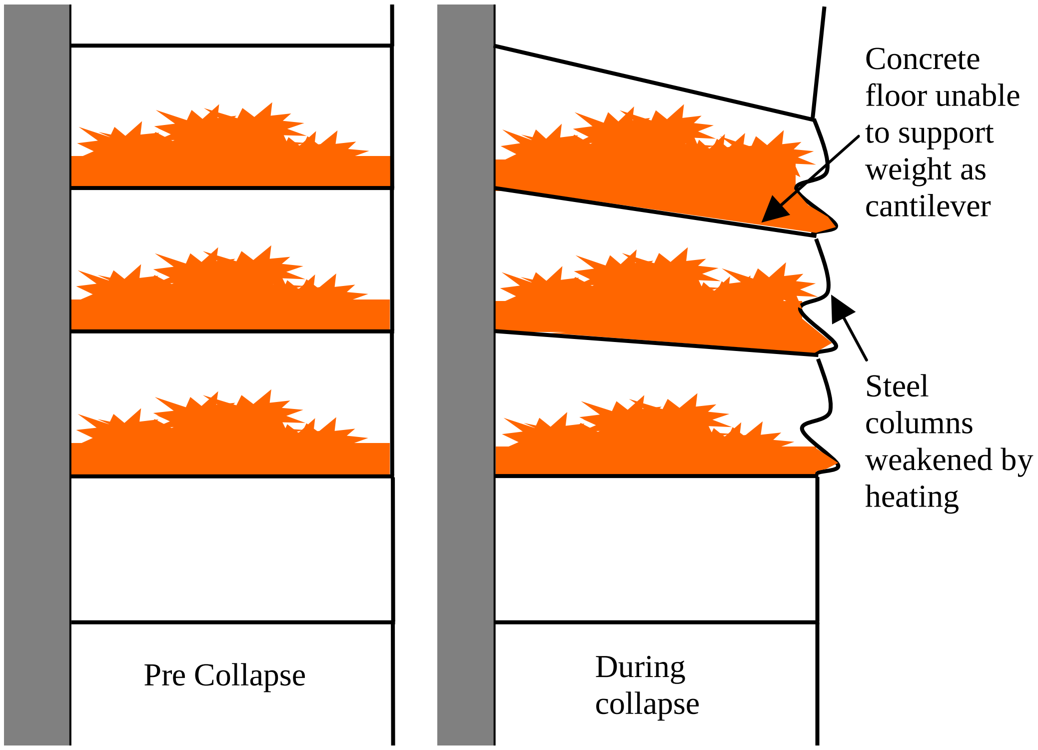

Figure 3.11 - Possible collapsed mechanism of perimeter floor area pre-collapse (left) and during collapse (right)

It can be seen that in the area where the concrete floor spans between the concrete core and the new fire escape, collapse did not take place. It is proposed that the new fire protected escape prevented this collapse. Alternatively, the concrete shear wall of the core may have had a higher supporting effect on this area of the slab than would be provided by a concrete column, enabling cantilever action.

In the interior area which underwent collapse, coloured yellow in Figure 3.10, it is less clear what may have initiated collapse. It has been suggested that the concrete columns failed under the fire, however given their lack of failure in the lower section of the building and the higher strength of the column concrete compared with the floor concrete this seems unlikely. A proposed alternative is that with the exterior area undergoing collapse, tension would have been applied to the floor slab between the concrete columns and the concrete core. As can be seen in the plan, this area of concrete is severely reduced by voids, and may have been insufficient to withstand this tension force (Figure 3.12). This collapse mechanism relies on the connections between the floor and the concrete column being able to withstand extremely high forces in order to cause overturning and this may be unrealistic.

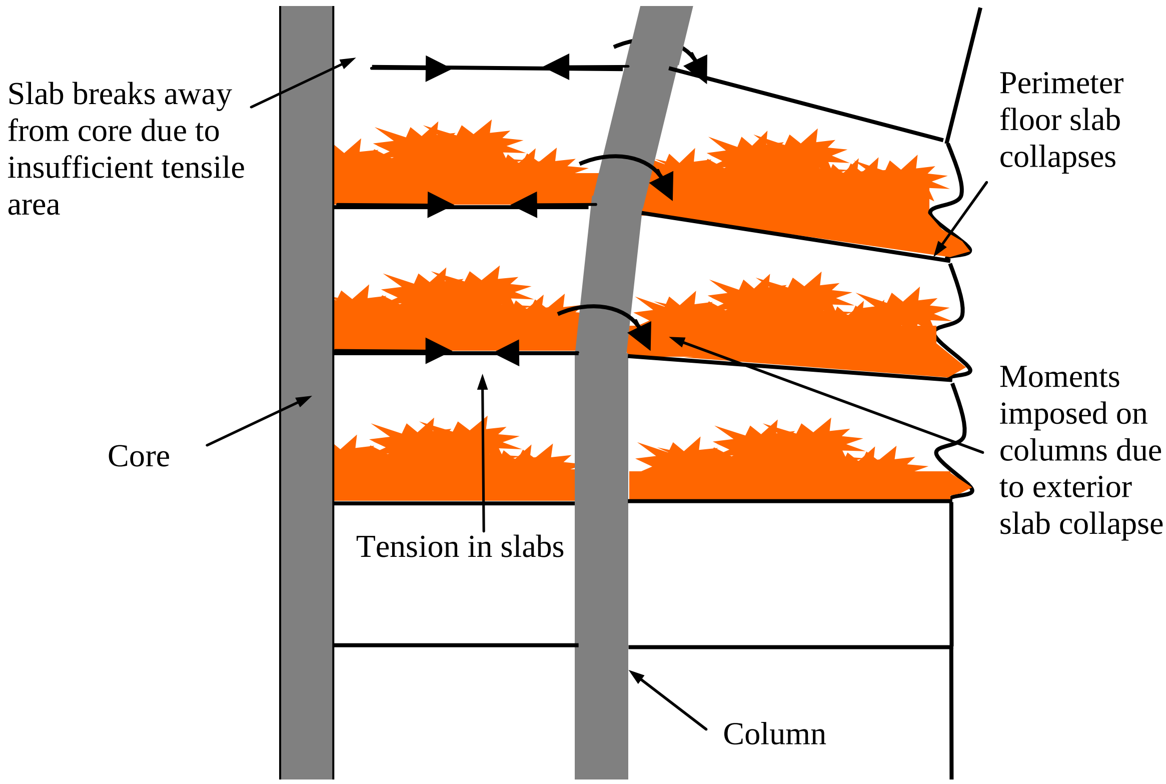

Figure 3.12 - Possible collapsed mechanism of internal floor area and concrete columns

Many alternative mechanisms are possible for the failure in this area, and the concrete floor slab between the core and the concrete columns may have failed leaving the concrete column 'freestanding' and unable to remain upright.

This proposed collapsed mechanism was not studied further but would be a valuable area of future research.

The concrete core of the building appears to have performed well throughout the fire, undergoing no collapse and apparently maintaining its own compartmentation, as fittings within the core were reported to be undamaged by heat (Calavera 2005).

3.6 Conclusions

The Windsor Tower, in those areas where fire protection was installed, performed remarkably well when subjected to a prolonged fire.

Two collapse mechanisms are proposed, along with potential methods of fire spread up and down the building. In the next chapter, there will be further discussion of which mechanisms to investigate further.

CHAPTER 4: RESEARCH METHOD

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4.1 Introduction

As can be seen from Chapter 3, a variety of areas of study within the Windsor Tower are available.

It is advantageous to examine the phenomena at work within the Windsor Tower with a view to applying them to other similar structures rather than restricting the analysis to a forensic study.

Possible methods to analyse the phenomena highlighted in the previous chapter will be examined in the following sections:

- **Establishing a modelling framework**
- **Modelling the fire**
- **Modelling heat transfer**
- **Modelling the structure**
- **Conclusions**

4.2 Establishing a modelling framework

Within the fields of structural and fire engineering, there are many different modelling packages and methods available. In order to model the fire that took place in the Windsor Tower and reaction of the structures, it will be necessary to create several models. These models must cover:

- Characteristics of the fire (Temperature, spread and duration)
- Heat transfer, allowing the following is assessed:
  - The depth of high temperature penetration in the concrete members
  - The internal temperatures attained
- The structural response of the building during the fire and cooling period, including:
  - The interaction of the concrete members with the fire
  - The interaction of the steel columns with the concrete members and their contribution to the holistic structural integrity of the structure during the fire and cooling phase.

Modelling packages and methods exist which can model either one, two or all three of these factors.

This chapter further investigates the methods of attaining these three modelling pre-requisites.

4.3 Modelling the fire

As discussed in section Chapter 2.3 many methods of characterising the fire imposed on a structural member are available. These range from the ISO curves to advanced Computational Fluid Dynamics (CFD) models. CFD modelling allows examination of localised effects within a fire compartment(s), including:

- Peak temperatures or heat fluxes at a specific structural member
- Gas flow around a compartment
- Mixture fraction – i.e. ratio of oxygen to fuel
- Re-radiation from heated objects within a compartment
- Smoke movement
- Fire development and spread
- The time to burnout of an area or compartment

Many CFD packages are available, each with advantages and disadvantages. These range from commercial products, to open source freeware, to packages developed within academic institutions for specific research interests.

Three such packages were investigated during the course of this research.

- ANSYS/CFX
- SOFIE
- FDS Version 4

The main advantages and limitations of each package is summarised in Table 4.1.

	ANSYS/CFX	SOFIE	FDS Version 4
1. Package Type	*Commercial* Difficulty obtaining licenses	*Free* Research/Commercial developed by BRE	*Freeware * Fire Dynamic Simulator program published by NIST
2. Training attended	1 week course undertaken	YES	NONE
3. Technical support	YES	YES	YES, but limited
4. Experience /expertise level required to achieve output.	High, as models often stop due to non convergence errors.	High, as models often stop due to non convergence errors.	Low, However, can give false impression of accuracy as models can easily run to completion
5. High level of expertise available within research group	NO	YES	NO
6. Ability to link to structural analysis package(s) i.e CFD to FEM.	YES	NO, not as standard	NO, not as standard
7. Graphical interface	YES (Good)	YES (Good - can simplify model development)	YES (Poor for development, good for results)
8. Specifically designed to model fire	NO	Yes, however mainly used for modelling "steady state" fires rather than spread and extinction	Yes, though originally designed to model smoke transport rather than fire.
9. Ability to model fire spread	NO	YES, but not to multiple materials	YES
10. Past Windsor tower models?	NO	NO	YES

Table 4.1 comparison of application packages

The FDS Version 4 package (McGrattan and Forney 2006) was chosen for the principal reason that a large amount of CFD modelling has already been performed by the team at GIDAI from the University of Cantabria, Santander using this package.

The University of Edinburgh has a strong collaborative relationship enabling the use and modification of GIDAI's FDS models (Capote 2006).

A major disadvantage of using FDS is the time consuming nature of transferring CFD results into the heat transfer or structural models, as there is no widely available automated method at present.

Alternative methods are being developed to couple a variety of CFD and fire modelling packages with structural modelling packages. (Liang and Welch 2007; Welch, Miles et al. 2008)

As CFD fire modelling can be computationally intensive, a parallel research approach was used to mitigate large time gaps between initiating the model and receiving results. This allowed work to continue while results of the CFD analysis were pending.

- The CFD model would be set up and run
- A more simplified but credible fire definition was used for the initial structural models until CFD results were available.

4.3.1 Simplified Fire Modelling Method

The simplified method involved developing a ‘basic’ fire curve based on the ISO curve, with modifications to take into account the cooling phase which exists in a real fire. A one hour linear cooling phase was used.

While the ISO fire curve will not precisely model the temperature within the fire compartment, as noted in Chapter 2.3, it provides a reasonable simplified fire definition.

This could be further expanded by comparison of the simplified model with the results of the time and computer intensive CFD method. This potentially allows the simplified fire definition to be applied to later structural models if there is reasonable agreement.

4.3.2 CFD method

While in might be desirable to perform fire and structural modelling of the entire Windsor Tower, the level of detail required makes this is impractical. The duration of computational time and the length of time that would be required to develop the models, input them and interpret the results would be excessive.

A highly detailed model of the building, especially for the fire, would also give the erroneous impression that few assumptions have been required and that the internal layout and usage of the building throughout its height is well known. It is reasonable to assume that partition walling, workstation layout, storage areas and other fire loads and compartment boundaries will vary from floor to floor throughout the building. The precise floor-by-floor layout is not known, though the generalised office layout of the 21st floor is provided by the GIDAI group.

It was necessary to identify individual areas of the Windsor Tower which could be assumed to be broadly representative of the overall structure. In the case of the fire, the model produced by GIDAI covered the entire 21st floor, and was developed in order to examine the rate of fire spread throughout the floor from a single point of ignition within a single office. This level of detail and extent is not required for a structural study, where the main interest in the fire is the duration of exposure of a given area and the maximum thermal exposure. In order to reduce computational time, the floor could be approximated as biaxially symmetrical, and therefore mirrored about the x and y axis. As the 21st floor is likely to be the only one on which the fire spread from a single point of ignition, this will also reduce what would otherwise be an excessive localisation effect and result in a more representative model.

This analysis could then be approximated to modelling the fire conditions occurring on all other floors, though it does not model the case where there are a large number of simultaneous points of ignition.

4.4 Modelling heat transfer

A variety of programs and techniques are available to approximate the heat transfer, depth of penetration, and reinforcement temperatures within the concrete member. The results of the heat transfer analysis can then be imported into a structural analysis package.

This heat transfer can either be performed by subroutines within most structural modelling packages, an additional external program or even hand calculations.

Hand calculations, even if automated with a spreadsheet, would be very complicated, and were therefore ruled out other than for checking model ouput.

The heat transfer capabilities of the Finite Element package Abaqus were used. This is capable of transferring large volumes of data into the structural analysis package.

4.5 Modelling the structure

As with the fire and the heat transfer, there are several methods available for modelling a structure, including:

- ‘Hand calculation’ (which in reality would use a spreadsheet) using either:
    - first principal stress/strain analysis,
    - adapting the design rules given a structural design code such as (BS 8110 1997; Eurocode 2 2003).
- Computational methods

Hand calculation is made problematic when investigating a structural element composed of multiple materials, and especially when the material involved has the highly insulating properties of concrete. Large temperature gradients will be present in a concrete member, resulting in material properties varying both temporally and spatially. This makes any hand calculation extremely difficult; however it is useful to perform some degree of hand calculation at ambient temperatures in order to confirm the results produced by computer modelling.

The level of complexity involved in this analysis generally necessitates the use of computational methods. The temperature data generated by the heat transfer analysis is applied to the structural system allowing temperature dependant physical properties to be automatically varied temporally and spatially.

A Finite Element analysis, where the structure is broken down into a series of smaller elements, will be used. There are a wide variety of commercial, freeware and academic Finite Element packages, some of which are specifically optimised to examine the effects of fire on structures. The following packages were examined.

The ANSYS/CFX suite of programs, as mentioned in Section 4.3, can carry out structural analysis, CFD analysis and heat transfer. However, since CFD analysis is to be carried out using another package (FDS), the automatic coupling of this package would not be used.

Abaqus is a commercial structural modelling package produced by Simulia and is widely used at the University of Edinburgh. Abaqus has two solution methods; either 'Standard' or 'Explicit'. With both these solvers, the analysis is divided into a number of timesteps. (Abaqus 6.8-1 Manual 2008). A brief explanation of the two modes is given below:

- Abaqus Standard solves a series of simultaneous equations for each timestep, with iterations being used to ensure convergence to a solution. This is computationally expensive for each timestep, and convergence may not be achieved in very non-linear analyses. However, the size of the timesteps used can be varied a great deal, often allowing the model as a whole to be run quickly.
- Abaqus Explicit does not attempt to reach a converged solution to each timestep. It calculates the accelerations within a model at the start of each timestep based on mass, applied loads and internal forces. This is then used to calculate displacements and velocities. A 'Double Precision' version of this executable is available, utilising longer floating point words and hence increasing accuracy at the cost of some computational speed.

An Explicit analysis typically uses many more timesteps than a Standard analysis, however the calculation time for each is shorter. This can result in shorter computational times for large or complex models and can allow the modelling of extremely discontinuous events (such as collapse), large deformations and contact forces.

No iterations are used in an Explicit analysis, nor is there any automatic accuracy checking. The run time of many explicit models can be long, but techniques such as time-scaling and mass-scaling can reduce this. Time-scaling involves artificially compressing the model's duration to reduce the number of timesteps whilst mass-scaling involves increasing the mass of the model for the purpose of inertia, to allow longer timesteps to be used.

A great deal of expertise in the use of the Abaqus package exists within the fire group at the University of Edinburgh. This has resulted in a large amount of pre-existing material data and a great deal of support within the department.

It was decided that the most current version of the Abaqus package (which was upgraded several times during the course of the project, from version 6.6-2 to version 6.8-1) would be used, using whichever solver (Standard or Explicit) was most appropriate for the model being examined.

4.5.1 Structural Model Method

There is a greater degree of certainty when developing the structural model compared to fire the model as the original building plans give some indication of the geometric layout.

While details of the refits to provide the new curtain wall and façade are not known it is not believed that any major structural modification took place at this time. Details of the voids cut through the floor slabs during the fire upgrading process are also unknown. These unknowns are of more importance for the fire model than the structural model, and will therefore be assumed to be unchanged since construction.

To decide the extent of the structural model, it is necessary to establish areas of particular structural interest. In the case of the Windsor Tower, these are:

- The partial fire protection and lack of collapse of the 9th floor,
- The concrete floor slab and concrete columns near the building core which appeared to undergo some degree of failure.

As discussed in chapter 3.5, the internal section of floor and concrete columns which collapsed may have had their failure triggered by the collapse of the external floor slab. This would apply tension to an excessively small area of concrete ‘unzipping’ the internal floor and columns from the core.

Study of the failure of the internal floor section and concrete columns would require study of the effects of fire on the peripheral floor section. The study of the peripheral floor section would largely coincide with the study of the lack of collapse of the 9th floor.

Therefore, the structural stability of the 9th floor becomes a focus of research. This will also allow conclusions to be drawn on the requirements for fire protection or major fire resistant structural members in other buildings such as transfer floors.

Two theories of prevention of collapse are proposed:

- The transfer floor above supporting the weight of the floors with the steel columns acting in tension.
- The fire protected steel columns and the concrete floor slabs transfer the load back to the core of the building in some form of cantilever action.

In order to evaluate these alternatives, or identify any others which may be involved, it is necessary to create a structural section or sliced view through the building encompassing at least one of the steel columns (Figure 4.1, Figure 4.2.)

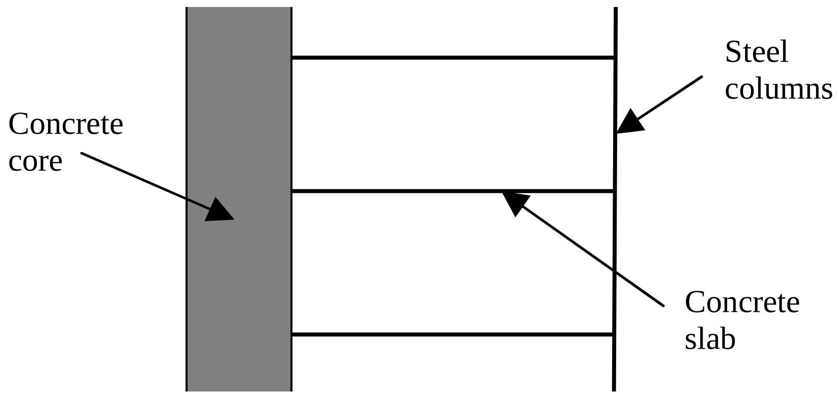

Figure 4.1 Example Structural section

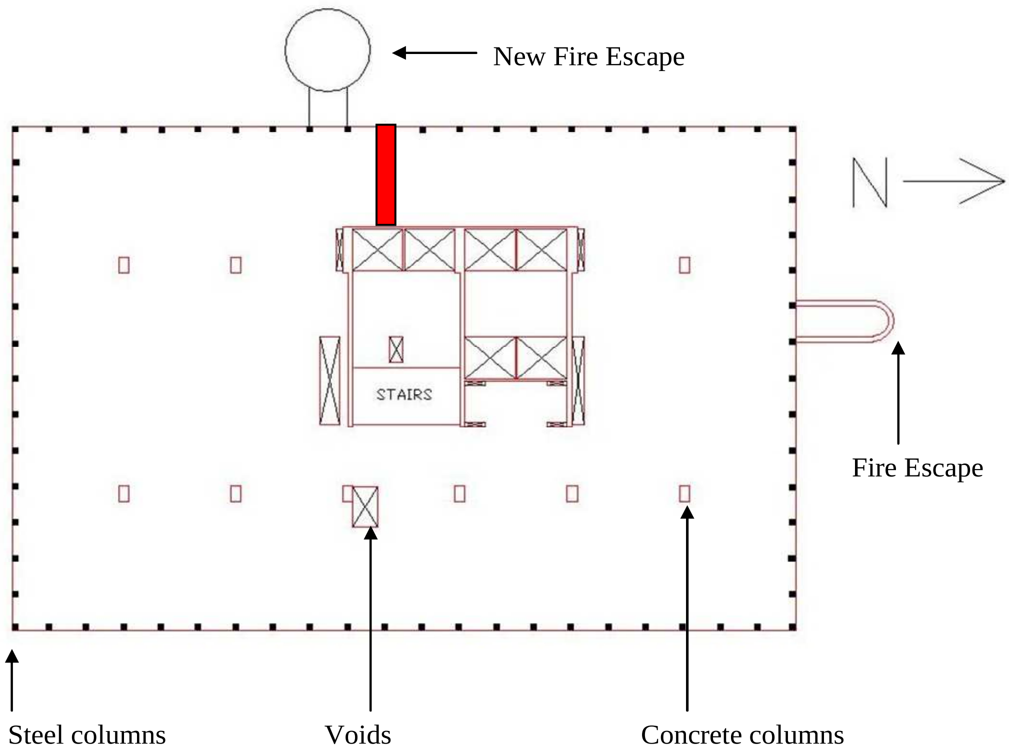

Figure 4.2 Structural Section location

This particular location within the building was chosen because of the equal span and reinforcement arrangement of the concrete slab on all floors between the core and the steel columns, both above and below the transfer floor. It is also in the area where fire protection had not been applied to the 9th floor (the West face of the building).

It should be noted that this particular section remained intact both below and above the transfer floor. In the case of the upper stories this may be due to the presence of the new fire protected fire escape which remained intact throughout the fire.

The inputs to this model can then be varied to represent hypothetical cases and to demonstrate the importance of combinations of multiple burning floors, upward and downward spread and presence or lack of fire protection.

For the model of this section of the building to be established, the development of smaller models was deemed necessary to test model variables.

4.6 Conclusions

This chapter has highlighted the advantages of the FDS4 and Abaqus packages, and these will therefore be used for subsequent fire and structural analyses, respectively.

Due to the level of assumptions required for both the fire and structural models, it will be impractical to carry out a totally forensic model of the Windsor Tower. However, a Windsor Tower-like structure can be modelled and the results shown to approximate the real world fire.

The specific collapse mechanism to be examined will be the failure of the perimeter area of slab, as this is a precursor to the 'unzipping' mechanism. This can also be compared with an equivalent area of the building where collapse did not occur, the unprotected areas of the 9th Floor.

CHAPTER 5: TESTING MODELLING VARIABLES

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5.1 Introduction

Due to the number of unknown variables required for a fire or structural analysis it is necessary to develop a 'worst credible case' method.

This is distinct from the 'worst *possible* case', as some degree of engineering judgement and comparison against known results is used to decide on the final values for future analysis.

This will be used to develop a fire case for application to a structural model and a set of physical properties for the concrete.

This will be examined in the following sections:

- **Fire Modelling**
- **Structural Modelling**
- **Sensitivity Analysis**
- **Conclusions**

5.2 Fire Modelling

As discussed in Chapter 4, two methods will be used to model the fire – a simplified ISO-based curve and Computational Fluid Dynamics.

A great many assumptions were required about the fire loading, geometry and ventilation characteristics of the Windsor Tower, with a large majority of such details being unknown.

5.2.1 CFD Model

As discussed in Chapter 4, the fire model chosen was based upon a CFD fire model originally developed by GIDAI (Figure 5.1). This model was originally developed to examine fire spread rates through a floor of the Windsor Tower and used an office arrangement reflecting the layout believed to be in place on the 21st floor, where the fire initially broke out. (Capote 2006)

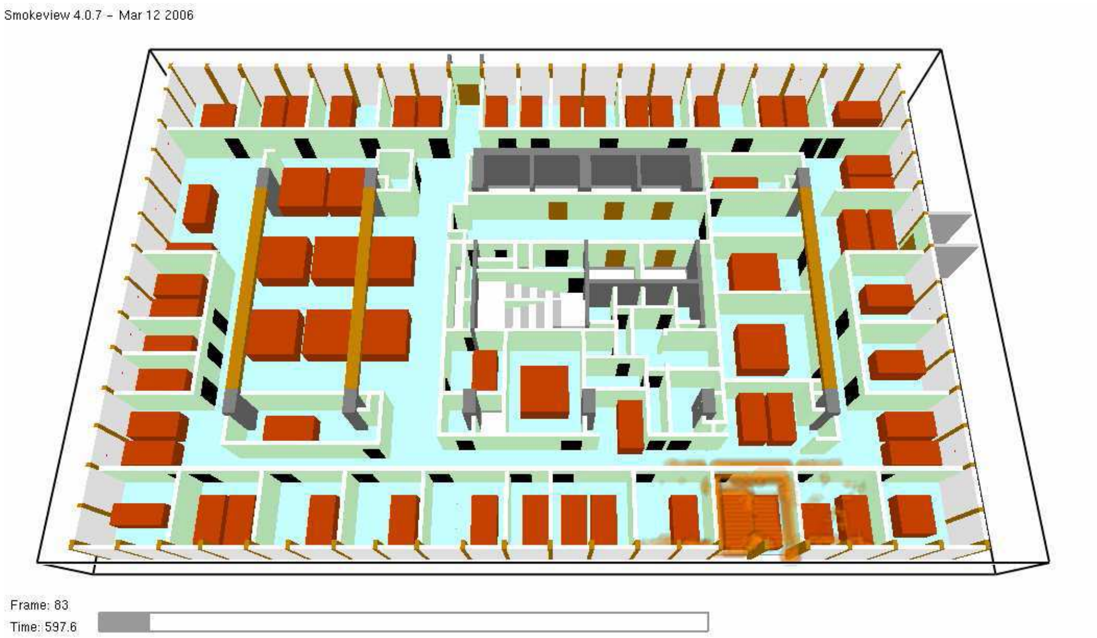

Figure 5.1 Original CFD model, Group GIDAI, University of Cantabria (Capote 2006)

Several adaptations to GIDAI's original model were made for the current research, shown in Figure 5.2:

- A more 'generic' layout to the structure was developed. This was achieved by removing all the office doors and therefore producing an 'open plan' layout, though with a similar quantity of combustible material.
- Quartered and mirrored the floorplan to reduce the computational run time required. This was decided as the level of information required from the fire model was less than for a full fire spread analysis.

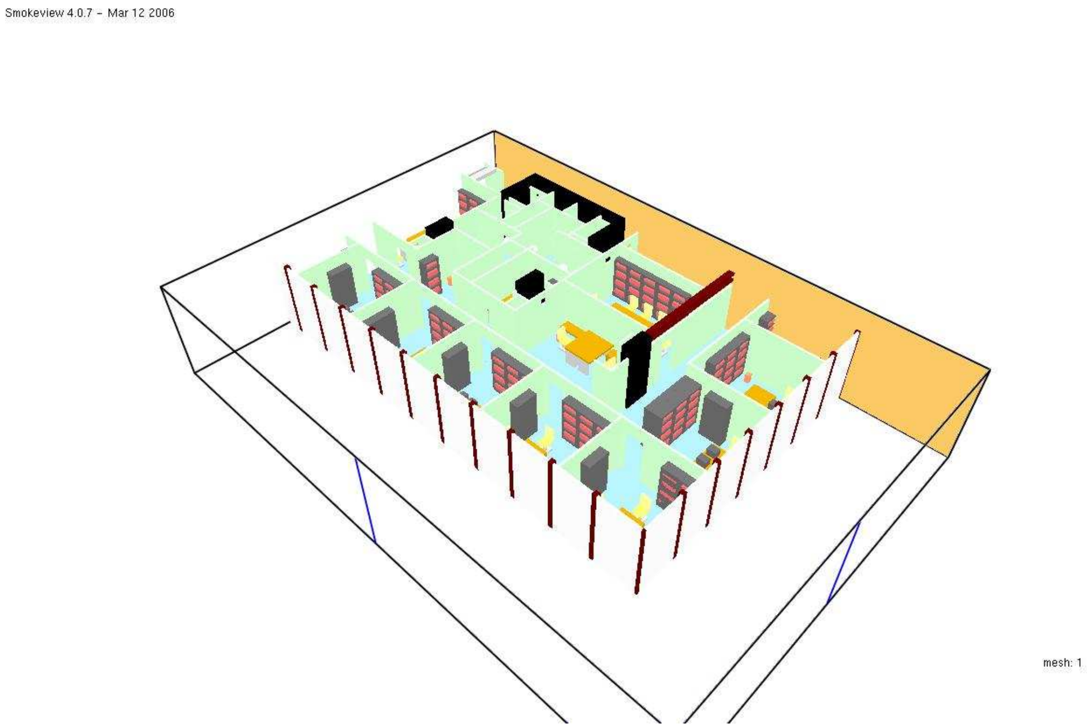

Figure 5.2 Altered CFD model

A reasonably fine grid size of 0.1m x 0.1m x 0.1m cubic cells was used, uniformly. This decision was based on previous research from GIDAI whereby size was shown to effect on the output.

The multiprocessor capabilities of FDS were used to further break the computational domain down into 4 grids, each calculated by a single processor. This can lead to some degree of inaccuracy as data is transferred between the computational domains.

Any inaccuracy produced by the subdivision and simplification of the model must be weighed against the number of other assumptions required.

The initial assumptions are likely to introduce a larger degree of variability and therefore any "inaccuracy" due to these simplifications can be ignored.

The windows around the perimeter were set to break in pairs when a temperature sensor placed beside them reached a threshold of 150˚C. This would have an effect of allowing increased ventilation to the fire compartments and is consistent with the spill plume observed in the Windsor Tower.

To improve modelling of the flames exiting the windows (Spill plume) a gap of 4.8 metres was left around the perimeter of the floor. Additionally the thickness of the upper floor was increased to an unrealistic level so that the flames exiting the windows would not 'loop back' over the top of the upper floor slab unrealistically before escaping from the domain.

Once the model had been developed, 4 processors within the University of Edinburgh's CLX cluster of machines were used to compute the results.

The computational time required to complete this model for a 1 hour fire duration was approximately one month. Figures 5.3-5.8 show model frames, demonstrating the fire spread.

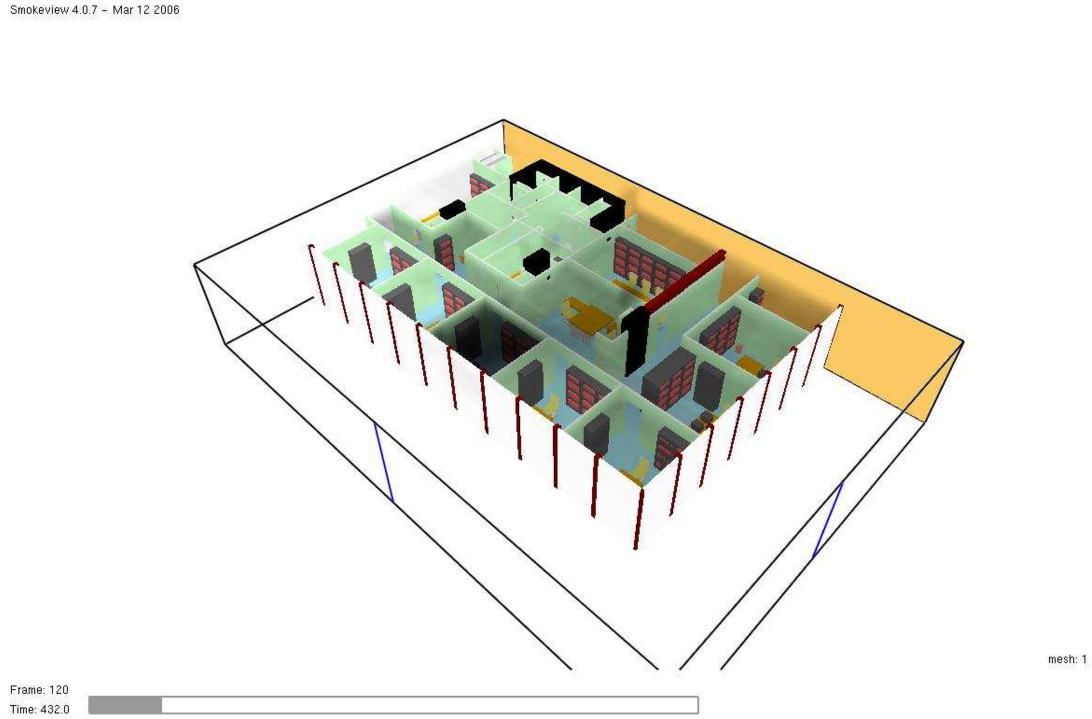

Figure 5.3 Smoke spreading from initial fire compartment

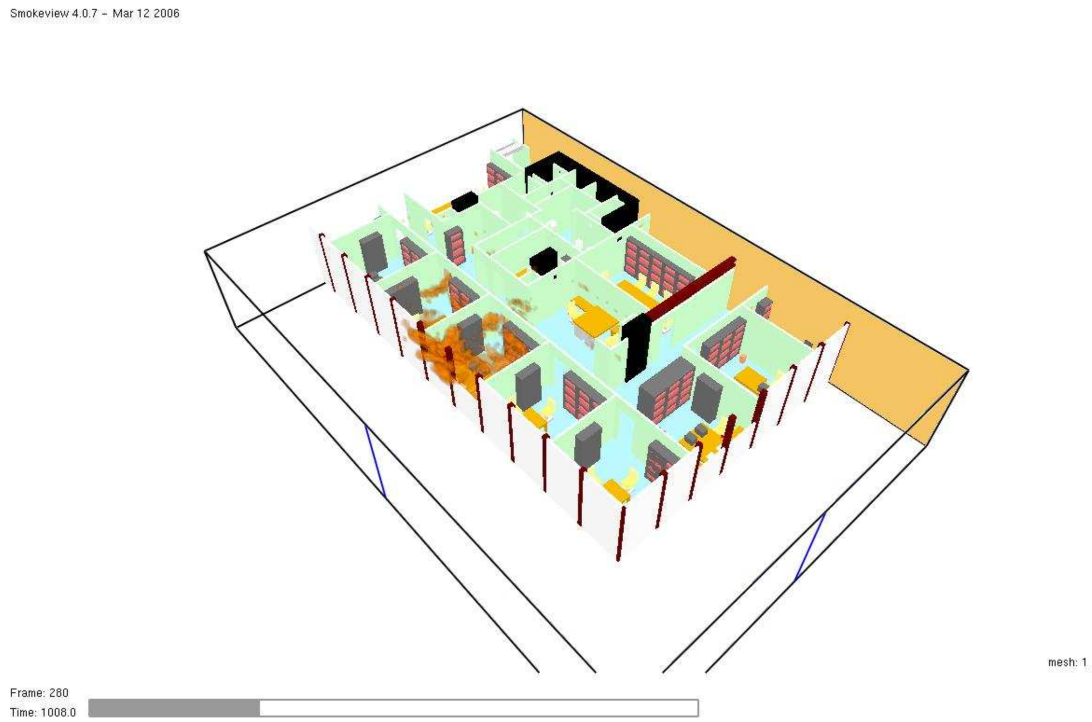

Figure 5.4 Flames breaking out of initial fire compartment window

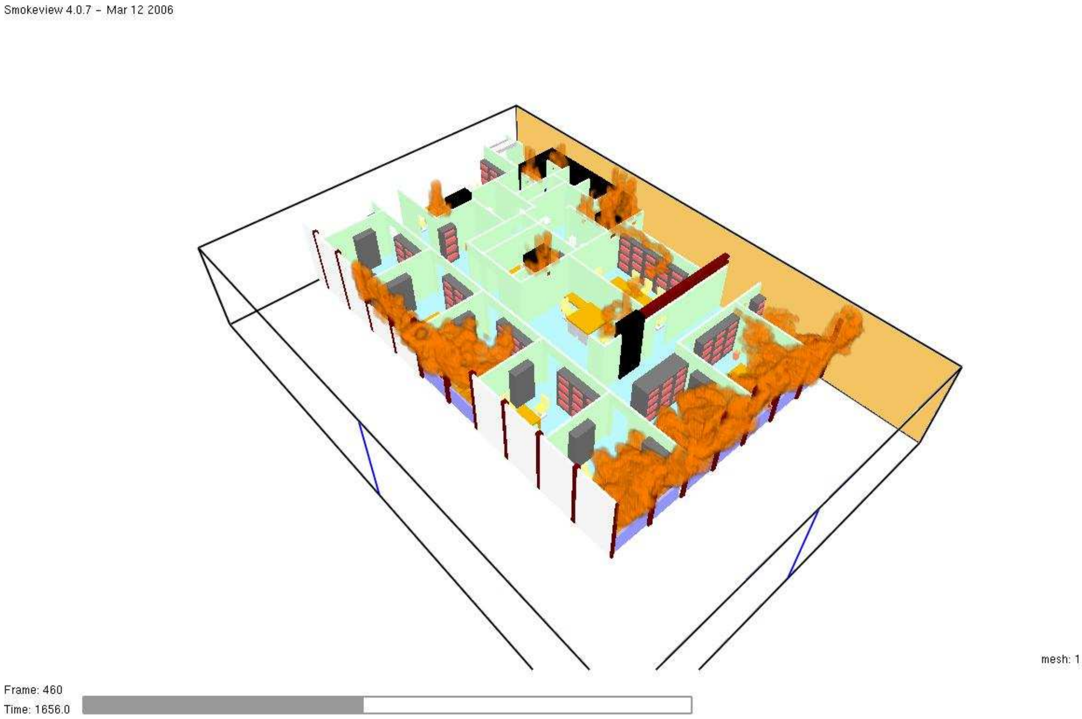

Figure 5.5 Fire spreads through floor

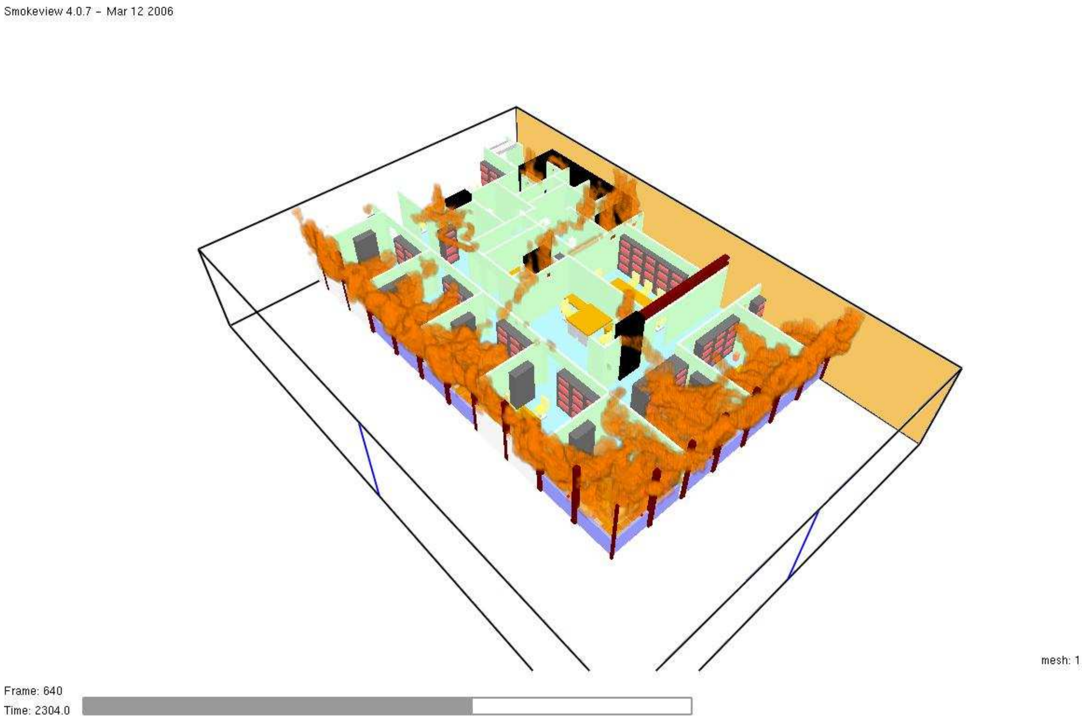

Figure 5.6 Fire through most of floor

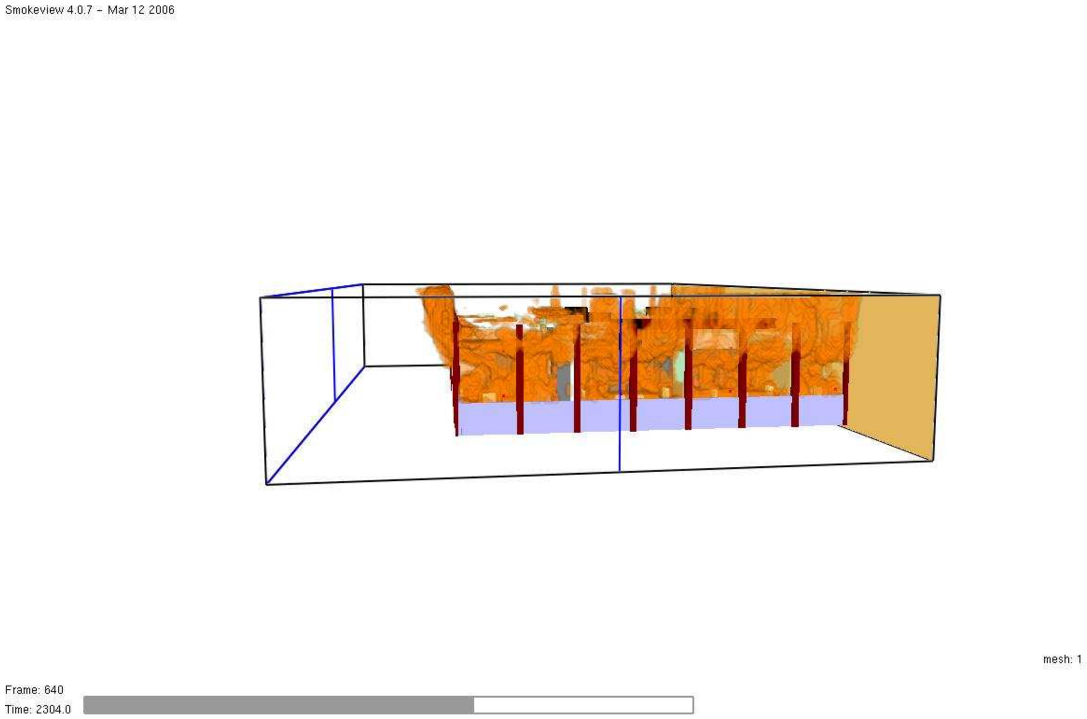

Figure 5.7 External flaming from windows

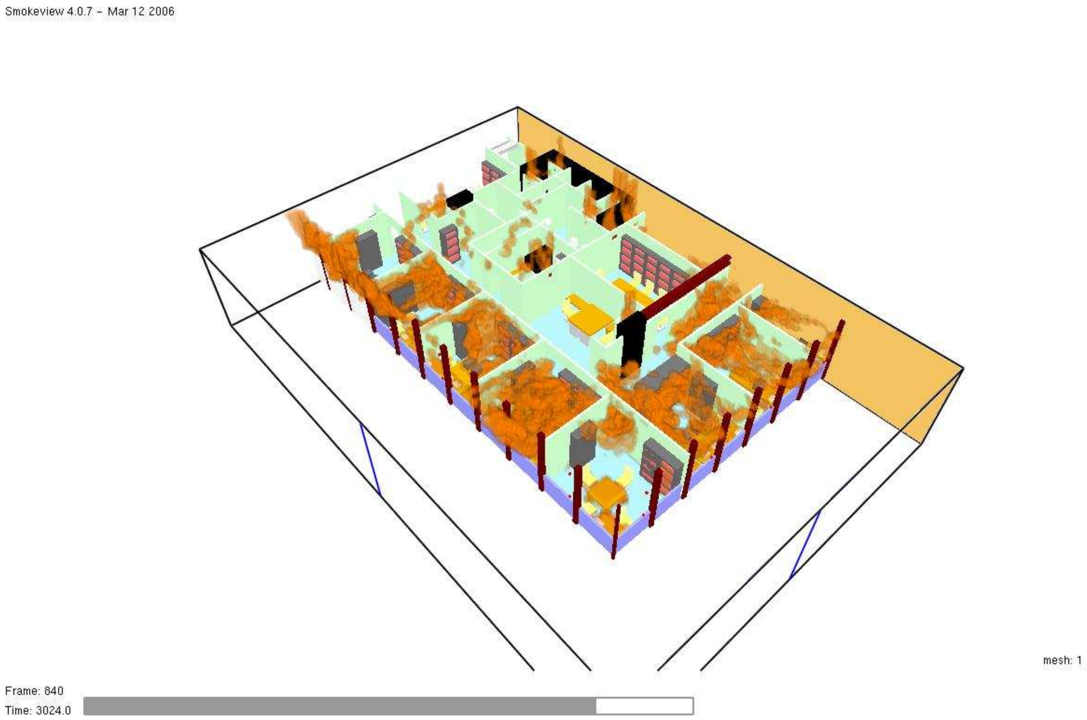

Figure 5.8 Localised extinction

As can be seen from the figures above the initial fire compartment did not remain in a constant and equal state of burning; the fire dies down and then re-ignites on several occasions. This shows that not all of the flammable material within this compartment, or within any of the compartments, has been entirely consumed by the conclusion of the model.

The fire spreads throughout the entire floor quarter during a period of roughly 20 minutes. This coincides with the timeline shown in Figure 3.6 (Kono 2005). It must be remembered that as a mirrored, quarter floor model the FDS model will automatically include multiple points of ignition. This is likely to be the case on most floors as external flames will re-enter the floor at various locations though not necessarily simultaneously. Similarly internal spread through voids could take place in several locations.

The model also demonstrates that it is unlikely that any specific structural member will be exposed to a consistently high temperature over the entire duration of the fire, as the extent of the fire fluctuates within the floor. The peak temperature generally only acts on any given area for a duration of 10 minutes.

Temperature cross sections through the fire show that the peak temperatures attained are in the region of 1000˚C. Again this coincides with observations of concrete conditions from the INTEMAC report which noted that temperatures in excess of 800˚C were reached (Calavera 2005).

Figure 5.9 shows the Heat Release Rate (HRR) figures for the entire floor and demonstrates that the HRR fluctuates about a constant value of 60MW after the initial growth phase.

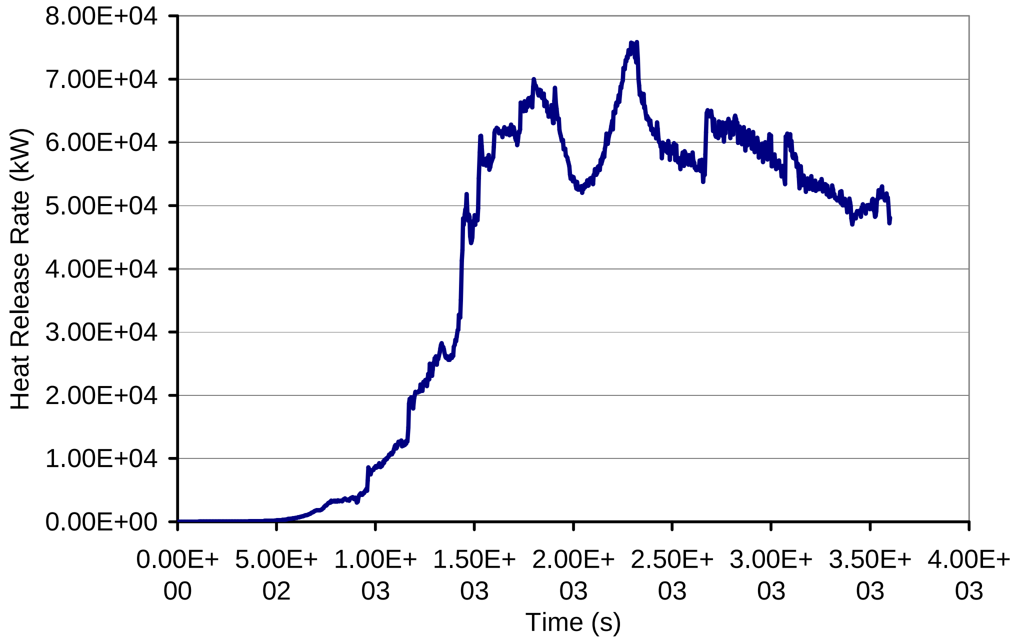

Figure 5.9 FDS Heat Release Rate, whole model

There appears to be some degree of decrease towards the end of the model. However localized and possibly temporary extinction may account for this while other areas of the fire are still in a growth. Therefore it cannot be reasonably assumed that after one hour the fire over the whole floor is entering a decay phase.

As the quantity of fuel present in the floor will vary greatly from floor to floor, and the 'burnout times' of each of the items has been decided empirically or from standard literature (Pons 2007), it becomes clear that using the FDS model to confirm the decay phase of the fire will be difficult. Thus the CFD model will be used to correctly confirm the growth phase of the model and the peak temperatures, with documentary observation being of more use to determine the onset of decay.

5.2.2 Fire Curve

As discussed in Chapter 4 a more simplified but still credible fire could be specified and then be compared against the results of the CFD model. This curve would be used for the initial structural modelling while the CFD analysis completes, and potentially afterward if it is shown to have reasonable agreement with the CFD results.

The simplified model used is based on the ISO fire curve, with a growth phase duration of 1 hour. This matches the approximate time that the fire was reported to be fully developed on each floor in the timeline in Figure 3.6 (Kono 2005).

A major problem of the ISO curve is its lack of a cooling phase. It was decided to introduce 1 hour of cooling to take into account not only the time of the fire undergoing extinction, but also the heat imposed by the reradiation within a compartment once the fire has been extinguished. This cooling phase is a linear decay.

A spreadsheet was developed in order to automate the temperature generation for the fire curve, with the temperatures being defined at five minute intervals. As the initial growth phase of the ISO curve is steep, the temperatures in the initial five minutes are defined at one minute intervals. A typical fire definition curve is shown in Figure 5.10. Note that Abaqus, the Finite Element package used for heat transfer analysis, interpolates between these temperature/time points.

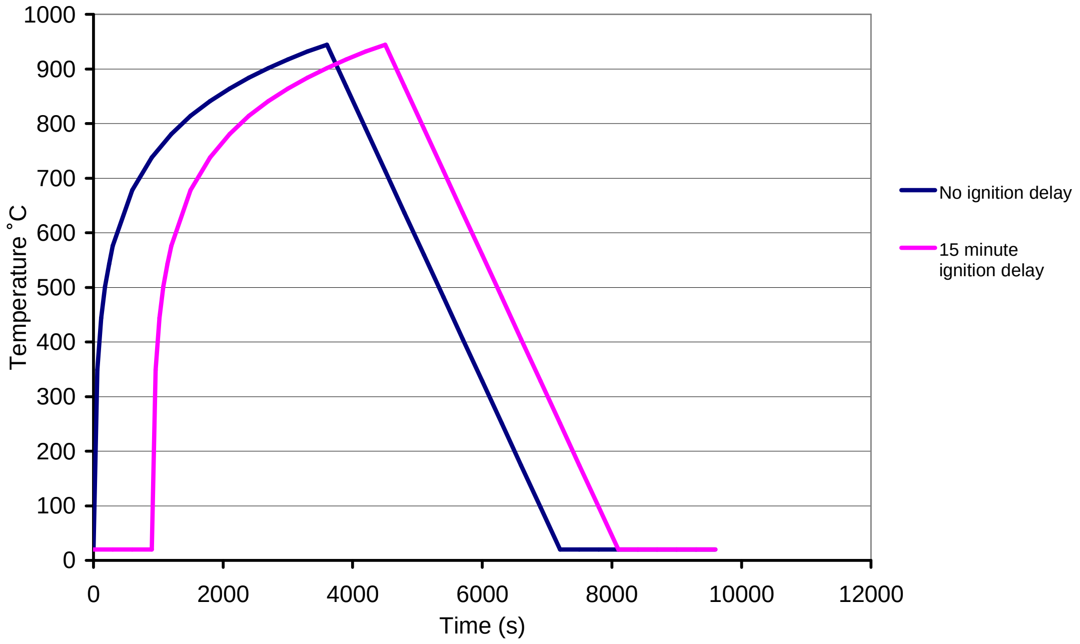

Figure 5.10 Two ISO based fire curves with varying ignition times

It is possible to vary this curve, in order to change variables including:

- time of ignition
- duration of the fire
- duration of the cooling phase
- maximum temperature attained.

For example, a fire can be defined with a temperature rise 110% of that reached in the normal ISO curve. The duration of the fire can likewise be altered, so that the fire can have a longer or shorter duration in such a way as to 'stretch' the curve. Examples of this are shown in Figure 5.11 and Figure 5.12

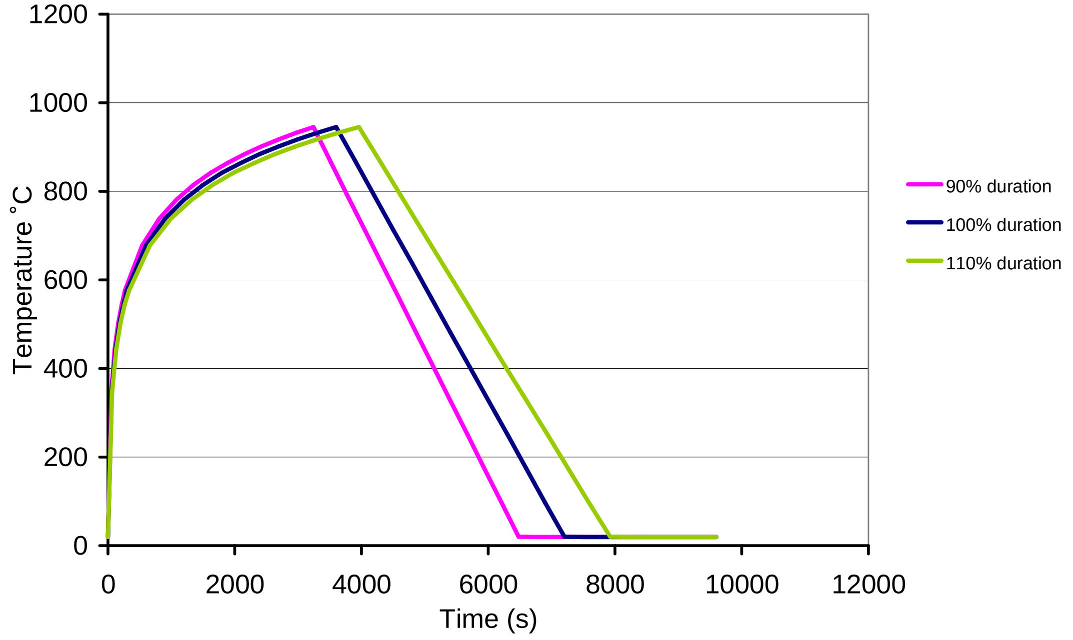

Figure 5.11 Variation in fire curve duration

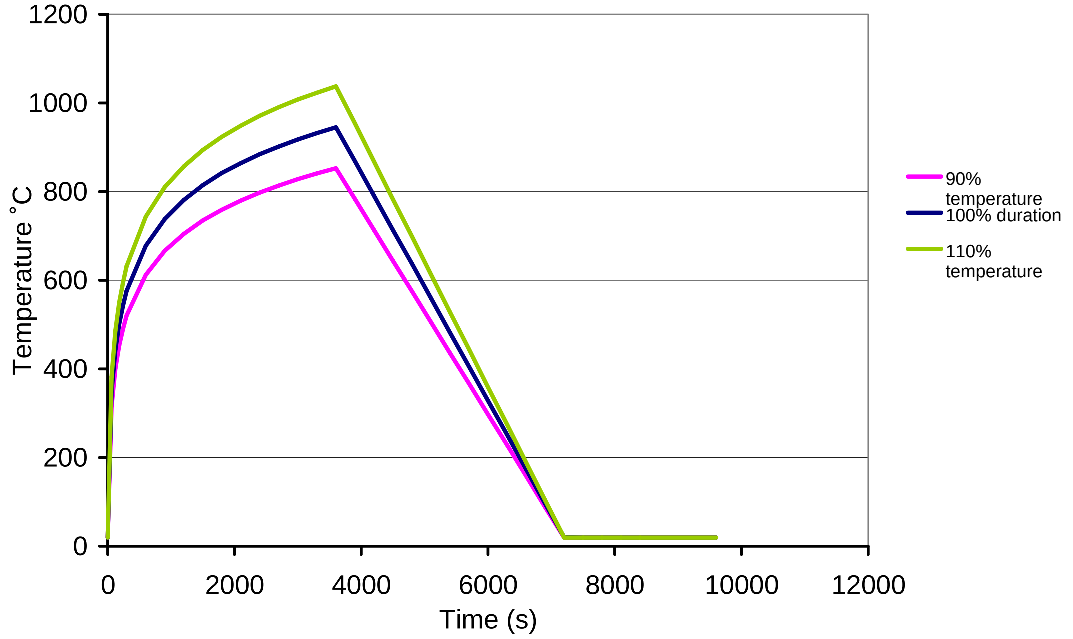

Figure 5.12 Variation in fire curve temperatures

5.2.3 Comparing the fire specification methods

The results of the CFD analysis were compared with the values produced by the developed fire curve. Three areas are examined – the room of initial fire outbreak, the corridor outside and the room next door. The average temperature of the hot layer of each compartment is estimated – as can be seen in Figure 5.13 to Figure 5.18 there is a large degree of spatial variation.

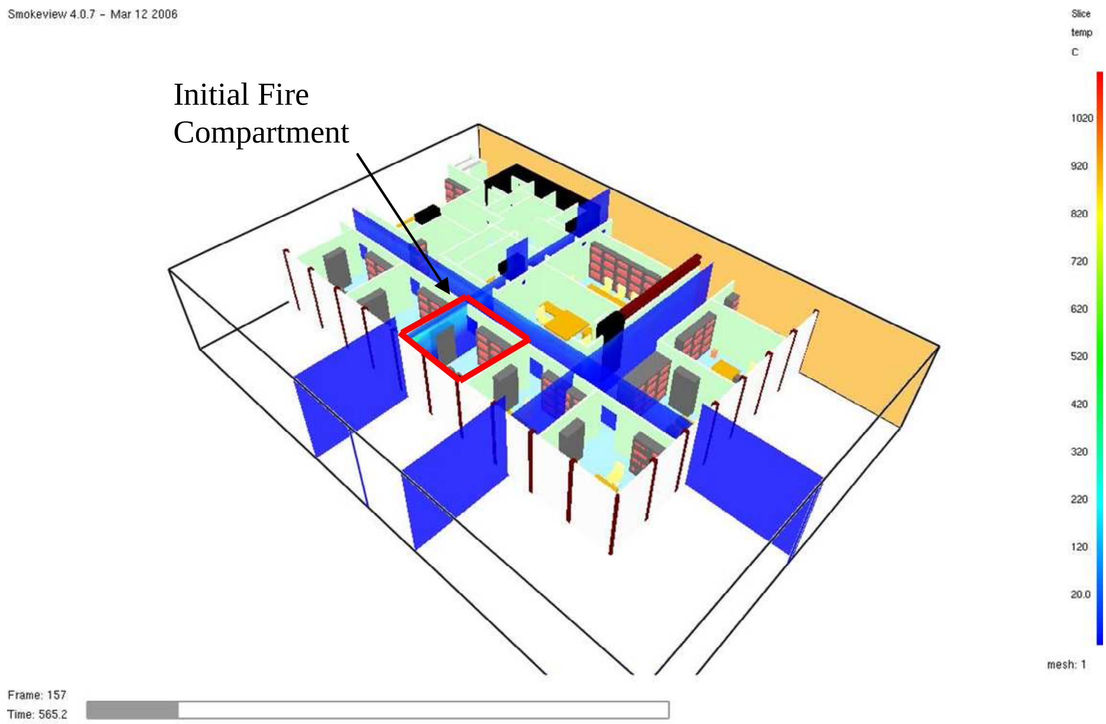

Figure 5.13 Elevated temperatures in initial fire compartment

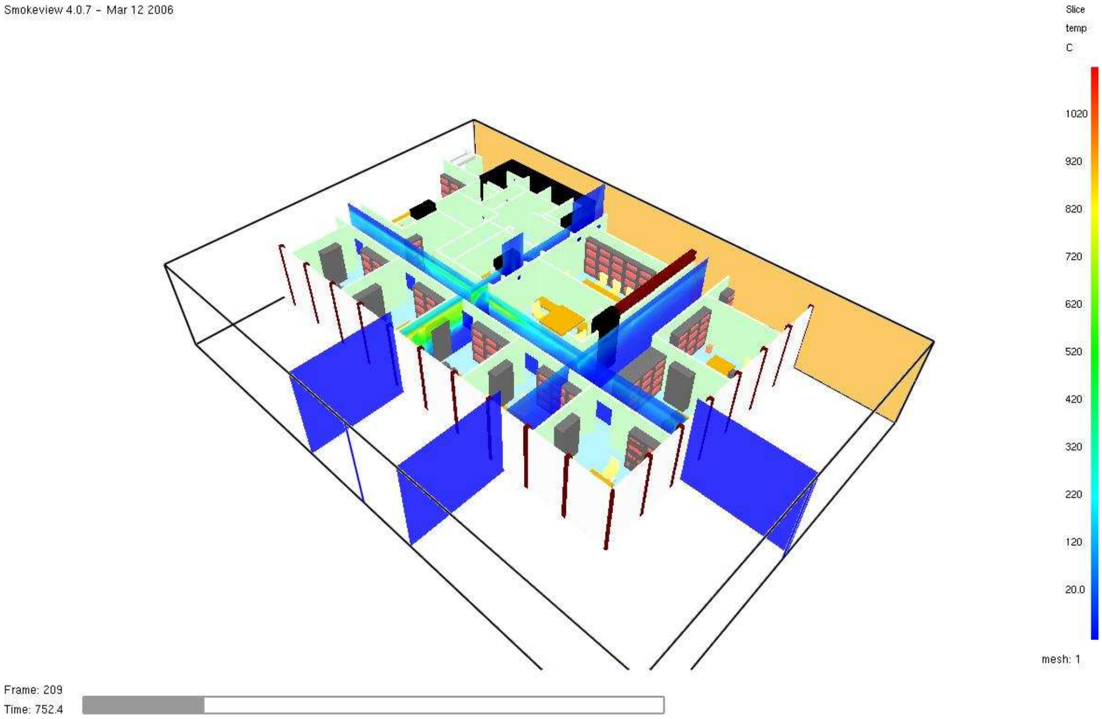

Figure 5.14 Elevated temperatures in hall

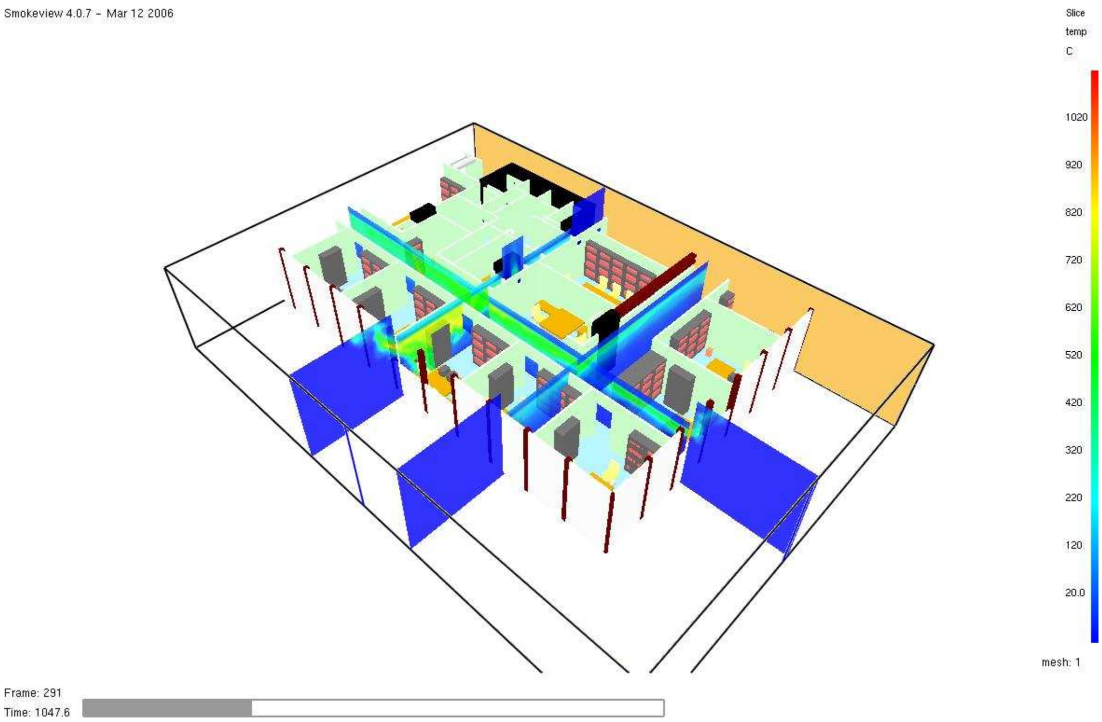

Figure 5.15 Elevated temperatures in room next door to initial fire compartment

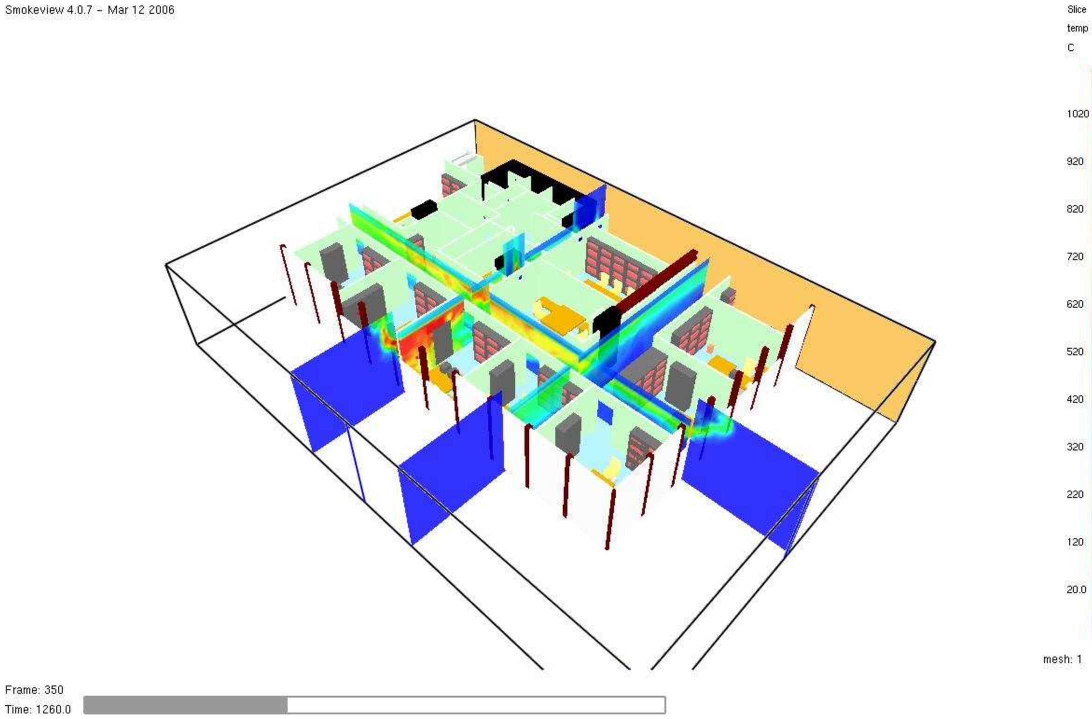

Figure 5.16 Increasing temperatures

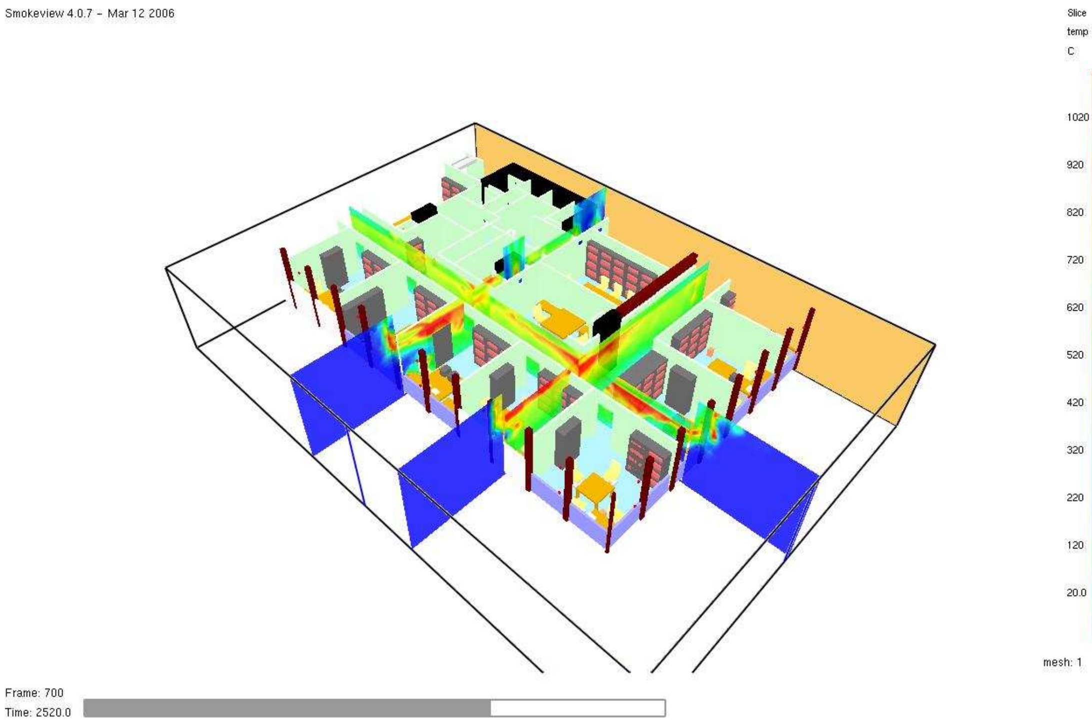

Figure 5.17 High temperatures throughout

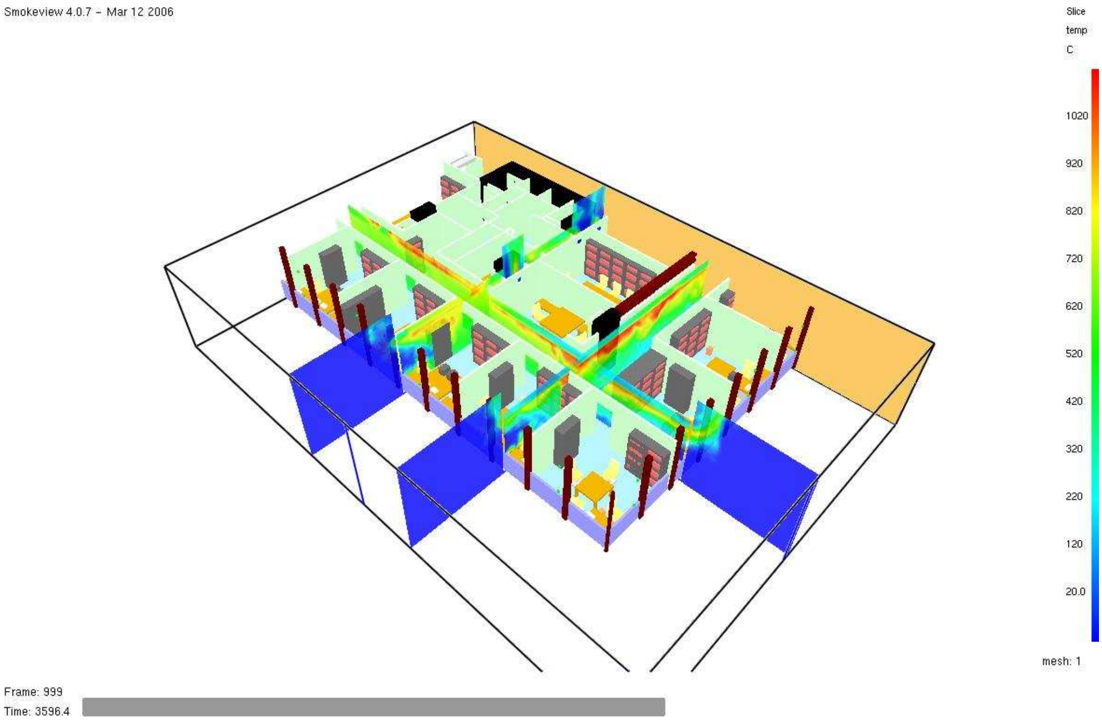

Figure 5.18 Localised fluctuations and reductions in temperature

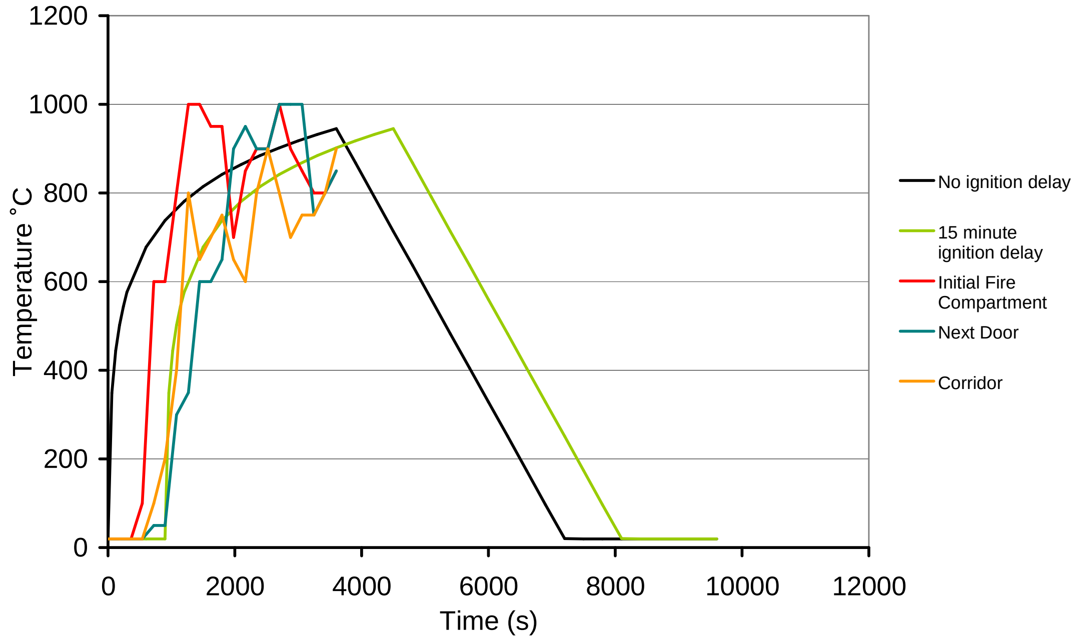

Figure 5.19 FDS temperatures vs. ISO-based curve temperatures

As can be seen in Figure 5.19, the peak temperatures are similar to the values of the ISO based curve and reach sustained temperatures which are similar to those exhibited by the middle portion of the ISO based curve. Note that the ISO based curve is duplicated with a 15 minute delay and coincides well with temperature increases in the corridor and the office next door. It should be noted that the FDS runs ended at 3600s, and hence the results in Figure 5.19 stop at this point.

The CFD method produces highly specific results and fire temperatures at many points, necessitating a very narrow choice of one specific member and the application of a specific fire to it. As has been discussed in Chapter 4, it is possible to automatically couple CFD output with structural analysis using a variety of packages such as ANSYS/CFX. (Liang and Welch 2007; Welch, Miles et al. 2008)

However, this specific application reduces the ability to generalise the model to other similar buildings, different areas of the same building or different fires within the same building. It can also create a false impression that the fire, and the exposure of the structure to the fire, can be characterised to a high degree of accuracy and exactitude. Given the number of assumptions which must be made to develop a fire model, this is unlikely. (Rein 2007a)

Therefore, given the comparison of the CFD results and the values produced by the ISO-based curve, it is reasonable to use the ISO-based curve and variations of it for all structural analysis.

5.3 Structural Modelling

A logical and systematic method of creating a structural model of the Windsor Tower was developed and used. This method consisted of 3 phases in order to determine the 'worst credible case' for structural modelling.

- Determination of Properties
  - Material Properties
  - Thermal Properties
- Development of simplified models
  - Beam model
  - Floor model
- Sensitivity analysis by testing modelling variations of :
  - Fire spread
  - Moisture content
  - Concrete type

5.3.1 Determination of Material & Thermal Properties

Construction drawings were used to determine basic structural properties and material characteristics associated with the Windsor Tower. However, these plans did not specify certain concrete properties such as the tensile capacity of concrete or type of aggregate used. This is significant as concrete with different aggregates will behave differently at raised temperatures.

While the type of aggregate was not specified in the original construction drawings, there is an indication the Windsor Tower may have been constructed using Siliceous concrete due to its post-fire performance (Calavera 2005).

Siliceous concrete is significantly weaker at high temperatures than Calcareous concrete. In order to prevent the aggregate type from having an overly large influence on the other sensitivity analyses, it was decided that the stronger Calcareous aggregate would be used for the majority of the sensitivity analyses.

The temperature dependant properties of the concrete and steel reinforcement were defined using the guidance from (Eurocode 2 2003) and (Eurocode 4 2003). Both the thermal and physical properties were varied in accordance with the Eurocode recommendations. The final values used for the concrete and steel properties are given in Appendix A.

Note that though many physical properties do not restore on cooling, the central core of the concrete is unlikely to reach high enough temperatures for this to have a major effect. Therefore the physical properties will be based on the current temperature rather than the maximum temperature attained.

Transient creep strain, as mentioned in Section 2.5, may make the Eurocode properties unconservative. However as the principal research topic of this thesis is the study of structural interaction with fire rather than evaluation of different material models, the Eurocode values will be used as they are widely available.

The concrete will be modelled using the Concrete Damaged Plasticity (CDP) model provided in Abaqus. This provides a yield surface based upon the work of Lubliner and Lee and Fenves (Lubliner, Oliver et al. 1989; Lee and Fenves 1998) and can be used to model the compressive and tensile behaviour of concrete separately, including the post-yield behaviour (Abaqus 6.8-1 Manual 2008) (Abaqus 6.8-1 Theory Manual Section 4.5.2). CDP is also able to model the degradation of elastic stiffness, i.e. damage, to concrete although this capability has not been used in the models discussed here.

5.3.2 Development of the 'beam model'

The first model developed for the sensitivity analyses is a simplified model of the floor slab, simply supported rather than using realistic support conditions. This model will be used to carry out the majority of the 'sensitivity study' to find the worst credible combination of material strength, rate of fire spread and other variables.

In order to simplify the structural modelling of the floor slab, the section of slab is modelled as a beam (Figure 5.20, Figure 5.21). As the floor slab is constructed as a waffle slab, the simplest method of selecting a suitable beam size is to take one of the ribs of the slab spanning in the desired direction, neglecting any structural capacity of the un-reinforced concrete screed which spans between the concrete beams (i.e. it is not modelled as a T beam). This is a worst case scenario for the sensitivity analysis, as the screed will provide additional compressive strength at the mid-span of the beam.

Figure 5.20 Initial 'Beam model'

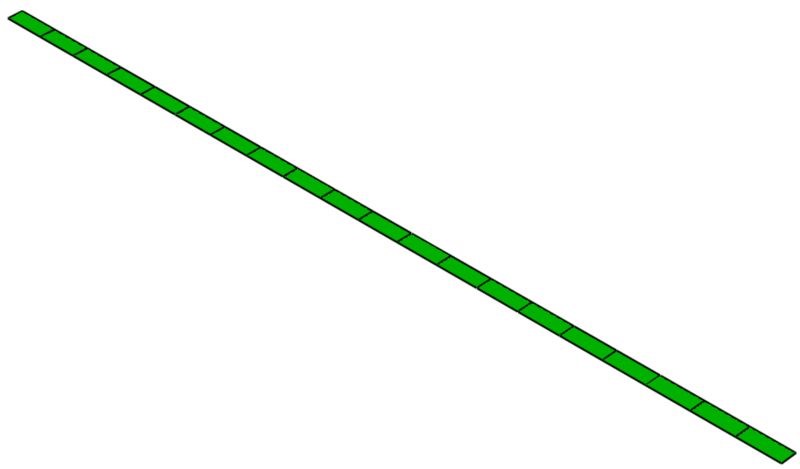

Figure 5.21 Abaqus model of 'beam model'

The dimensions of the beam are available from the construction drawings for the Windsor Tower. All waffle slab ribs are 230mm deep (including the 30mm screed directly on top of the beam, which will be included) and 100mm wide on all floors of the tower as discussed in Chapter 3. In the area for examination selected in Chapter 4, the selected beam spans 5.2m from the concrete core of the building to the exterior steel columns on all floors. It was felt that using simple supports for the sensitivity analysis phase of modelling would be a worst case scenario, as it will increase the moment at mid span. Fixed end supports are used in the later models.

Details of the reinforcement present in the beam were also available from the construction drawing of the Windsor Tower. The top reinforcement consisted of two 8 mm bars, and the bottom reinforcement was two 12 mm bars on all floors. The levels of cover to the reinforcement were derived from these construction drawings and the INTEMAC report (Calavera 2005). While it is possible to be accurate about the depth of cover to the bottom reinforcement, the details of the depth to the top reinforcement are less clear and the INTEMAC report derived them by scaling from the diagram (Calavera 2005). See Figure 3.4 for details of the waffle slab construction.

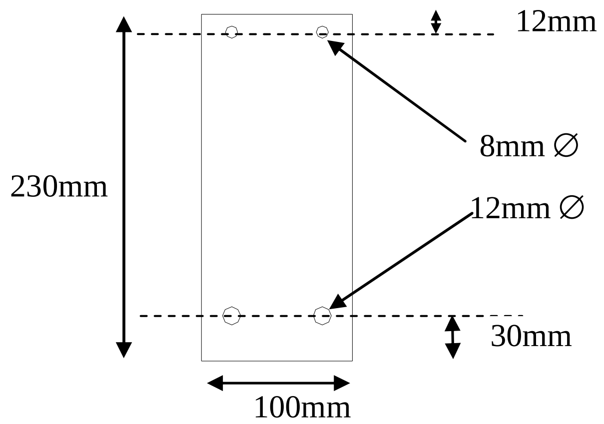

Figure 5.22 Cross section of 'beam model'

The beams being examined run East-West and therefore have the profile shown in diagram Figure 5.22. The method of construction used in the Windsor Tower waffle slabs involved permanent clay formwork around 10 mm thick (Calavera 2005). This will cover both the bottom and sides of the beam.

In order to simplify the thermal model being used, this clay was omitted on the sides of the beam and been defined as part of the concrete at the bottom of the beam. While it will not have the same physical properties, this modelling scenario will result in broadly similar thermal properties to concrete and will therefore provide enhanced insulation to the reinforcement. The effect of the variation in structural capacity provided by this additional "concrete" will be limited in tension though it may have an effect when the clay is in compression.

The strength of both the reinforcement and the concrete are also shown on the construction plans. The steel has a strength of 5000 kg/cm^2 (Modelled as 50N/mm^2) while the concrete has a compressive strength of 175 kg/cm^2 (Modelled as 17.5 N/mm^2). This is a weak concrete mixture. The tensile capacity of the concrete at ambient temperature was unknown, and therefore a value of 1.5N/mm^2 was selected, under 10% of the compressive strength.

The concrete beam was modelled in Abaqus using S4R Shell elements. Shell elements can have up to 19 temperature definition points throughout their depth, and are therefore well suited for a combined thermal/mechanical analysis. Beam elements cannot be used to model concrete with reinforcement in Abaqus Explicit, and therefore would be a significant drawback when modelling of a collapse state is required. The reinforcement is modelled as 'smeared' layers within the shell elements rather discreet rebars. These layers are defined by specifying the size and distribution of the rebars.

Reinforcement Layer	Area/Bar (mm^2)	Spacing (m)	Vertical distance from centre line (m)
Top Rebar	50.3	0.06	0.103
Bottom Rebar	113	0.06	-0.085

Table 5.1 Reinforcement in 'beam model'

The loads applied to the 'beam model' were calculated based upon the building construction drawings and are tabulated below.

Name	Loading	Applied to	Basis of calculation
Gravity	-9.81 m/s^2	Whole model	Automatically calculated by Abaqus
Live Load	25309.8 N/m^2	Top of Beam	430kg/m^2 from construction drawings; Beams at 0.6m centres; Top of beam 0.1m wide
Slab Self Weight	3531.6 N/m^2	Top of Beam	Concrete density 2400 kg/m^3; Slab 30mm thick

Table 5.2 'Beam model' loads

5.3.3 Single 'Floor Model'

The single 'floor model' is a further development of the simply supported model whereby the 'beam' representing the floor lab is attached at one end to the concrete core of the building, with the other being fixed to a steel column above and below the floor slab (Figure 5.23, Figure 5.24). These connections are fixed against rotation. This model is a precursor to producing a multi floor sectional model (Chapter 6).

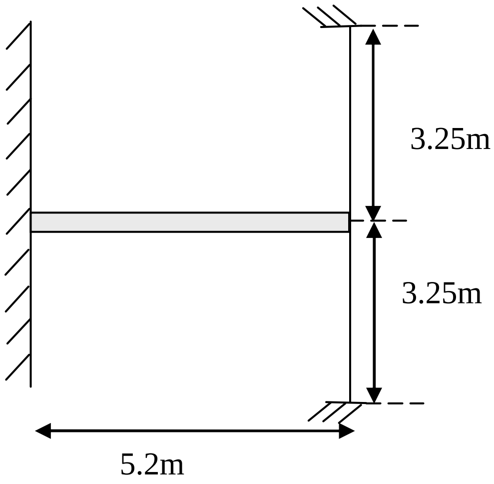

Figure 5.23 Second model 'Floor Model'

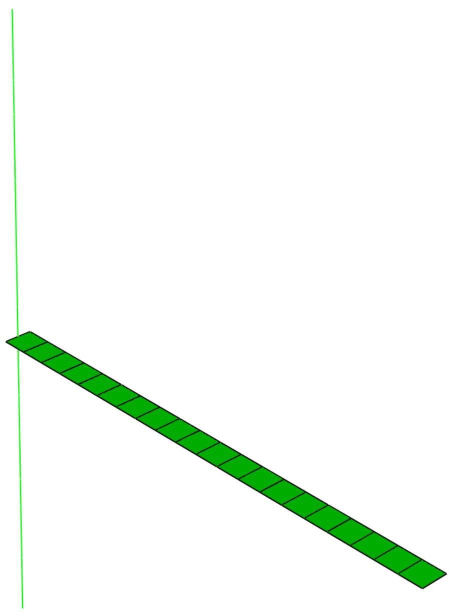

Figure 5.24 - 'Floor Model' in Abaqus

The steel columns on the perimeter of the building are spaced at 1.8m intervals, while the ribs of the waffle slab are spaced at 0.6m intervals, supported at their ends by an edge beam 1m wide (Figure 3.2). Therefore for the loading on the steel columns to be correct, three ‘beams’ must be modelled.

Rather than model the floor slab as a grillage of primary and secondary beams, the three primary beams are joined to form a beam 0.3m wide. The loading is scaled to take into account the additional area acting on the ‘beam’. As this is a 3D representation of a 2D section, and lateral movement is not being considered, this will produce accurate results at a much reduced computational cost. (Figure 5.25)

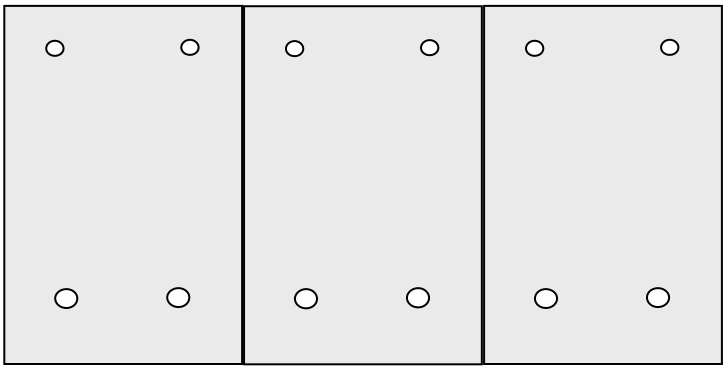

Figure 5.25 Three beams joined together

The structural effect of the edge beam is neglected, though the load it imposes on the steel columns is included, as is the loading from the exterior cladding. The edge beam is 1m wide and 0.23m deep. Note that when expanded to the multiple floor model the edge beam is modelled as a lateral restraint preventing out-of-plane movement and rotation.

Name	Loading	Applied to	Basis of calculation
Gravity	-9.81 m/s^2	Whole model	Automatically calculated by Abaqus
Live Load	25309.8 N/m^2	Top of Beam	430kg/m^2 from construction drawings; Beams at 0.6m centres; Top of beam 0.3m wide
Slab Self Weight	3384.45 N/m^2	Top of Beam	Concrete density 2300 kg/m^3; Slab 30mm thick
Secondary beams	3924 N/m^2	Top of Beam	Reinforced Concrete density 2400 kg/m^3; Beams 0.1m x 0.2m x 0.5m; 0.6m centres
Façade Load	3531.6 N	Join of beam and column	200kg/m from construction drawings; 1.8m long
Edge Beam	8475.84 N	Join of beam and column	Reinforced Concrete density 2400 kg/m^3; Beam 1m x 0.2m x 1.8m
Axial column load	3196333 N	Join of beam and column	Load from floors and columns above floor 21.

Table 5.3 Load on ‘Floor Model’

Note that in Table 5.3, the Edge Beam and Secondary Beams are 0.23m deep, but the load of the top 0.03m is already included in the Slab Self Weight.

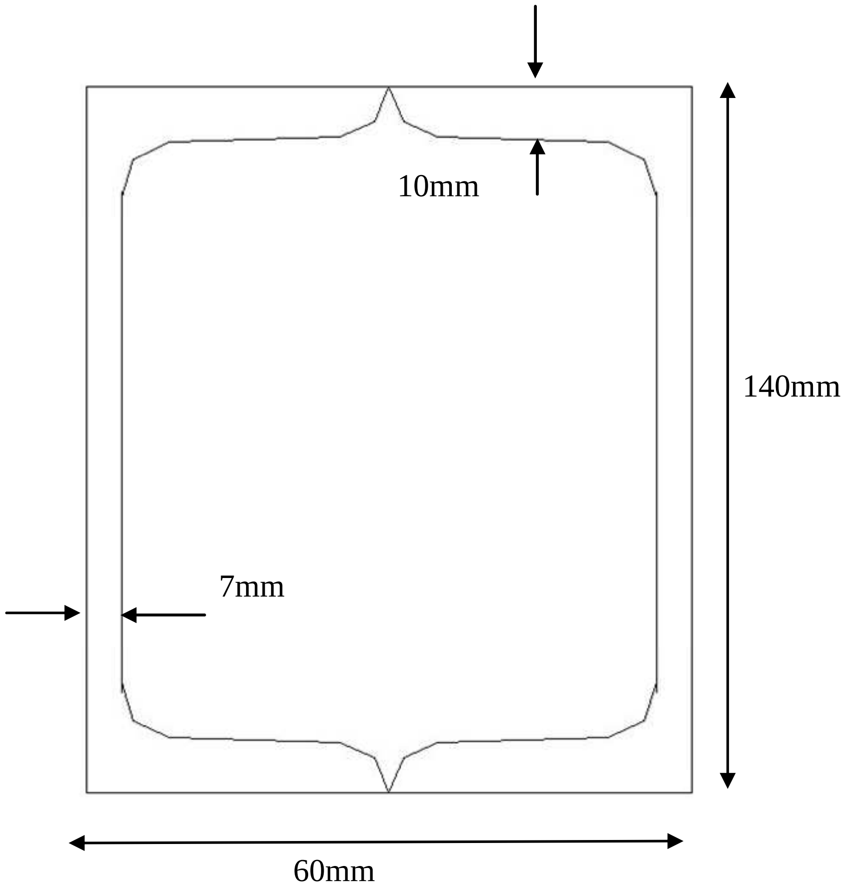

Figure 5.26 Column cross section for 'Floor model' fabricated from Two UPN 140 sections

The cross section of the exterior steel columns varies from floor to floor, so a section was chosen to represent the column above and below Floor 21, the floor of initial fire outbreak. The columns are fabricated from a pair of channel sections connected at the tips of their flanges (Figure 5.26). The channel sections were not welded together over their full height (Kono 2005) though details of what proportion of the length was welded are unclear. As can be seen from the post fire photographs (Figure 3.7) some of these sections appear to have become separated when subjected to heating though Figure 3.8 and Figure 3.9 may indicate that the separation was between the structural steel columns and the façade steelwork.

Any splitting of the steel columns will have raised the temperature of the individual channel sections further than would be the case for the larger composite section and reduced their overall buckling capacity.

Calculation shows that an equivalent rectangular hollow section has the same cross sectional area and second moment of area about the major axis, which can then be used easily in Abaqus. The second moment of area was not considered about the minor axis. See Figure 5.27 and Table 5.4

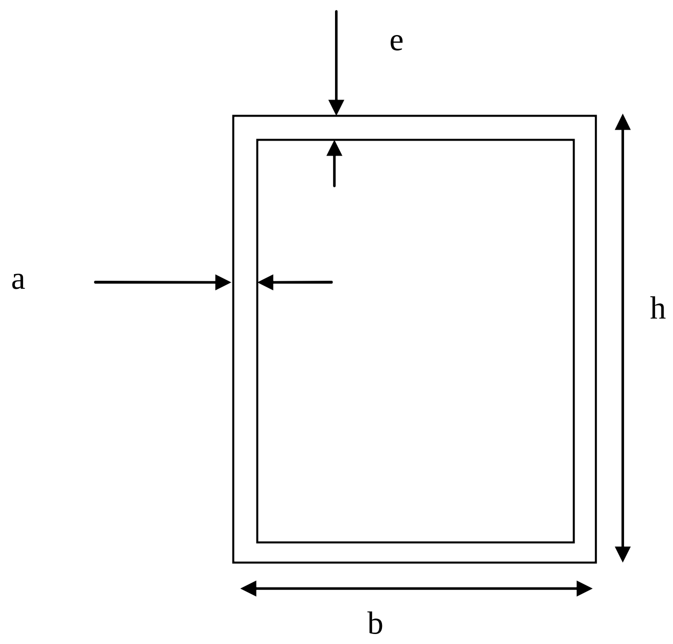

Figure 5.27 Cross section of equivalent column box section

Section (x2)	Area (mm^2)	I_{xx} (mm^4)	Equivalent box section (mm)	Area (mm^2)	I_{xx} (mm^4)
UPN 100	2700	4,120,000	b=100 h=100 a=6 e=8.5	2696	4,140,229
UPN 120	3400	7,280,000	b=120 h=110 a=7 e=9	3408	7,350,336
UPN 140	4080	12,100,00	b=140 h=120 a=7 e=10	4080	12,176,000
UPN 160	4800	18,500,000	b=160 h=130 a=7.5 e=10.5	4815	18,636,151

Table 5.4 Properties of double UPN sections and equivalent box sections used in model

In the case of these single floor models, the steel columns are assumed to be perfectly fire protected, and do not increase in temperature throughout the duration of the model.

5.3.4 Heat Transfer

When conducting a heat transfer analysis it is necessary to characterise the temperature to which the steel reinforcement is subjected. It was assumed that, since the concrete had equal or greater levels of cover from the side of the beam to the reinforcement, in addition to the insulating effects of the clay formwork to the side, a 1D heat transfer would be sufficiently detailed. As can be seen from Figure 5.28, the temperature to which the reinforcement would be exposed is approximately equivalent.

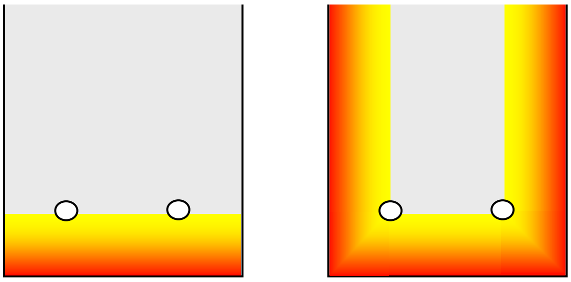

Figure 5.28 1-D vs 2-D heat transfer

In order for a heat transfer to work correctly, the nodal points of a member must be identical in both the thermal and mechanical models. This is achieved simply by copying the model from one type of analysis to the other, and changing the element and step types used. Type DS4 shell elements were used for the heat transfer, which are the equivalent to the type S4R elements used in the stress analysis. These have an equal number of temperature points throughout the depth of the element.

As the beam is to be modelled using Abaqus' library of shell elements, it will be impossible to transfer heat in from the edge of the beam. When using Abaqus, a full 3D heat transfer analysis would require Continuum or 'block' elements. While block elements exist which are capable of modelling a mechanical analysis involving bending accurately, it is computationally cheaper to use Shell elements.

External temperatures were used for the heat transfer analyses rather than external heat fluxes, as these results were easier to obtain from the CFD analysis and the ISO-based curve. The temperature was defined as both a radiative and convective film on the top and bottom surface of the shell elements.

5.4 Sensitivity Analysis

Having established simplified structural models, denoted as the ‘beam model’ and the ‘floor model’, and established the method of heat transfer analysis to be used, sensitivity analysis can be performed.

The sensitivity analysis establishes the ‘worst credible case’ of variables that can be applied to the more detailed structural models of the Windsor Tower.

The ‘beam model’ was used to examine:

- Variation in fire spread rate
- Variation in moisture content
- Variation in fire duration
- Variation in fire temperatures
- Mesh sensitivity

The “floor model” was used to examine

- Variation in concrete type
- Variation in concrete tensile definition

Both models were also used to investigate the effects of change of Concrete Damaged Plasticity parameters and Thermal Expansion characteristics.

5.4.1 Initial Beam Model

The first structural model used no heat transfer at all, and simply specified the temperature throughout the entire member. This is broadly similar to heating a totally unloaded member in an oven to the desired temperature, and then applying load to it.

This is a “proof of concept” model, and uses higher strength (30N/mm^2) Siliceous concrete and steel that loses plastic strength at a lower rate than for subsequent models. Expansion is neglected.

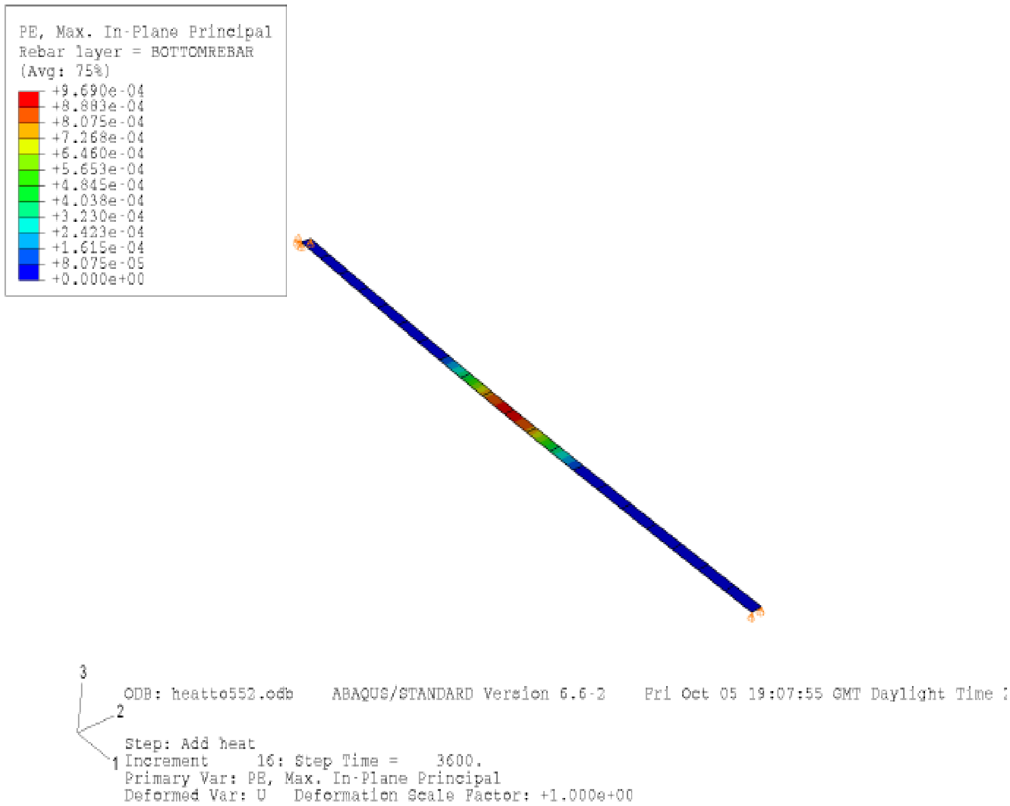

Figure 5.29 Final state of proof of concept model prior to failure

The ‘constant temperature’ involved increasing the temperature linearly throughout the entire stress model and demonstrated structural stability until the member reached a temperature of approximately 552˚C (Figure 5.29).

This is approximately the temperature at which steel and concrete are regarded to lose the majority of their strength, so it is unsurprising that the member appears to fail (or not converge due to severe discontinuities in the material behaviour) at this temperature. Further checks of the validity of the model can be made by simple hand calculations, for example for the deflection. The final vertical mid-point deflection of the beam is 0.036m. A very simple calculation using the well know formula:

Deflection = $5/384x(WL^4)/EI$

Where I is the second moment of area of the section and E is the modulus of elasticity of the concrete gives a deflection of 0.02m. While this is a lower deflection than that produced by the Abaqus analysis, it is based on the steel and concrete still behaving elastically at the end of the analysis rather than having yielded. As the deflection is of the correct order of magnitude this is regarded as acceptable.

This proof of concept demonstrates that a heated reinforced concrete member can be successfully modelled using the Abaqus application, and produces results in keeping with published values.

5.4.2 Variation in Fire Spread Rate

One of the critical points of interest in the Windsor Tower fire is that the fire spread between floors, both upwards and downwards. The first set of sensitivity analyses were designed to examine:

- The critical fire spread rate
- The impact of upward or/ downward spread

Thermal models were created with the ISO-based fire curve applied with differing time delays to the top and bottom surfaces of the beam. A case was also specified where the fire was applied only to the bottom surface of the beam, rather than both the top and bottom. In all but one case upward spread was assumed.

The 100% temperature (Figure 5.12), 100% duration (Figure 5.11) ISO-based fire was applied to the top of the beam as well as the underside as this is likely to be more severe, ignoring any insulation effect of the carpet and the generally lower air temperatures in the lower areas of the fire compartment. The main temperature driver for the top of the beam is the radiation from the ‘hot layer’ at the top of the compartment rather than local air temperatures, and therefore the temperatures are likely to be equivalent. In the case of large quantities of debris having fallen on top of the floor slab, the smouldering of flammable materials is also likely to impose large temperatures to the top of the slab

All of the heat transfer models used Abaqus Standard with a duration of 100 hours, to allow the concrete at the core of the member to cool to ambient temperature. The models are referred to by the difference in time between the fire being applied to the bottom and top surfaces of the 'beam model'. A "15 minute delay upward spread" model would apply the fire to the top surface 15 minutes after the bottom surface, simulating the fire taking 15 minutes to travel from one floor to the floor above.

Name	ISO-Based temperature applied to bottom at time (min)	ISO-Based temperature applied to top at time (min)	Input file names (Heat Transfer/Stress Analysis)
-30	30	0	HeatTB100Lngneg30.inp TBStress100Delayneg30NoStabv2.inp
0	0	0	HeatTB100Lng0.inp TBStress100Delay0NoStabv2.inp
15	0	15	HeatTB100Lng15.inp TBStress100Delay15NoStab.inp
30	0	30	HeatTB100Lng30.inp TBStress100Delay30NoStab.inp
35	0	35	HeatTB100Lng35.inp TBStress100Delay35NoStab.inp
45	0	45	HeatTB100Lng45.inp TBStress100Delay45NoStab.inp
50	0	50	HeatTB100Lng50.inp TBStress100Delay50NoStab.inp
55	0	55	HeatTB100Lng55.inp TBStress100Delay55NoStab.inp
60	0	60	HeatTB100Lng60.inp TBStress100Delay60NoStabv2.inp
Bottom	0	N/A	HeatB100Lng.inp BStress100v2.inp
15v68	0	15	HeatTB100Delay15v68.inp TBStress100Delay15v68.inp

Table 5.5 Heating conditions

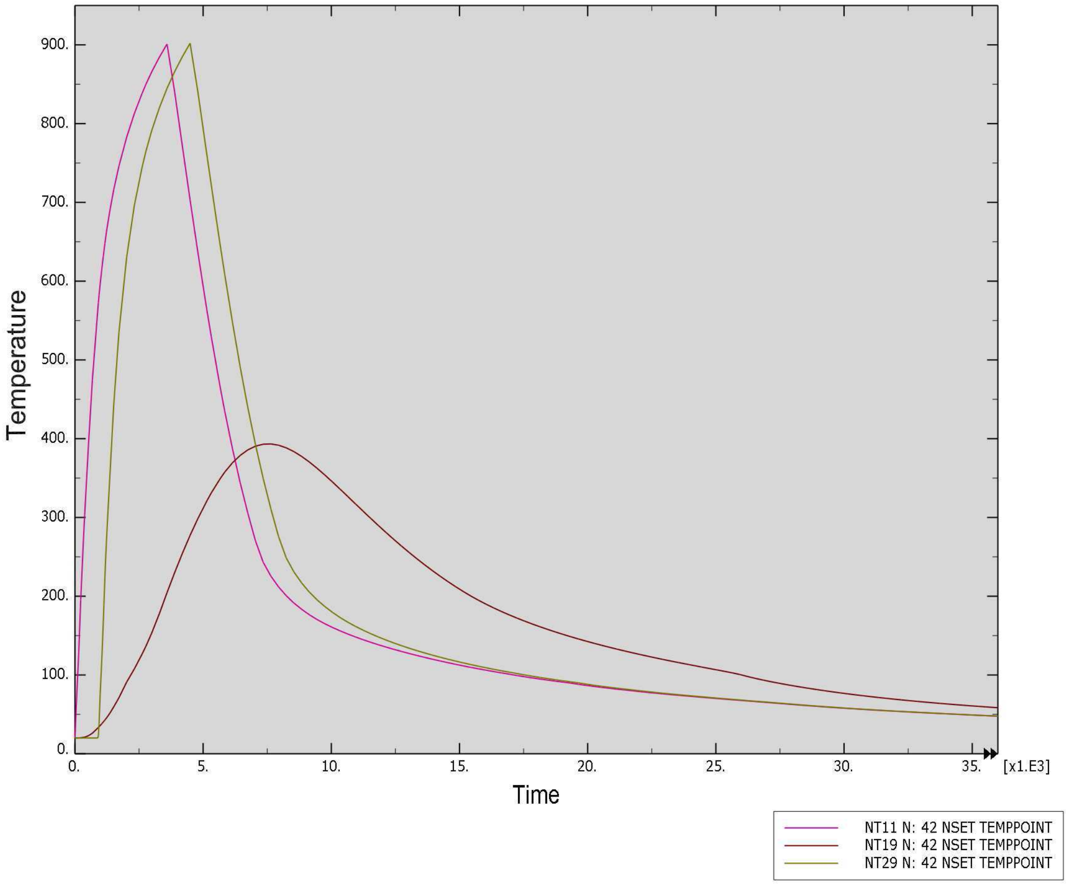

Figure 5.30 Example Heat Transfer output for 15 minute delay upward spread. NT11=Bottom surface, NT19=Middle of section, NT29=Top surface.

The outputs from the heat transfer models (Figure 5.30) were imported into the simplified 'beam model' again in Abaqus Standard. The stress model and temperature model must have an equal duration when using Abaqus Standard, otherwise the heat transfer results will be 'compressed or 'stretched' to fit the stress model duration. This can lead to unexpected outputs in Abaqus Standard.

The results from the stress models demonstrated that in the case of all scenarios, the concrete beam underwent severe plastic deformation, with large plastic strains in both the bottom and top reinforcement (Figure 5.31, Figure 5.32)

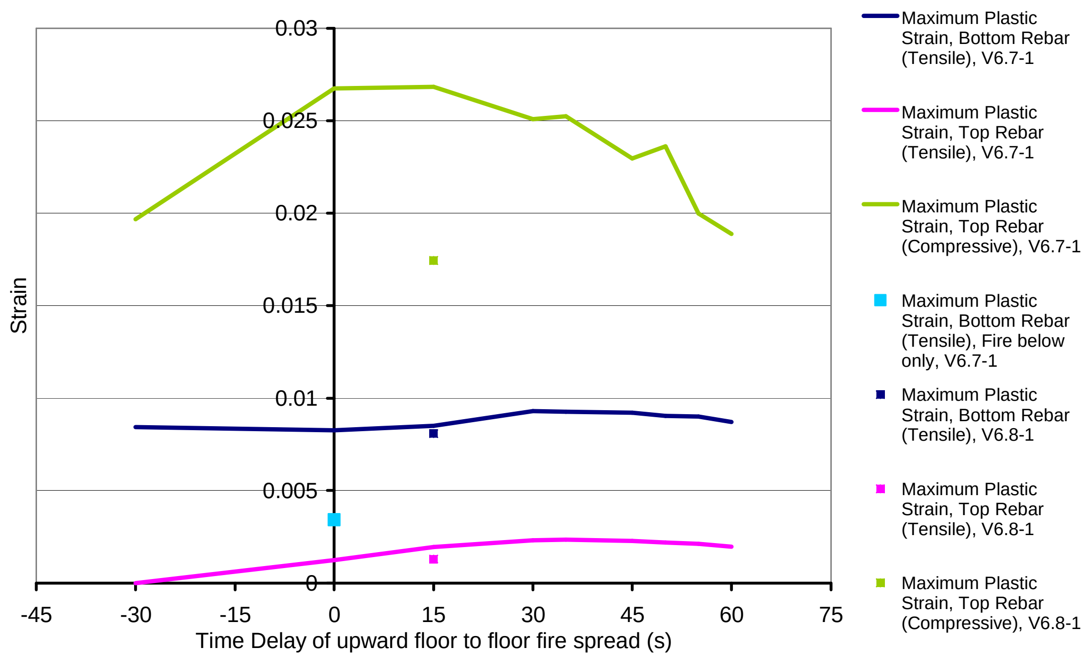

Figure 5.31 Time delay analysis, Plastic reinforcement strains. Note that Compressive strain in the bottom rebar is always zero.

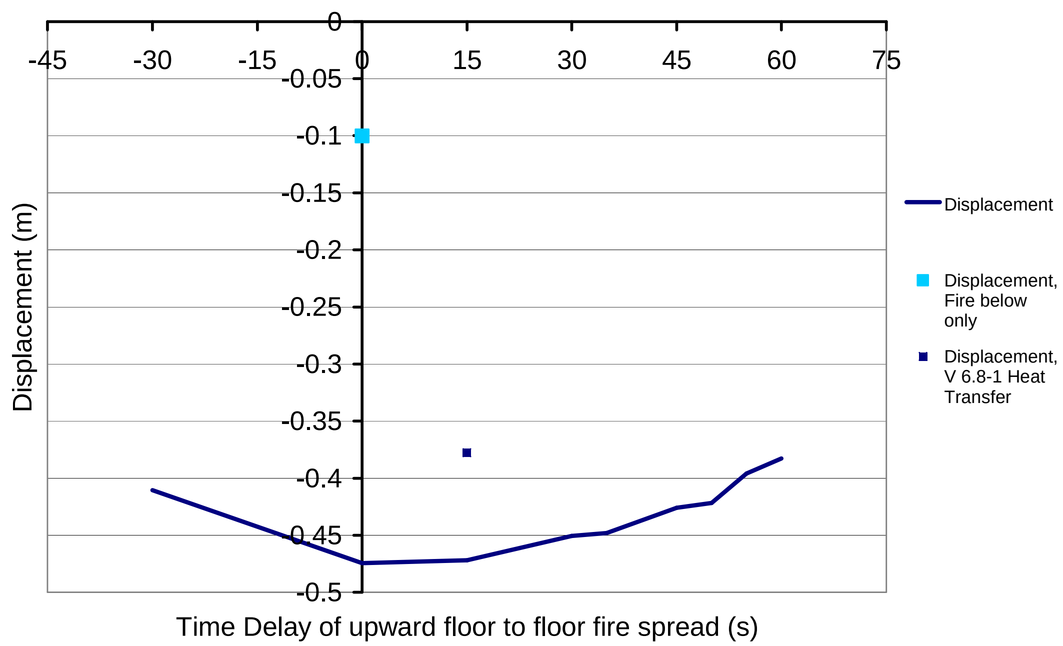

Figure 5.32 Time delay analysis, vertical displacement.

From these results, it can be seen that there is a peak in deflections when the delay between the bottom and top fires was in the range 0-15 minutes, and that the peak plastic strains in the reinforcement occurred in the range 15 to 30 minutes.

During this phase of the research a computational error in Abaqus v 6.7-1 was identified. This error related to heat transfer and resulted in higher than expected temperatures within the member. The modelling for fire spread rate had been completed by this stage. As the error produced a more severe condition, the models were not re-run except the most severe case, 15 minute upward spread. This was re-run with a temperature analysis produced by Abaqus v6.8-1, and the results are shown for comparison in Figure 5.31 and Figure 5.32. The new heat transfer results are shown in Figure 5.33 for comparison with Figure 5.30.

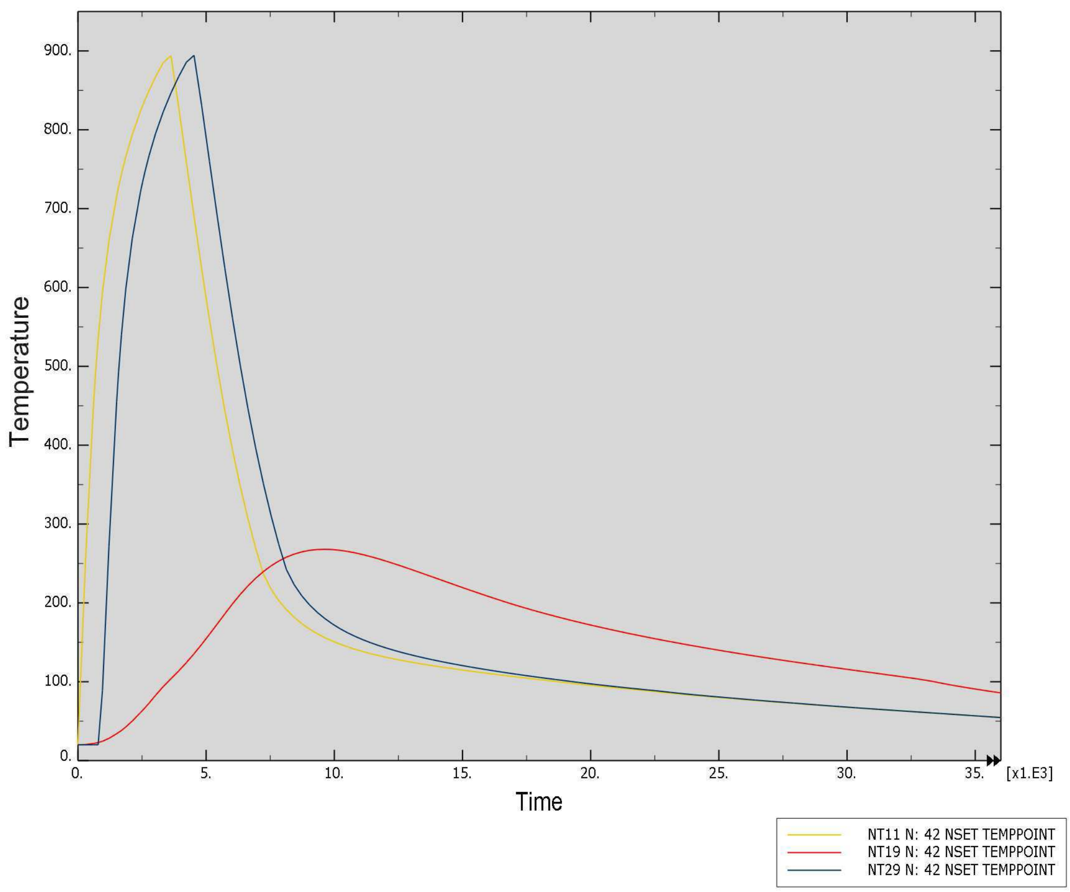

Figure 5.33 Heat Transfer output for 15 minute delay upward spread using Abaqus V6.8-1. NT11=Bottom surface, NT19=Middle of section, NT29=Top surface.

While the temperature analysis was incorrect in Abaqus v6.7-1, the stress analysis worked correctly in both versions.

In conclusion, the 15 minute upward spread case can be considered the most severe and will be used for all subsequent sensitivity analysis models.

5.4.3 Variation in Moisture Content

The initial 'basic' moisture content is 1.5%, the 'mid range' value provided in the Eurocode concrete properties (Eurocode 2 2003). In order to check that variations in moisture content would not have a disproportionately high effect on the structure, the model was also run with 0% moisture and 3% moisture, with a 15 minute delay in the upward fire spread. The moisture content affects the effective Specific Heat Capacity of the concrete via an additional latent heat. Note that the conductivity of the concrete is set to the upper bound suggested in the Eurocode.

Moisture Content	Input file names (Heat Transfer/Stress Analysis)
0%	HeatTB100LngDryv68.inp TBStress100Delay15dryv68.inp
1.5%	HeatTB100Delay15v68.inp TBStress100Delay15v68.inp
3%	HeatTB100Lng3moistv68.inp TBStress100Delay15moist3v68.inp

Table 5.6 - Moisture contents to be analysed

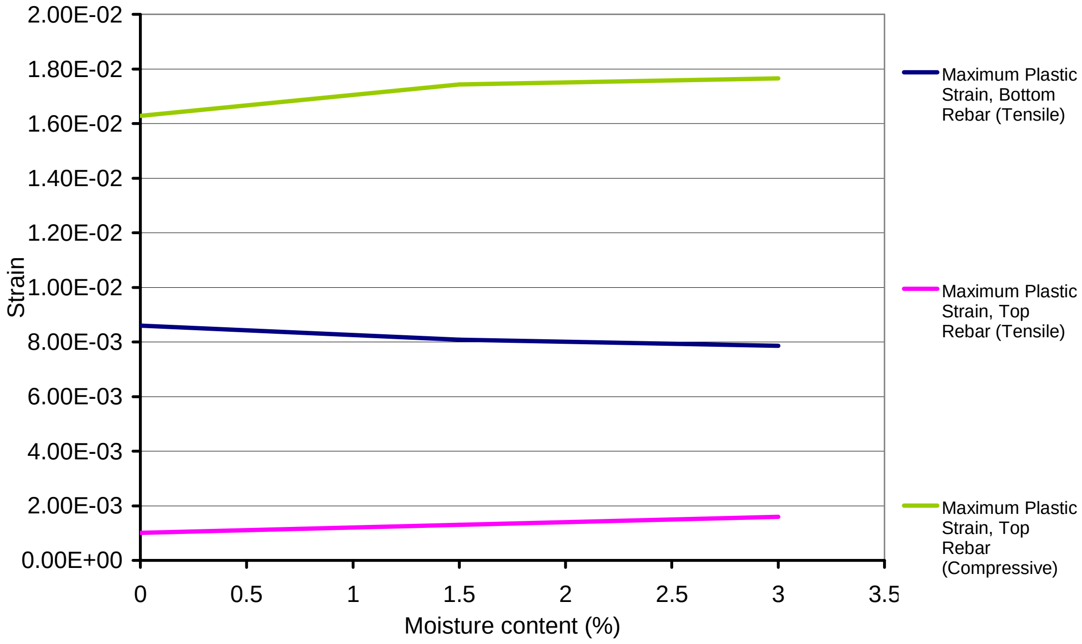

Figure 5.34 Moisture content analysis, Plastic reinforcement strains. Note that Compressive strain in the bottom rebar is always zero.

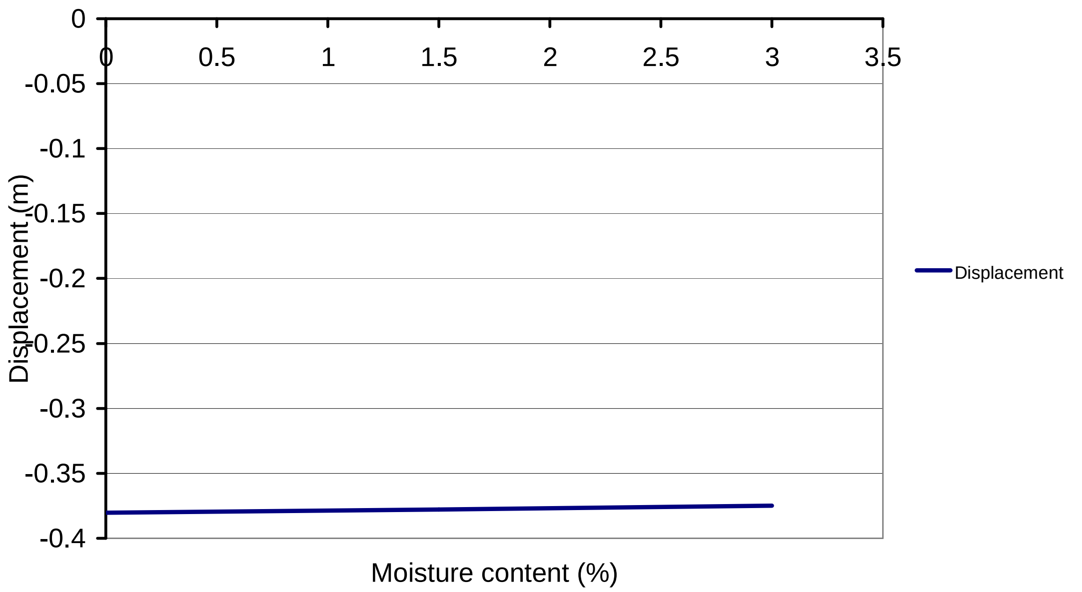

Figure 5.35 Moisture content analysis, vertical displacement

These results indicate that the 0% moisture content situation is more severe as it increases the internal temperature of the member. This increase in severity is not of a significant magnitude. It is difficult to have a completely 'dry' (0% MC) concrete structure, however as an enclosed structure which was constructed several decades ago, a moisture content higher than 1.5% seems unreasonable.

Therefore a moisture content of 1.5% will be used for the remaining analyses.

It should be noted that moisture content has a significant impact on spalling. However, this is not being considered in these analyses as it is difficult to predict and unlikely to occur uniformly over an entire member or indeed structure. Also, it was noted in the INTEMAC report that where concrete had fallen from the bottom of the ribs of the slab, there was little soot staining. This may indicate that this concrete broke off during the cooling phase (Calavera 2005).

5.4.4 Variation in Mesh size

In order to ensure that mesh sensitivity does not introduce inaccuracies into the model, several refinements of the mesh were made. These consisted of reducing the element size and therefore increasing the number of elements. 15 minute upward spread, using calcareous concrete with a moisture content of 1.5% will be used, as noted in earlier sensitivity analyses.

Number of elements	Notes	Input file names (Heat Transfer/Stress Analysis)
20		HeatTB100Delay15v68.inp TBStress100Delay15v68.inp
40		HeatTB100Delay15v68Fine.inp TBStress100Delay15v68Fine.inp
160	Analysis did not complete – stopped after 24.7 hours	HeatTB100Delay15v68Finer.inp TBStress100Delay15v68Finer.inp
480	Analysis did not complete – stopped after 23.3 hours	HeatTB100Delay15v68Finest.inp TBStress100Delay15v68Finest.inp

Table 5.7 Element numbers to be analysed

Note that the finest two meshes did not complete. The cause of this is unknown, however the final results were comparable with those produced by the two coarser meshes at similar times. It is not believed that collapse was initiated, but that some other unknown numerical instability prevented these models from continuing.

As can be seen from the graphs below, while mesh size did affect the results from the models, there is no clear trend as the deflection first increases and then decreases as the number of elements used increases. The variations are generally minor.

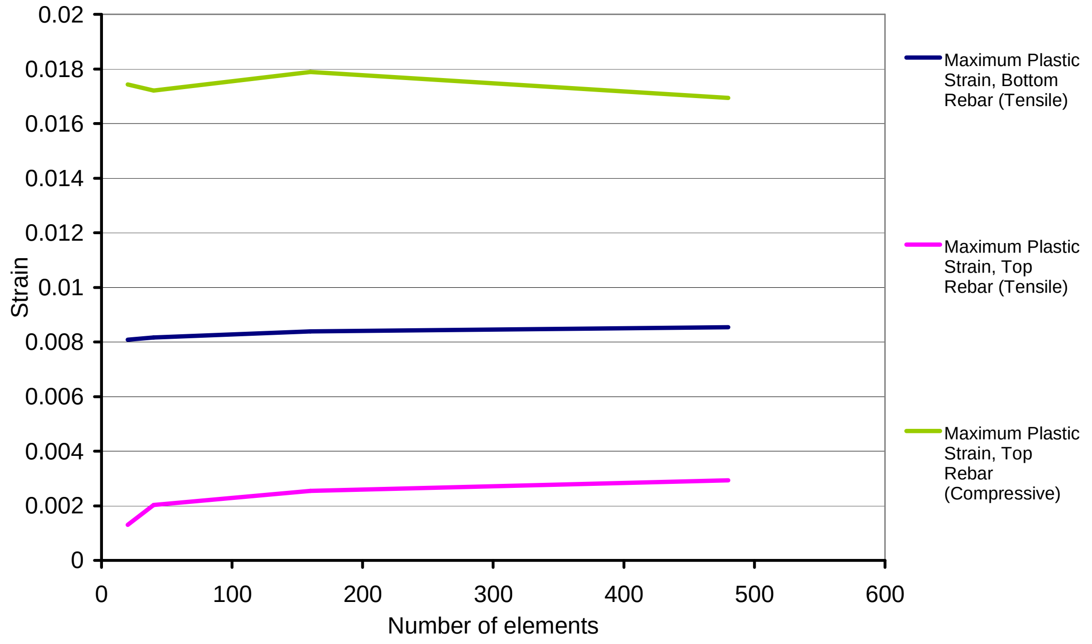

Figure 5.36 Mesh size analysis, Plastic reinforcement strains. Note that Compressive strain in the bottom rebar is always zero.

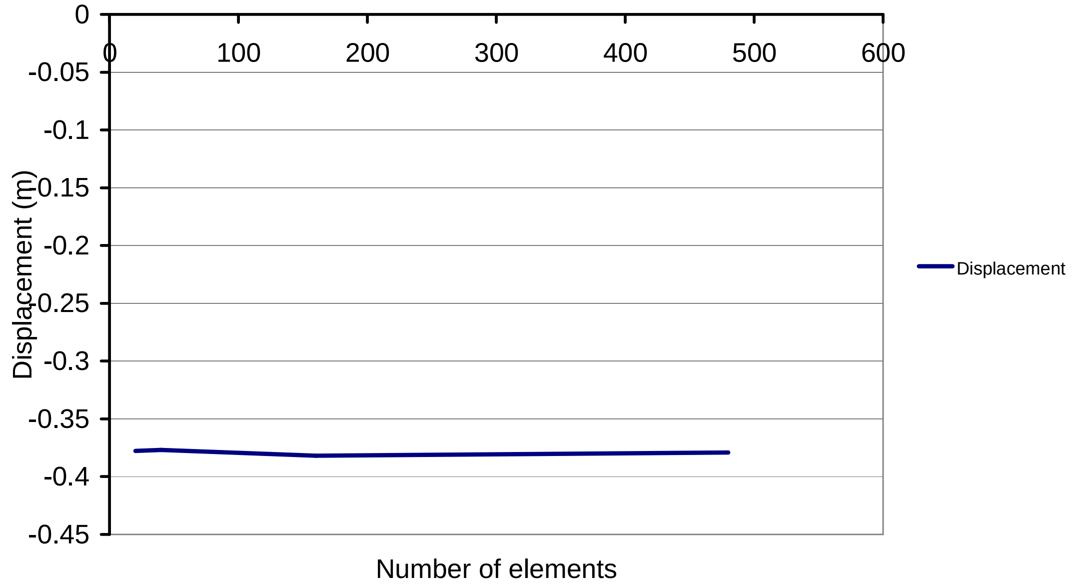

Figure 5.37 Mesh size analysis, vertical displacement

It can therefore be concluded that mesh sensitivity issues will be negligible in the analysis when using these values.

5.4.5 Variation in Fire Duration and Temperature

As discussed in Chapter 5.2.2, the heating curve used can be altered to provide both longer and shorter duration fires, and fires with higher and lower peak temperatures.

The duration of the fire was extended and shortened by 10% to study the impact of longer and shorter fires. The fire curves used are shown in Figure 5.11.

Percentage of basic duration	Total fire duration (minutes)	Input file names (Heat Transfer/Stress Analysis)
90%	108	HeatTB100Lng90LngV68.inp TBStress90LngV68.inp
100%	120	HeatTB100Delay15v68.inp TBStress100Delay15v68.inp
110%	132	HeatTB100Lng110LngV68.inp TBStress110LngV68.inp

Table 5.8 - Fire durations to be analysed

The maximum temperature rise that the fire reached was also increased and decreased by 10%. The fire curves used are shown in Figure 5.12.

Percentage of basic temperature	Peak fire temperature (˚C)	Input file names (Heat Transfer/Stress Analysis)
90%	852.5	HeatTB100Lng90TempV68.inp TBStress90TempV68.inp
100%	945	HeatTB100Delay15v68.inp TBStress100Delay15v68.inp
110%	1037.5	HeatTB100Lng110TempV68.inp TBStress110TempV68.inp

Table 5.9 - Fire temperatures to be analysed

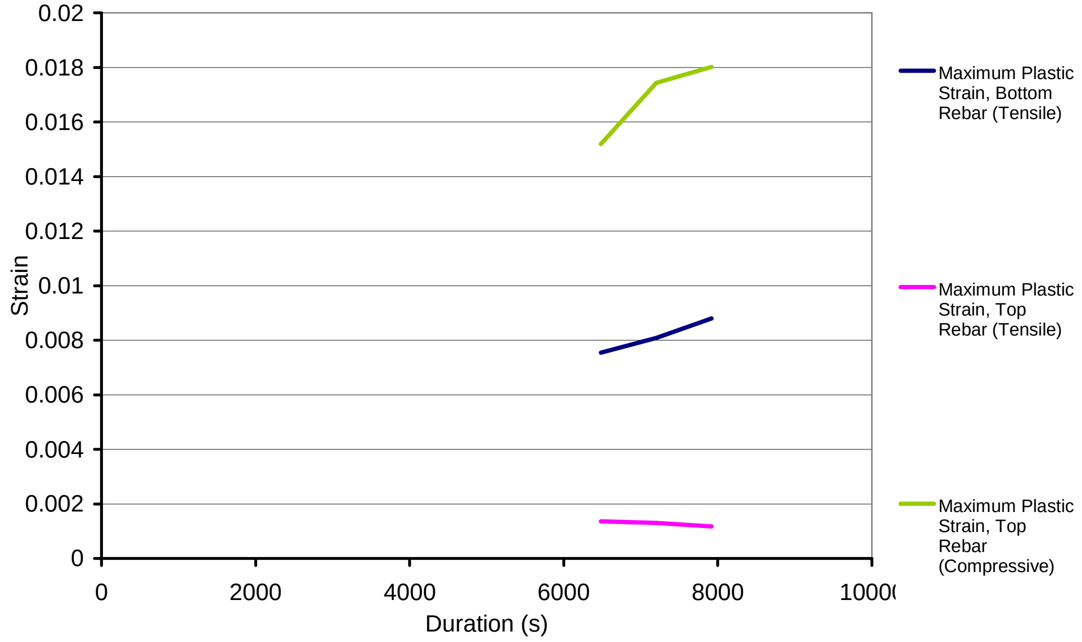

Figure 5.38 Fire duration variation analysis, Plastic reinforcement strains. Note that Compressive strain in the bottom rebar is always zero.

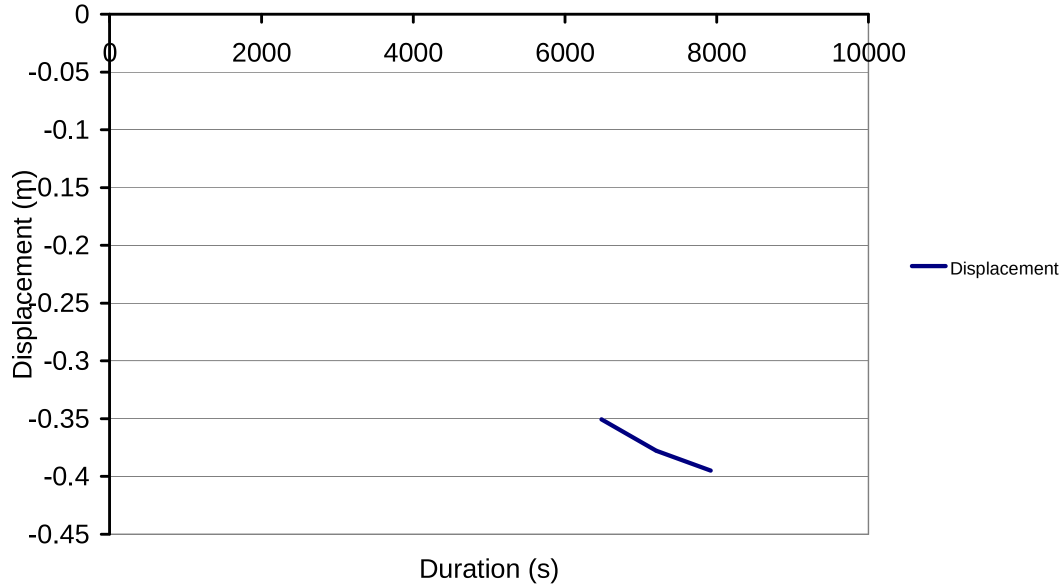

Figure 5.39 Fire duration variation analysis, vertical displacement

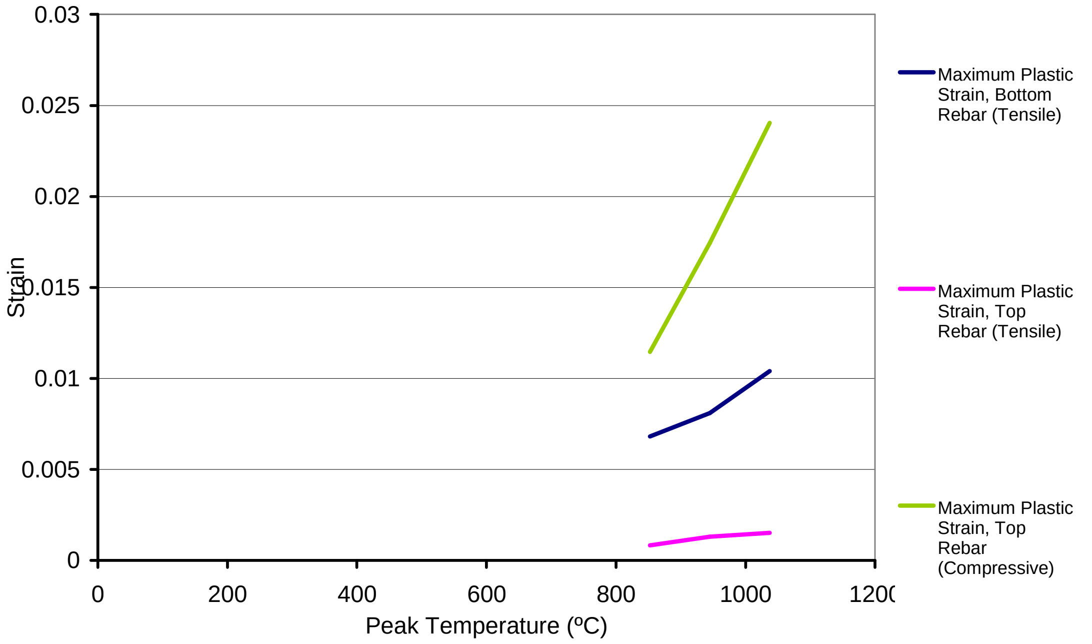

Figure 5.40 Fire temperature variation analysis, Plastic reinforcement strains. Note that Compressive strain in the bottom rebar is always zero.

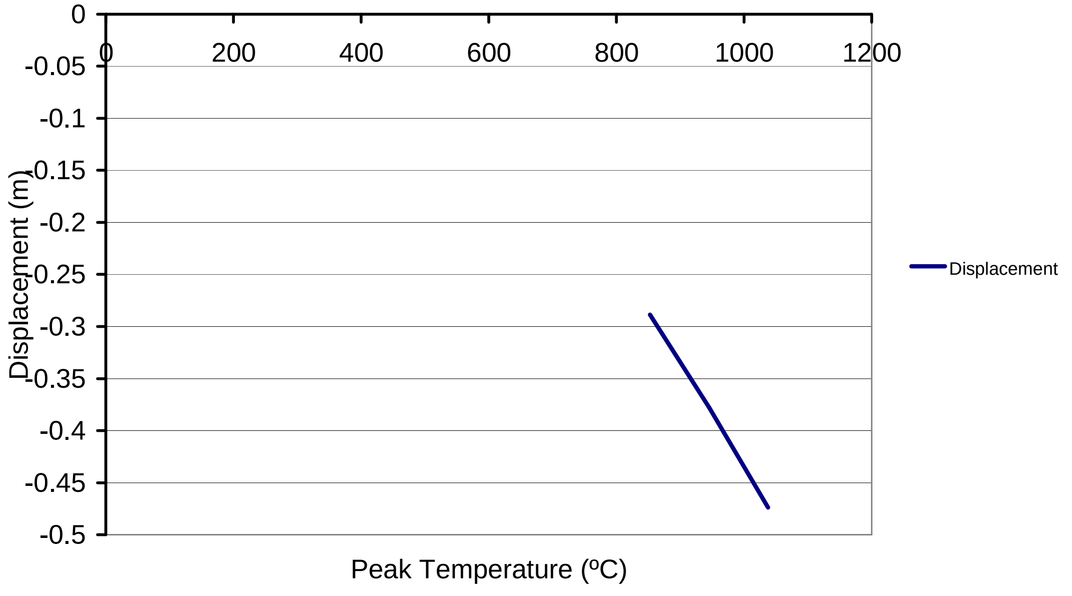

Figure 5.41 Fire temperature variation analysis, vertical displacement

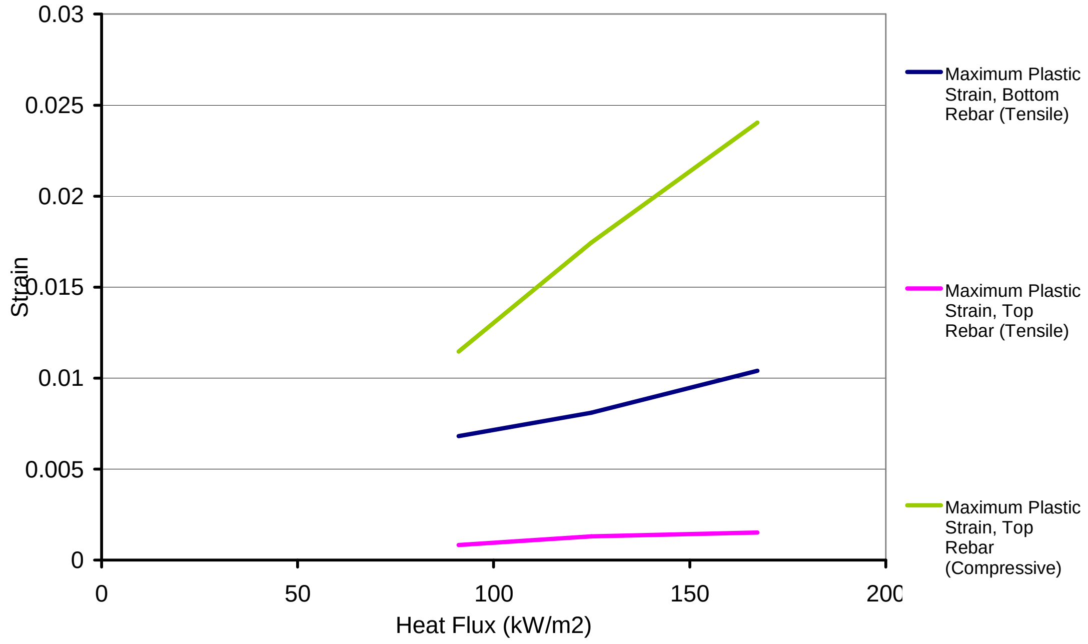

Figure 5.42 Fire temperature variation analysis, converted to heat flux, Plastic reinforcement strains. Note that Compressive strain in the bottom rebar is always zero.

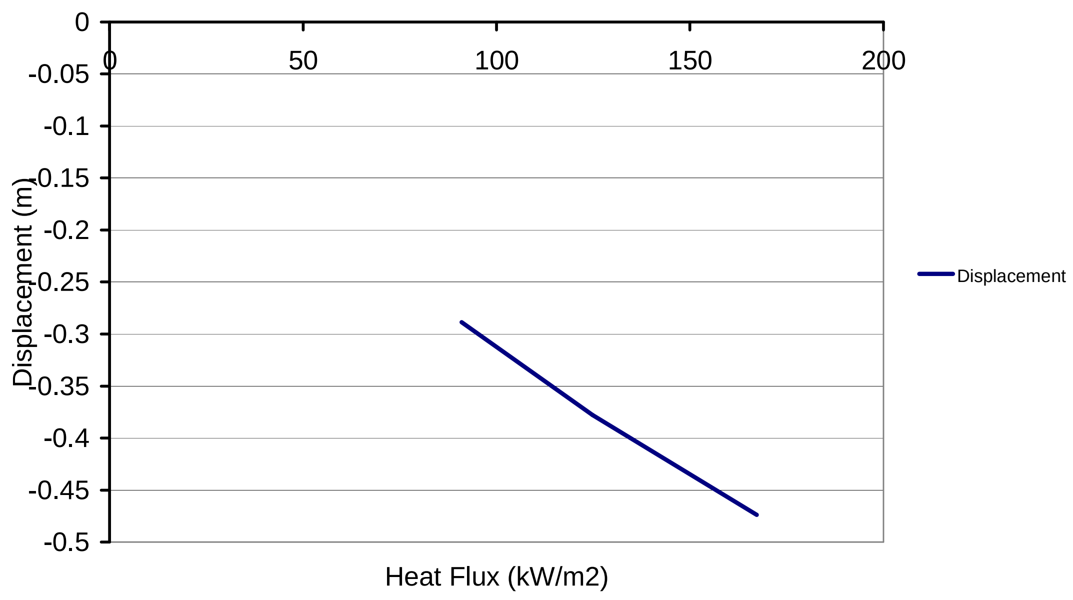

Figure 5.43 Mesh size analysis, converted to heat flux, vertical displacement

These results show that some additional strain and deflection occurs in a more 'severe', i.e. longer or hotter fires than the 100% fire case (Figure 5.38 to Figure 5.41). Figure 5.42 and Figure 5.43 show the effects of temperature variation with the temperature converted into a radiative heat flux.

None-the-less, the 100% fire appears to remain reasonably close to witness observations and average temperatures calculated using the CFD model. The 100% fire will be used in future models, as although a higher temperature or duration fire has a more severe effect on the concrete member failure still does not occur.

5.4.6 Variation in Concrete Type

Initial models used the Eurocode values for Calcareous concrete, rather than Siliceous concrete, as the latter exhibited lower compressive strength at high temperature. Table 5.10 shows the difference between Calcareous and Siliceous concrete peak strength at various temperatures.

Temperature	Calcareous	Siliceous
20˚C	17.5	17.5
100˚C	17.5	17.5
200˚C	17.0	16.6
300˚C	15.9	14.9
400˚C	14.9	13.1
500˚C	13.0	10.5
600˚C	10.5	7.9
700˚C	7.5	5.3
800˚C	4.7	2.6
900˚C	2.6	1.4
1000˚C	1.1	0.7

Table 5.10 Concrete strength, N/mm^2

Initial modelling of the Siliceous concrete using the 'beam' model could not be made to converge due to numerical instability using Abaqus Standard. It was unclear whether the member was truly undergoing collapse, or if the Concrete Damaged Plasticity model was failing to resolve due computational divergence. Reductions in the level of imposed load on the top of the beam showed that the 'beam model' was capable of withstanding 60% of the imposed load and 100% dead load, under fire conditions.

It is therefore possible that due to the unrealistic restraint conditions used for this model, the Siliceous beam underwent failure. As the live loading being applied is unfactored, this might be expected. Generally it is regarded as unlikely that the full live load will be applied during a fire and a factored live load of around 50% is often used.

Modelling of variations in concrete type will be discussed further in Section 5.4.9

5.4.7 Concrete Damaged Plasticity Values

The initial values used to specify Concrete Damaged Plasticity (CDP) for the above sensitivity analyses were not those recommended in the Abaqus Manual and may have been unphysical. Therefore the 'beam model' was re-run using the recommended values for CDP.

The floor models use 1.5% moisture, normal fire duration and temperature and a 15 minute upward spread with Calcareous concrete.

	Dilation Angle	Eccentricity	fb0/fc0	K	Viscosity Parameter
Initial CDP values	15	0.1	0.666	0.01	0
Recommended CDP values	15	0.1	1.16	0.66	0

Table 5.11 - Changes in CDP values

	Max Vertical Deflection (m)	Plastic Strain, Bottom Reinforcement		Plastic Strain, Top Reinforcement	
		Max	Min	Max	Min
Initial CDP values	-0.3777	0.0081	0	0.0013	-0.017
Recommended CDP values	-0.3655	0.0097	0	0.0024	-0.014

Table 5.12 – Results of change in CDP

The recommended values were used in the subsequent full section model (Chapter 6) and the models discussed later in this chapter. It was decided that re-analysing the previous models with the new CDP values would provide little added value. It was judged unlikely that replication of the work would have highlighted more severe results from the sensitivity analysis.

	Input file names (Heat Transfer/Stress Analysis)
Initial CDP values	HeatTB100Delay15v68.inp TBStress100Delay15v68.inp
Recommended CDP values	HeatTB100Delay15v68.inp NewCDPCalc.inp

Table 5.13 - Input files for CDP value change

5.4.8 Variation in concrete tensile definition

All previous models used a constant tensile capacity value for the concrete of 1.5N/mm^2 as noted in Section 5.3.2. , retaining constant tensile strength throughout the analysis with no reduction at higher temperatures. The concrete tensile behaviour was assumed to be ductile. The modulus of elasticity was assumed to have the same value as for compression.

The tensile capacity for concrete is usually governed by cracking. The behaviour of a specific concrete material can be modelled in Abaqus using a stress strain curve with a descending branch.

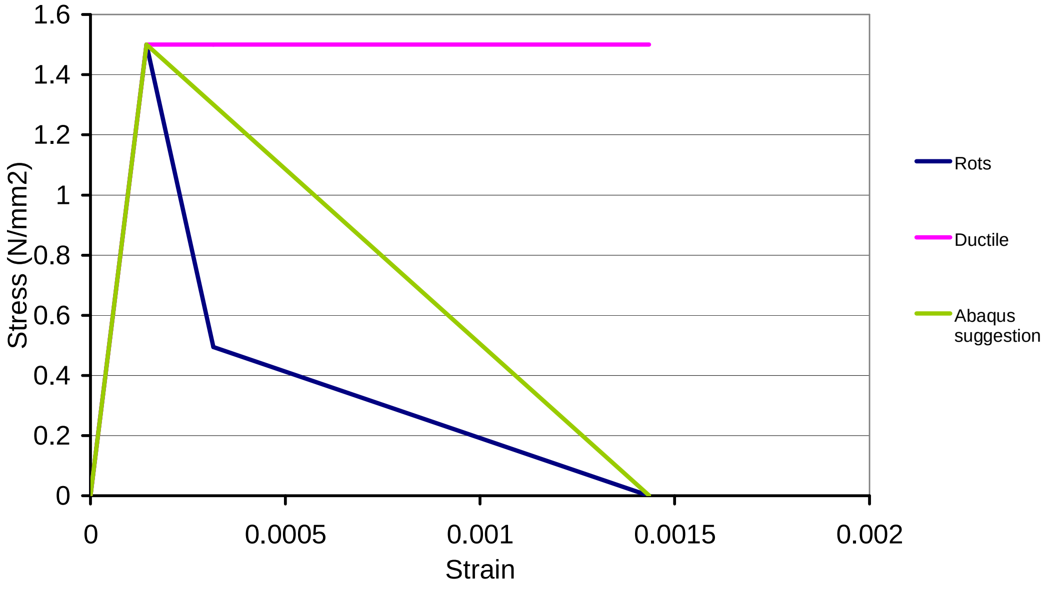

Figure 5.44 Potential concrete tension models

Many methods of selecting values for the descending branch of the stress strain curve are available (Figure 5.44), and Abaqus can be used to model these. Values suggested in the Abaqus manual are zero stress of at a strain 10 times the strain at yielding, though the bilinear post yield behaviour suggested by Rots (Rots, Kuster et al. 1984) may more accurately model the real behaviour of concrete. Abaqus does not allow zero values for the tensile capacity of a material, and therefore stress/strain values were never allowed to drop below a nominal value, beyond which the material is assumed to strain in a ductile fashion.

Unfortunately, it was found to be impossible to achieve convergence of any of the models using a tensile capacity model other than ductile. This is similar to the results from Hong and Varma. (Hong and Varma 2009). This simplification may be unnecessary in Abaqus Explicit and would be a useful area for further study.

However for further analyses, concrete is modelled as a material which behaves in a ductile fashion after yield.

In order to model concrete behaviour, at high temperatures the reduction factors for tensile strength from the Eurocode can be used.

Temperature	Reduction Factor
20˚C	1
100˚C	1
200˚C	0.8
300˚C	0.6
400˚C	0.4
500˚C	0.2
600˚C	0
700˚C	0
800˚C	0
900˚C	0
1000˚C	0

Table 5.14 - Reduction factors for concrete in tension at high temperatures

These reduction factors were used in a version of the ‘floor model’ using Calcareous concrete as discussed in Section 5.4.9.

5.4.9 Variation in Concrete Type using ‘floor model’

As Siliceous concrete could not converge using the ‘beam model’ at full load, the single ‘floor model’ was used to further examine the effect of concrete type. With the additional end restraint provided by both the concrete core of the building and the steel column, the Siliceous concrete beam could be made to reach convergence.

When reducing the yield strength and subsequent ductile behaviour of the concrete in line with the Eurocode reduction values, it was found that the maximum possible reduction factor for siliceous concrete was 0.6 at 300˚C+ (Table 5.14), equating to a tensile strength of 0.9N/mm^2, 5.1% of the ambient compressive strength of the concrete. With any greater reduction factor, numerical instability prevented the model from reaching completion.

Therefore the tensile behaviour of siliceous concrete will be modelled as ductile, reducing in line with temperature according to the Eurocode reduction factors, to a minimum value of 0.9N/mm^2. As can be seen, the Siliceous concrete member performs worse in the case of a fire, with higher deflections. As this type of concrete appears to be closely analogous to that used in the Windsor Tower, and it is the worst case, Siliceous concrete is used in the final multi-floor model of the Windsor Tower.

	Max Vertical Deflection (m)	Plastic Strain, Bottom Reinforcement		Plastic Strain, Top Reinforcement	
		Max	Min	Max	Min
Calcareous 'floor model'	-0.124	0.00276	0	0.02997	-0.0086
Siliceous 'floor model'	-0.127	0.00293	0	0.02951	-0.00835

Table 5.15 Results of change in concrete type

	Input file names (Heat Transfer/Stress Analysis)
Calcareous 'floor model'	HeatOneFloor.inp CalcFullLoadNewCDP.inp
Siliceous 'floor model'	HeatOneFloor.inp SilicMin5Ductile100.inp

Table 5.16 - Input files for concrete type change

5.4.10 Thermal Expansion

During the subsequent modelling of the multiple floor section of the Windsor Tower, it was noticed that the Expansion Coefficients for the concrete and steel and been defined in the wrong form for input to Abaqus.

Expansion Coefficients for Calcareous concrete	Initial Expansion Coefficient (α, 1/˚C)	Corrected Expansion Coefficient (α, 1/˚C)
20˚C	0.000006174	0.000000006
100˚C	0.000006980	0.000006175
200˚C	0.000008660	0.000006622
300˚C	0.000011180	0.000007350
400˚C	0.000014540	0.000008358
500˚C	0.000018740	0.000009646
600˚C	0.000023780	0.000011214
700˚C	0.000029660	0.000013062
800˚C	0.000001520	0.000015190
900˚C	0.000000000	0.000013636
1000˚C	0.000000000	0.000012245

Table 5.17 Expansion coefficients for Calcareous concrete

Expansion Coefficients for Siliceous concrete	Initial Expansion Coefficient (α, 1/˚C)	Corrected Expansion Coefficient (α, 1/˚C)
20˚C	0.000009285	0.000000009
100˚C	0.000010610	0.000009288
200˚C	0.000013370	0.000010022
300˚C	0.000017510	0.000011218
400˚C	0.000023030	0.000012874
500˚C	0.000029930	0.000014990
600˚C	0.000038210	0.000017566
700˚C	0.000000000	0.000020601
800˚C	0.000000000	0.000017949
900˚C	0.000000000	0.000015909
1000˚C	0.000000000	0.000014286

Table 5.18 Expansion coefficients for Siliceous concrete

Expansion Coefficients for Steel	Initial Expansion Coefficient (α, 1/˚C)	Corrected Expansion Coefficient (α, 1/˚C)
20˚C	0.000000000	0.000000000
100˚C	0.000012425	0.000012480
200˚C	0.000012856	0.000012880
300˚C	0.000013264	0.000013280
400˚C	0.000013668	0.000013680
500˚C	0.000014071	0.000014080
600˚C	0.000014472	0.000014480
700˚C	0.000014874	0.000014880
750˚C	0.000015074	0.000015080
800˚C	0.000014103	0.000014103
860˚C	0.000013095	0.000013095
900˚C	0.000013409	0.000013409
1000˚C	0.000014082	0.000014082
1100˚C	0.000014630	0.000014630
1200˚C	0.000015085	0.000015085

Table 5.19 Expansion coefficients for steel

An analysis using the corrected values was performed for both the ‘beam model’ and the ‘floor model’ and compared to the values produced with the recommended CDP values (Section 5.4.8). The ‘beam model’ used a constant tensile capacity 1.5N/mm^2, as the previous beam models did.

	Max Vertical Deflection (m)	Plastic Strain, Bottom Rebar		Plastic Strain, Top Rebar	
		Max	Min	Max	Min
Initial expansion coefficient, Calcareous concrete	-0.3655	0.009707	0	0.0025	-0.014
Corrected expansion coefficient, Calcareous concrete	-0.301	0.008232	0	0.0003	-0.012
Corrected expansion coefficient, Siliceous concrete	-0.3254	0.008869	0	0.000564	-0.0124

Table 5.20 - Results for correction of expansion coefficients using ‘beam model’

	Input file names (Heat Transfer/Stress Analysis)
Initial Expansion coefficient, Calcareous	HeatTB100Delay15v68.inp NewCDPCalc.inp
Corrected expansion coefficient, Calcareous	HeatTB100Delay15v68.inp NewCDPNewEXPCalc.inp
Corrected expansion coefficient, Siliceous	HeatTB100Delay15v68.inp NewCDPNewEXPSilic.inp

Table 5.21 Input files for expansion coefficient change using 'beam model'

As can be seen, the new expansion coefficients allowed a Siliceous concrete 'beam model' to run to completion. These had previously failed due to numerical instability. Therefore the correct definition of expansion coefficients is extremely important. The Siliceous concrete still performs notably worse than Calcareous concrete.

	Max Vertical Deflection (m)	Plastic Strain, Bottom Rebar		Plastic Strain, Top Rebar	
		Max	Min	Max	Min
Initial expansion coefficient, Siliceous 'floor model'	-0.127	0.0029	0	0.0295	-0.00835
Corrected expansion coefficient, Siliceous 'floor model'	-0.09814	0.0017	0	0.0247	-0.0065

Table 5.22 Results for correction of expansion coefficients using 'floor model'

Table 5.22 shows that the new expansion coefficients result in significantly lower deflections and strains than the initial values when using the 'floor model'. This could be attributed to the higher values of the new expansion coefficients, resulting in increased axial compression of the concrete member and a lower area of the beam being in tension.

	Input file names (Heat Transfer/Stress Analysis)
Initial expansion coefficient, Siliceous 'floor model'	HeatOneFloor.inp SilicMin5Ductile100.inp
Corrected expansion coefficient, Siliceous 'floor model'	HeatOneFloor.inp SilicMin5Ductile100NewExp.inp

Table 5.23 Input files for expansion coefficient change using 'floor model'

It was again decided that re-analysing the previous sensitivity analysis models with new Thermal Expansion coefficients would provide little added value. It was judged unlikely that replication of the work would have highlighted more severe results from the sensitivity analysis with respect to time delay, moisture content, mesh size or fire duration.

It is possible that using the corrected values of expansion coefficient could allow more realistic modelling of the tensile properties of concrete. This would be a useful area of further research.The correctly defined values were used in the subsequent full section model (Chapter 6).

The values used to define the expansion coefficient α at 20˚C are artificially low in the corrected values, which will lead to incorrect expansion between 20 and 100˚C. Again variation of these values would be a valuable area for future research. At these low temperatures thermal strains will be relatively low regardless of the expansion coefficient used.

5.5 Conclusions

The ‘simplified ISO-based curve’ provides a reasonably accurate temperature representation of the fire in the Windsor Tower, comparable to the results from CFD and real world observations.

It also reduces the erroneous impression that there is a ‘definitive’ fire that occurred in the Windsor Tower and provides an envelope of temperatures rather than a constant spatial and temporal variation. This is likely to be a more sever case.

The sensitivity study of the physical properties of the concrete model leads to the conclusion that the ‘worst credible case’ variables are:

- 15 minute upward spread
- 1.5% moisture content
- Siliceous concrete
- A 1 hour ISO-based heating phase with 1 hour linear cooling phase
- Mesh effects were found to be negligible.
- Tensile capacity of concrete was simplistically modelled as ductile with a minimum threshold to allow model convergence.
- Shell elements will be used to model the concrete beams

It is also important to ensure correct definition of material properties such as expansion and CDP.

Shell elements are used for the concrete members as they allow accurate modelling of deflected shapes in both Abaqus Standard and Abaqus Explicit. They also have multiple temperature definition points through their depth.

These properties and fire definitions can therefore be used in a larger structural model to examine the collapse mechanisms in a ‘Windsor Tower-like’ structure.

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CHAPTER 6: THE FULL SIZED MODEL

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6.1 Full scale Modelling and Analysis

This section details a large-scale structural model of the Windsor tower based on the results from the small-scale models and sensitivity analyses from the previous section. The aim of this model is to demonstrate holistic structural behaviours, including load redistribution, corresponding to observations during and after the fire. The objective of the model is to provide the ability to examine a large range of possible fire and fire protection scenarios, thus providing a range of results which are generic and can be applied to a wider range of situations.

This will be discussed in the following sections:

- **Full scale Modelling and Analysis**
- **Developing a full scale Windsor Tower structural model**
- **Material values**
- **Relating full scale analysis to sensitivity study**
- **Heat Transfer**
- **Multifloor Variables**
- **Conclusions**

6.2 Developing a full scale Windsor Tower structural model

This model is developed by expanding the single 'floor model' (see Chapter 5), effectively creating additional copies of the individual floors.

A variety of model sizes were considered for this analysis. A model containing the entire section of the building, between the transfer floors, was seen as the best option to investigate holistic structural behaviours and fire protection with different fire scenarios. The transfer floors will tend to isolate mechanical effects from more remote locations.

This model constituted a total of 12 floors, from Floor 5 (The first floor above the lower transfer floor) to Floor 16 (Just below the upper transfer floor). This is shown in Figure 6.1, with the transfer floors represented as boundary conditions at the top and bottom of the steel column. This assumes that the transfer floor is infinitely rigid and does not undergo any deflection. This simplified assumption was made based on the transfer floor being composed of eight 3.5 m deep concrete beams.

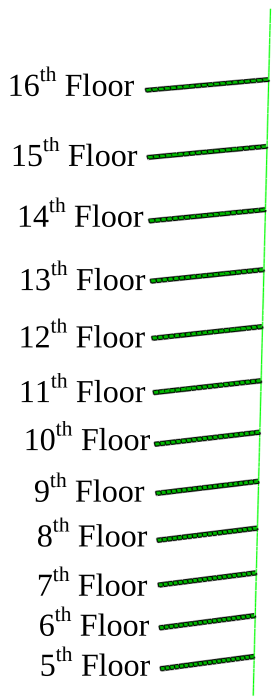

Figure 6.1 - Multi Floor Abaqus model

The steel columns change profile from floor to floor, with the largest sections at the bottom. The properties of these sections are previously noted in Table 5.4.

Column	Column Section
Floor 16- Transfer 2	Special
Floor 15-16	Two UPN 100
Floor 14-15	Two UPN 100
Floor 13-14	Two UPN 120
Floor 12-13	Two UPN 120
Floor 11-12	Two UPN 120
Floor 10-11	Two UPN 120
Floor 9-10	Two UPN 140
Floor 8-9	Two UPN 140
Floor 7-8	Two UPN 140
Floor 6-7	Two UPN 160
Floor 5-6	Two UPN 160
Transfer 1 – Floor 5	Two UPN 160

Table 6.1 Steel column sections

Although construction drawings do not show a column between Floor 16 and the upper transfer floor (T2) there appear to be some members connecting the two, as can be seen in photographs Figure 3.8 and Figure 3.9.

It is unknown whether the members between Floor 16 and the upper transfer floor have any structural capacity, and it is possible that they are purely part of the façade. In the following 21 models this member has been represented by two UPN 100 sections.

The connection between the columns and the top transfer floor is varied, as discussed later, and so will model both the cases where these members have structural capacity, and where they do not.

This model can be adapted to allow further examination of variations in support conditions, member dimensions and material strengths to represent any specific area of the building, or a similar building.

The top and bottom of the steel column, and the connection at each floor, can be reasonably regarded as fixed against both rotation and displacement due to the connection detail shown in Figure 3.2. The 'Transfer Floors' are assumed to be too strong to exhibit deflection.

Additionally, to prevent out-of-plane warping of the steel columns during expansion, the ends of the floor slabs are assumed to be fixed against displacement perpendicular to both the beam and floor. Rotation about the axis of the floor is also restrained. This simplification is used to model the restraining effect of the edge beam. (Figure 6.2) Note that the sides of the beam were not restrained against twisting or horizontal displacement as would be the case in a continuous slab. Inclusion of this restraint would be more realistic.

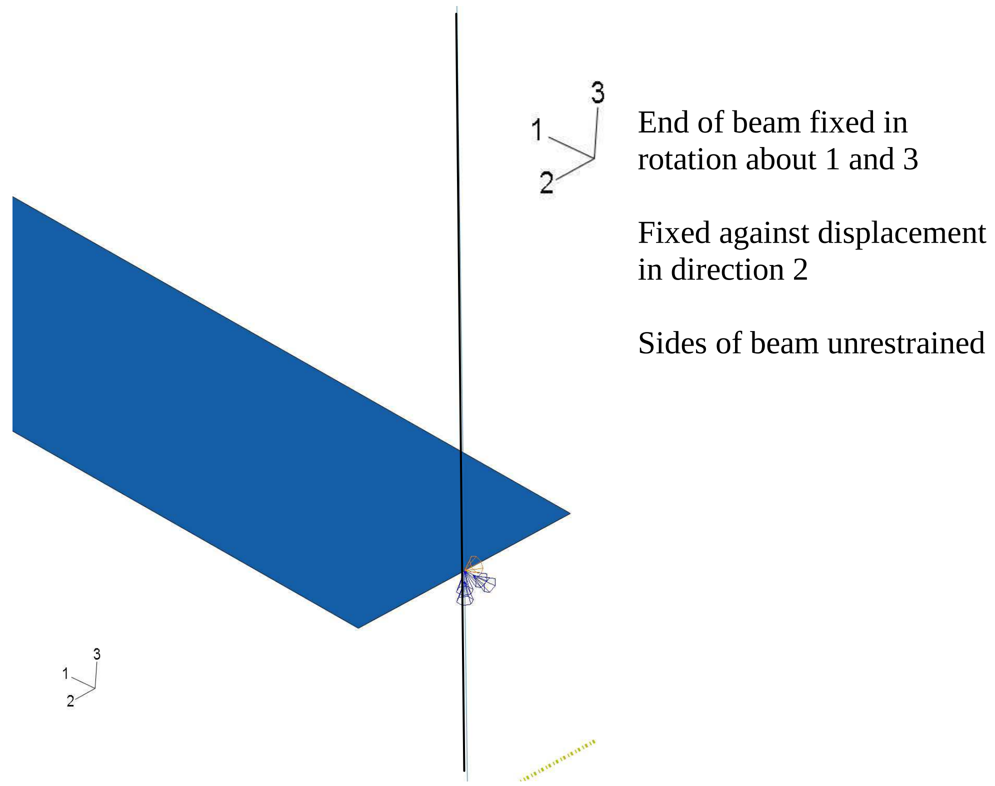

Figure 6.2 Diagram of End of Beam showing restraints

6.3 Material values

The multiple floor model is related to the sensitivity analysis by using the 'worst credible case' developed from the analyses in Chapter 5. To summarise, these variables are:

- Siliceous concrete - this type of concrete is the closest analogue to the concrete mix used in the Windsor Tower (Calavera 2005)
- Full Live loading
- 1.5% moisture content – while a completely dry concrete would have worse thermal properties where spalling is neglected, it is difficult to achieve

1 hour ISO-based fire with 1 hour cooling phase was used, although a longer or hotter fire would have a more severe effect on the concrete. It was considered that as the concrete member did not fail during exposure to more severe fires, this case would be reasonable.

Fire spread rate is a variable considered during the analyses of the multiple floor model, with values based on either:

- Estimates derived from eyewitness observation and media reports i.e. a 15 or 30 minute downward spread.
- The worst case in the sensitivity analysis, 15 minutes upwards spread, as derived in Chapter 5.

6.4 Relating full scale analysis to sensitivity study

As Abaqus Standard is unable to model structural collapse, the Abaqus Explicit solver was used in the anticipation of several cases resulting in structural failure. However, as the sensitivity analyses was carried out using the Abaqus Standard solver, it was deemed necessary to provide a comparison analysis of one case of the multiple floor model using Abaqus Standard. The results from this Standard analysis were then compared against an Explicit analysis to ensure consistancy. In all Abaqus Explicit analyses, the Double Precision solver was used as this is recommended for analyses with a large number of increments.

These analyses used the material properties as detailed in Section 6.3, with a 30 minute downward spread rate and 'perfect' fire protection (i.e. no heating in steel columns)

In an Explicit analysis several factors must be taken into account beyond those necessary for an Abaqus Standard analysis. The initial loading stage should apply the load smoothly rather than instantaneously, as this will prevent oscillations in the loaded members.

Due to the computational methods used in an Explicit analysis, a model with a duration as high as 100 hours will require a degree of either 'time-scaling' or 'mass-scaling' to allow completion in a reasonable time period (less than two days). Time-scaling was used in the analysis as no rate dependant material properties are involved in the analysis (Abaqus 6.8-1 Manual 2008).

Abaqus uses no fixed units, such as seconds, for time. A 100 hour fire scenario may be set as 360,000 units (1 unit =1 second), though Abaqus could equally regard one unit as one minute, hour or day. The system of units chosen for this project defines one unit as one second.

Time-scaling is achieved by reducing the number of time units used in the analysis. For example, a 100 hour fire scenario can be reduced from 360,000 time units (1 unit

=1 second), to a matter of tens or hundreds of time units (1 unit =10mins, for example). As Abaqus has no built-in time-scale, this can be done freely.

Time scale factor	Duration (s)	Maximum vertical displacement in model (m)	Comments
3.6×10^{6}	0.1	-4.363×10^{-2}	Oscillations
360,000	1	8.555×10^{-2}	Oscillations
100,000	3.6	1.511×10^{-2}	Oscillations
10,000	36	-1.422×10^{-1}	Job completes
3,600	100	-4.987×10^{-1}	Failed to Complete
1000	360	-1.718×10^{-1}	Job completes
600	600	-2.592×10^{-1}	Failed to Complete
Abaqus Standard Analysis	360,000	-1.351×10^{-1}	Job completes
10,000	36	-1.356×10^{-1}	Loads added smoothly

Table 6.2 Comparison of time-scaling factors

To ensure that the time-scaling factor produces results in line with those provided by the Abaqus Standard analysis, a sensitivity analysis was carried out using different durations.

In these initial models, the loads were added instantaneously at the start of the loading step. This caused oscillations and resulted in displacements at the end of the loading stage which did not agree with those produced by the Standard analysis. It should be noted that all the analyses produced the same results at the end of the loading stage as no times scaling was used for this step.

A time-scaling factor of 100,000 or greater results in oscillations in the model (it 'bounces') as if the fire has been applied to it overly suddenly. In the case of scaling factors of 600 and 3,600, the Explicit analysis could not be made to complete due to stability errors. Collapse did not appear to be occurring in these analyses, and it is possible that a numerical error was causing the analyses to terminate.

A time-scaling factor of 10,000 with the duration thereby set to be 36s, produces the closest results to those of the Standard Analysis. No oscillations were observed as in the shorter duration analyses. This analysis was then re-run using a smooth load application in the initial loading step, resulting in even closer agreement with the output from the Abaqus Standard analysis as shown in Table 6.2. Comparison of these two results can be seen in Figure 6.3 and Figure 6.4. Loading was applied smoothly in all further analyses.

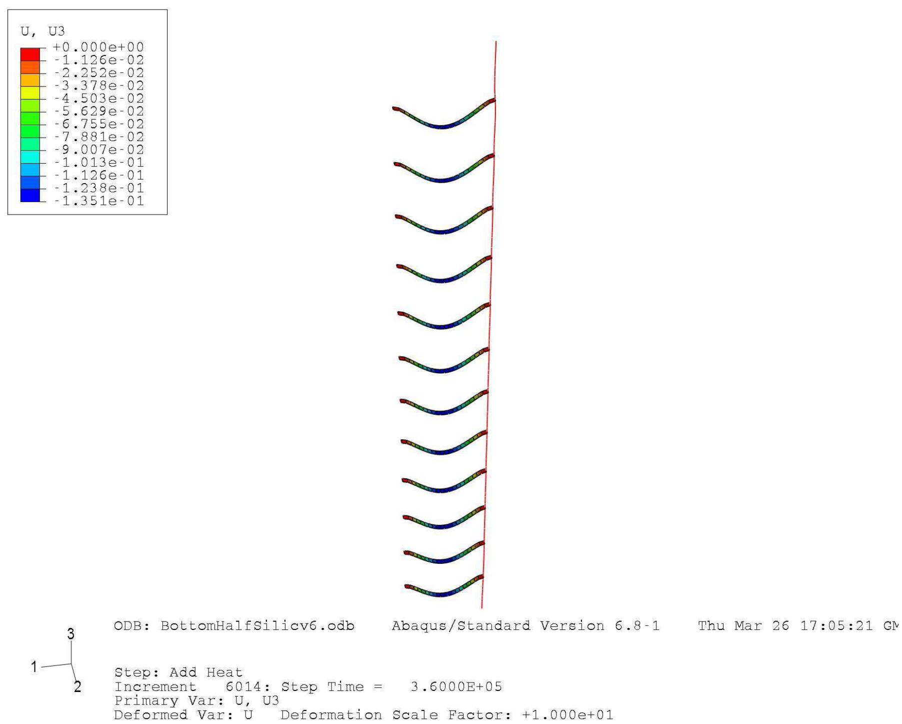

Figure 6.3 Final vertical displacements of Abaqus Standard analysis

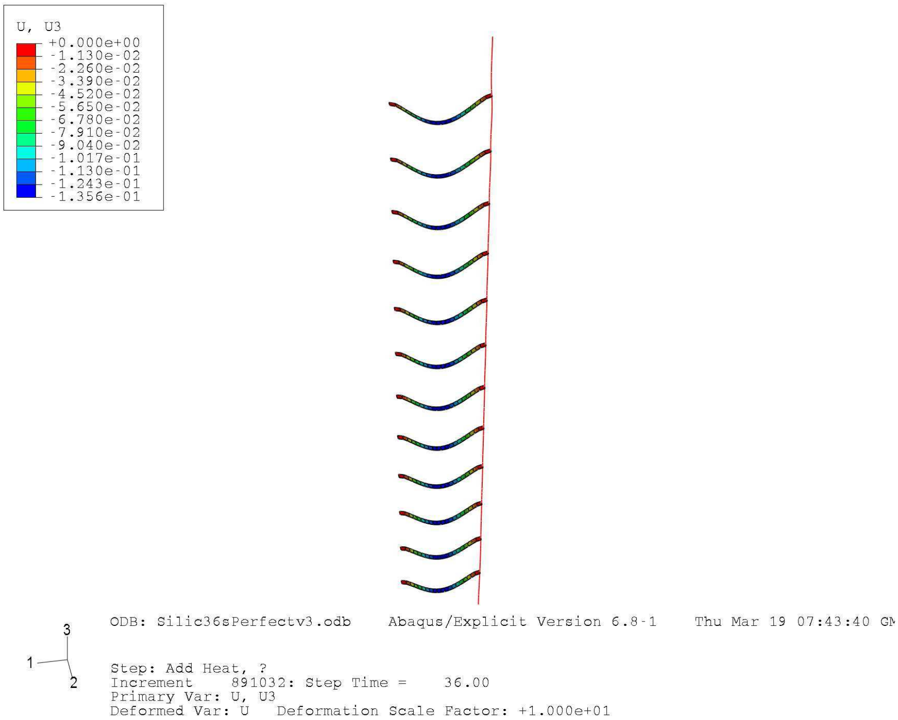

Figure 6.4 Final vertical displacements of Abaqus Explicit analysis, Time-scaling factor 10,000

In summary of this sensitivity analysis, a time-scaling factor of 10,000 was chosen for subsequent Explicit analyses, reducing the duration of the heating phase to 36s.

6.5 Heat Transfer

A series of heat transfer analyses were performed in order to model the temperatures within the structural members undergoing a variety of fire scenarios.

6.5.1 Concrete Temperatures

In order to determine the temperature within the concrete elements of the building, a heat transfer analysis was performed using the methods discussed in Chapter 5.

This heat transfer analysis applies temperatures to the top and bottom of the shell elements in sequence with the rate of spread either up or down the building. For example, in the case of 30 minute downward spread, the fire on top of floor 16 was started at time 0s, whereas that beneath floor 5 (on top of the lower transfer floor) began at time 21600, 6 hours later.

Nodal temperature output data from an Abaqus heat transfer analysis requires no additional time scaling before application to the stress analysis. This is because the temperature data is automatically scaled to fit the overall duration of the stress analysis. Whilst this is useful in an explicit time-scaled analysis, care must taken when using Abaqus Standard as this feature can act as an overly sudden application of conditions, resulting in a failure to converge.

6.5.2 Steel Temperatures

No temperatures were applied to the exterior steel columns in single floor analysis (see chapter 5). This variable is included in the multi-floor model to examine the effects of a fire in larger vertical sections of the building.

The definition of the steel columns as ‘beam elements’ in the stress analysis poses an interesting problem, as no directly compatible equivalent heat transfer element for this stress element exists. As an alternative to a complicated heat transfer analysis and data processing using a different type of heat transfer element, a simple calculation of the steel temperature was used to provide the temperatures in both an insulated and uninsulated steel member.(Drysdale 1998)

The following formulae were used:

- For uninsulated steel

$$T_s^{j+1} = T_s^j + \frac{A_s}{V_s}\left\{\frac{\varepsilon_r\sigma}{\rho_s c_s}\left[\left(T_{F(ave)}^{j+1}\right)^4 - \left(T_s^j\right)^4\right] + \frac{h}{\rho_s c_s}\left[T_{F(ave)}^{j+1} - T_s^j\right]\right\}\Delta t$$

T_s^{j+1} = Steel Temperature

T_s^j = Steel Temperature, previous step

A_s = Surface Area of steel

V_s = Volume of steel

ε_r = Emissivity

σ = Stefan-Boltzmann constant

ρ_s = Steel density

c_s = Thermal capacity

$T_{F(ave)}^{j+1}$ = Average furnace temperature during timestep

Δt = timestep

- For insulated steel.

$Q_{in} = \frac{k}{L}\left(T_{F(ave)}^{j+1} - T_s^j\right).A_i\Delta t$ and $T_s^{j+1} - T_s^j = \frac{Q_{in}}{V_s\rho_s c_s}$

Q_{in} = Energy entering steel

k = Thermal conductivity of insulation

L = Thickness of insulation

A_i = Internal surface area of insulation

The vertical spatial variation in the temperature of the steel column is disregarded, as any weakening of a tensile or compressive member in one location is likely to reduce the capacity of the member as a whole.

Differential heating in the horizontal plane was also neglected as the steel columns can be assumed to be exposed to fire on all sides, as observed and recorded during the fire.

The insulated member analysis used ‘Vermiculux’ insulation with a thermal conductivity of 0.13 W/mK to a thickness of 50 mm, as this would produce a fire resistance of 3 hours in accordance with the requirements of the building upgrade (Kono 2005). The thickness was calculated in accordance with the recommendations in the Association for Specialist Fire Protection guidance (ASFP 2004). This manufacturer’s insulation is commonly used in Spain (GIDAI 2009).

A sensitivity analysis was performed to investigate the effect of varying timesteps associated with calculation of steel temperature. Two different timesteps were selected – a ‘Large’ timestep which matched the times used in the ISO-based fire applied to the concrete, and a ‘Small’ timestep.

	'Large' Timestep	'Small' Timestep
Timestep prior to ignition	300s	5s
Timestep in first 5 minutes of fire	60s	1s
Timestep in remainder of duration	300s	1s

Table 6.3 – Timesteps used for steel temperature sensitivity analysis

It was found that in the case of insulated steel, the timestep had little impact, with the 'Large' timestep producing temperature variations in the range +1.1% and -1.8%. Therefore, for expediency, the 'large' timestep values were used to reduce the quantity of tabular data copied to Abaqus (Figure 6.5).

In the case of uninsulated steel timestep variation produced a more noticeable variation in temperature. A 'Large' timestep produced a temperature variation ranging from -21.3% to +8.9%. Therefore the small timesteps were used for the uninsulated steel (Figure 6.6).

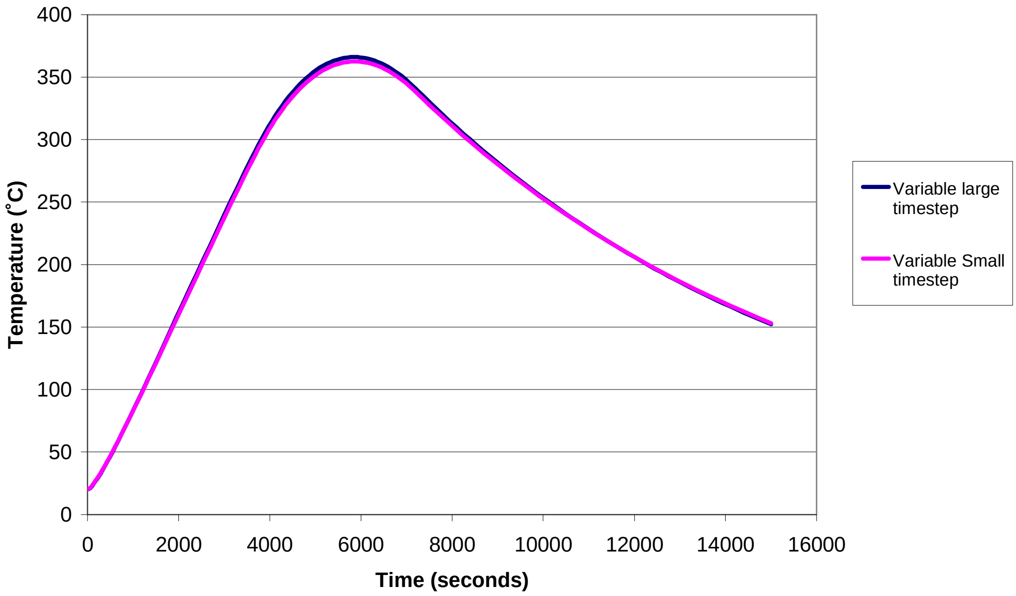

Figure 6.5 - Comparison of timesteps for Insulated steel

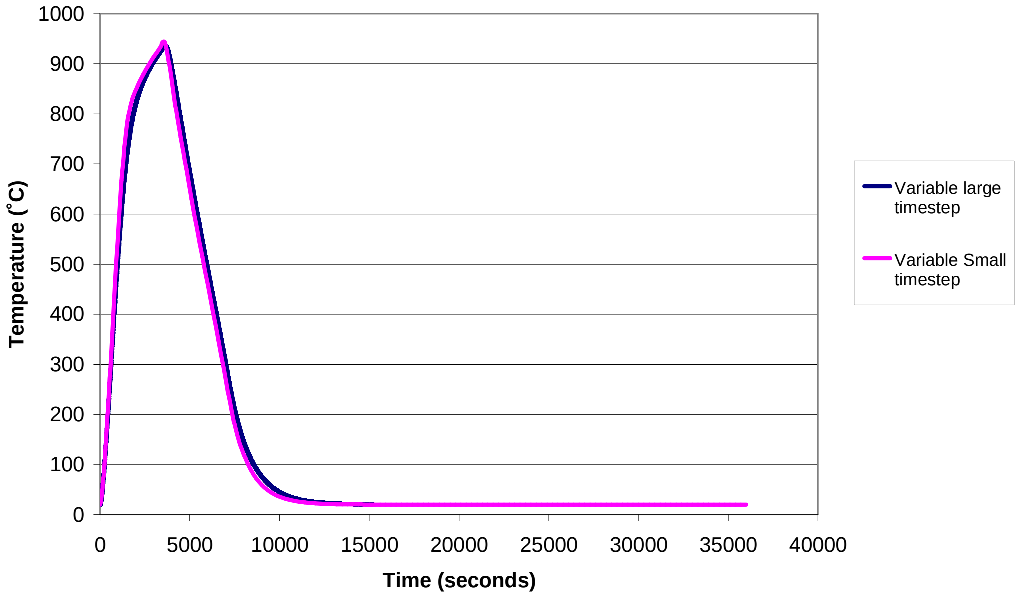

Figure 6.6 - Comparison of timesteps for Uninsulated steel

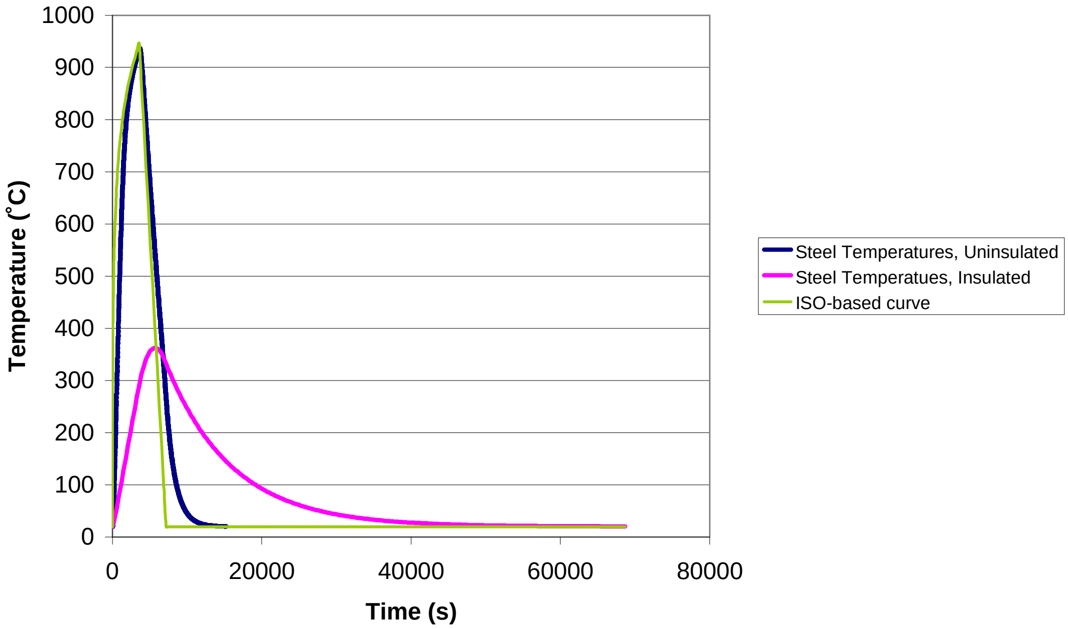

Figure 6.7 Steel temperatures

The steel temperatures are time delayed to match the delay in the fire spreading up or down the building.

The steel temperatures are applied directly to the stress model as predefined fields (Figure 6.7). This allows the steel members to be protected and unprotected in any configuration of interest without the need to produce a new concrete member heat transfer. The temperatures were calculated for a steel column formed from two UPN100 sections – this simplification is results higher temperatures being applied to the larger box sections. The uninsulated peak temperatures and heating and cooling times are very similar regardless of section size. The double UPN100 section has the highest peak temperatures when insulated and again displays very similar heating and cooling times.

The duration of the temperature inputs from the simple heat transfer calculations must be scaled in order to match any time-scaling used in an Abaqus Explicit analysis, as the Explicit Analysis will not carry this out automatically. Therefore the duration of these predefined fields must be reduced in line with the timescaling factor used in the analysis.

6.6 Multifloor Variables

A wide range of variables are explored using this model and this section seeks to examine the effects of:

- Variation in fire protection
- Variation in steel column end supports
- Variations in fire spread rate and direction

The particular variations under consideration are shown in the table below.

	Fire Spread rate				
Fire Protection on columns	30 minutes downward	15 minutes downward	Applied to all floors at same time	15 minutes upward	Only on 9th Floor
'Perfect'	A				A
Full	C				C
None 5-6	B				
None 9-10	C				C
None 12-13	B				
None 13-14	B				
None 14-15	B				
None 15-16	B				
None	C	A	A	A	
None 9-11 (Two floors)	A				

End conditions – **A** = Fixed against all rotation and translation at top
B = As **A** but free against vertical displacement at top
C = Both **A** and **B** modelled

Collapse condition - = No collapse
= Collapse in support condition **A**
= Collapse in support condition **B**

Table 6.4 - Fire, support and fire protection conditions examined, with results.

6.6.1 Variation in insulation

Three differing levels of fire protection were used in the model, namely 'Perfect' (where no temperature data was applied the steel columns), 'Insulated' and 'Un-insulated'. Table 6.4 shows how the insulation was applied to the building.

As can be seen from Table 6.4, where the exterior steel columns are fully insulated collapse does not occur, irrespective of support condition.

However, a horizontal displacement can be seen in the steel columns between floors 14 and 15 (Figure 6.8). This appears to have been caused by the expansion of the steel column being restrained by the top support condition. As can be seen in Figure 6.9, where no top vertical restraint exists the upward displacement is 77mm.

(Note that in the following Abaqus output figures, a scale factor is used for deformation for emphasis. Generally this is a factor of 10, though a factor of 1 is used to show collapse).

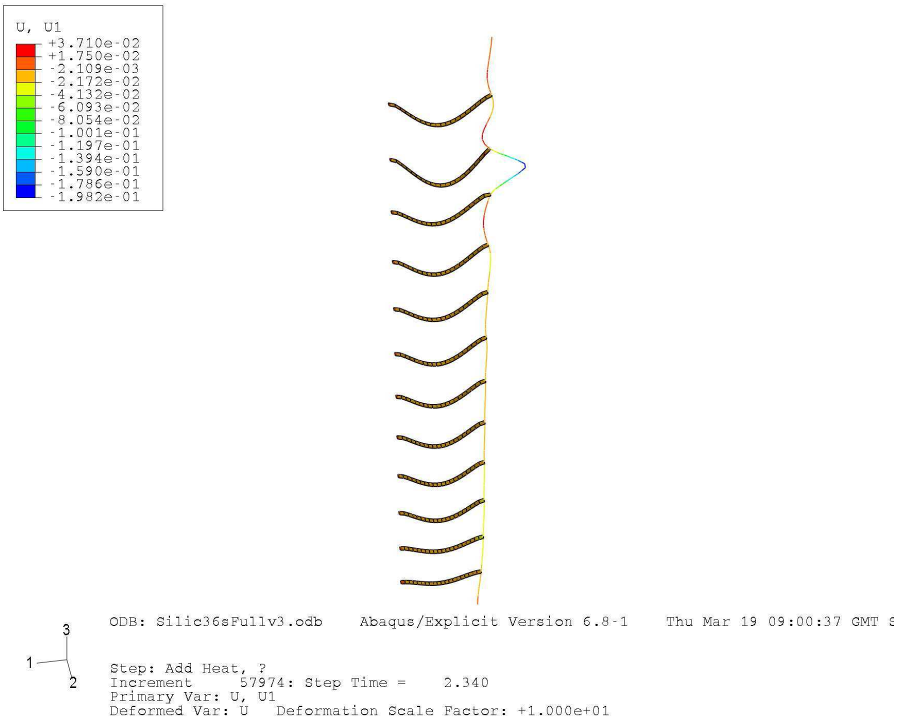

Figure 6.8 - Maximum horizontal displacement, Full insulation, vertical restraint at top.

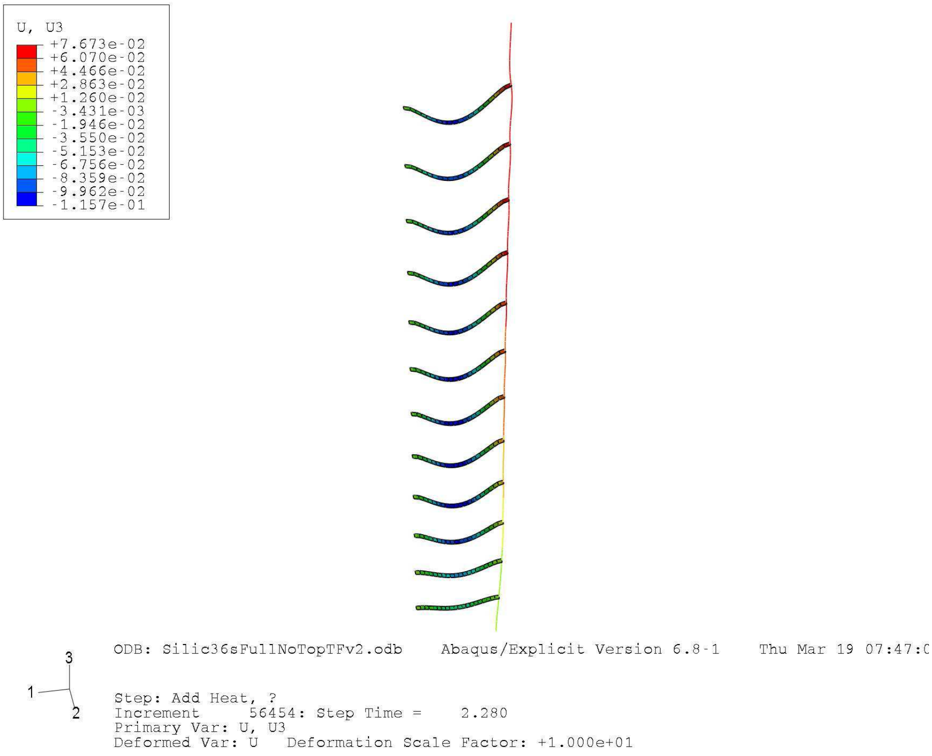

Figure 6.9 - Maximum upward displacement, Full insulation, no vertical restraint at top

This 'bulge' between floors 14 and 15 may be due to the lack of eccentricities in the steel columns. It is possible that in the Windsor Tower, this 'bulge' did not occur as smaller horizontal displacements evolved on individual floors. It is possible that the outward displacement in the Abaqus model is unrealistically concentrated into one of the first columns to demonstrate a significant rotation and therefore create an eccentricity. It is also likely that this bulge is a result of the unrealistically stiff support conditions at the top and bottom of the column, and that in reality no such bulge would have taken place due to deformation of the supports.

In the case of a model where one column remains uninsulated, such as the column between floors 9 and 10, the horizontal displacement initially evolves in the same position between floors 14 and 15 (as the model involved downward spread with a delay of 30 minutes per floor) and then concentrates at the uninsulated floor. This occurs at the time that the fire reaches the uninsulated floor (Figure 6.10, Figure 6.11).

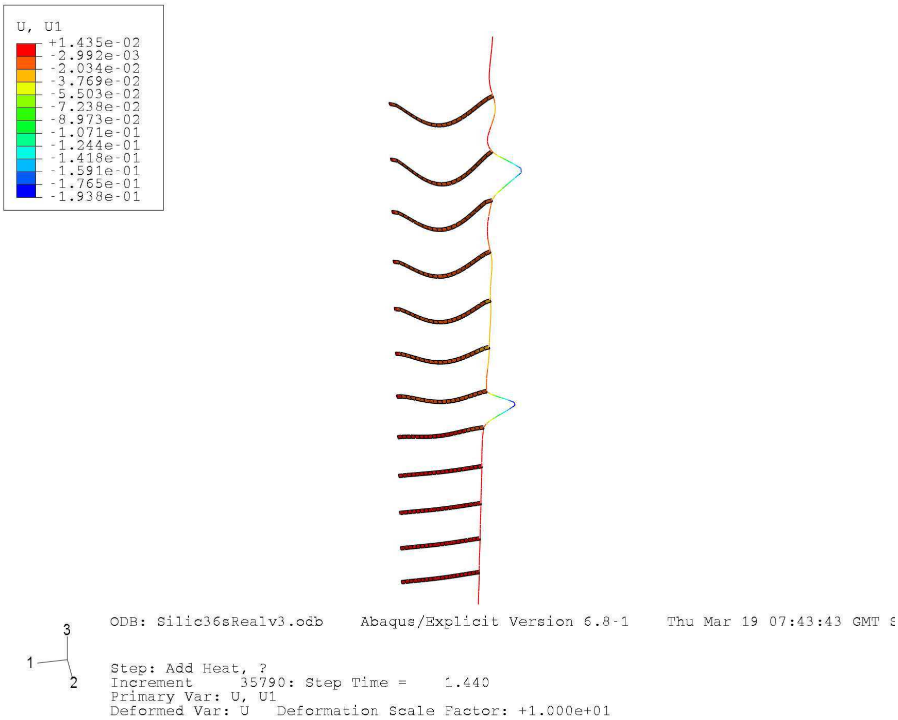

Figure 6.10 - Horizontal displacement between floors 9 and 10 and 14 and 15

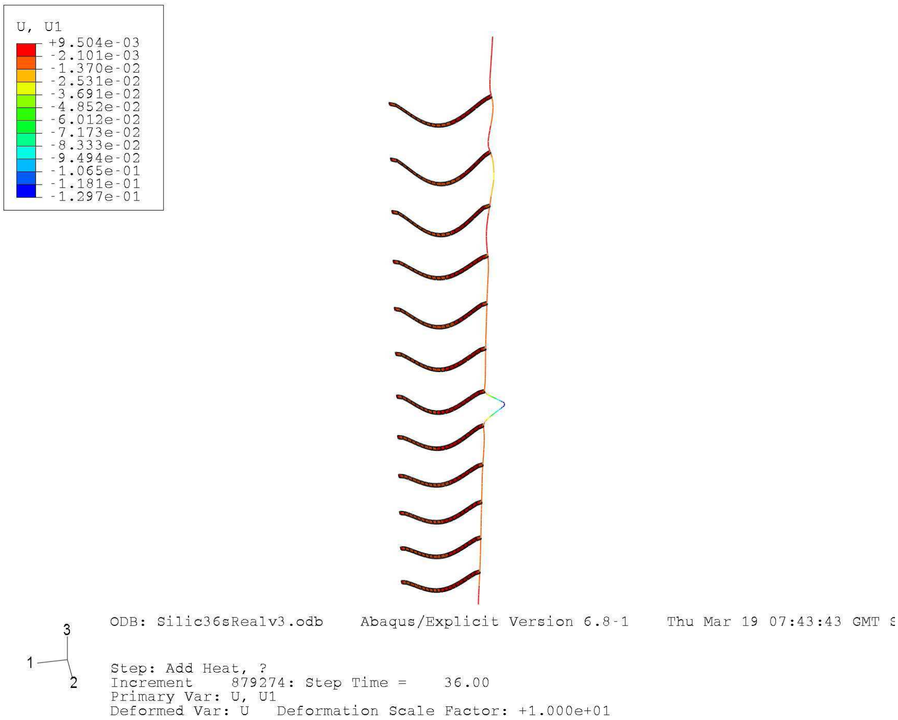

Figure 6.11 - Final horizontal displacement, column 9 to 10 uninsulated, vertical restraint at top

As can be seen from the final horizontal displacement in Figure 6.11, which approximates the 'Real' fire case, the bulge demonstrates a similar level of deflection to that seen in the Windsor Tower (Figure 3.7)

The evolution of horizontal displacement of the mid-points of the steel columns is shown in Figure 6.12, demonstrating that the column between floors 14 and 15 reaches a peak displacement immediately prior to the displacement concentrating in the column between floors 9 and 10.

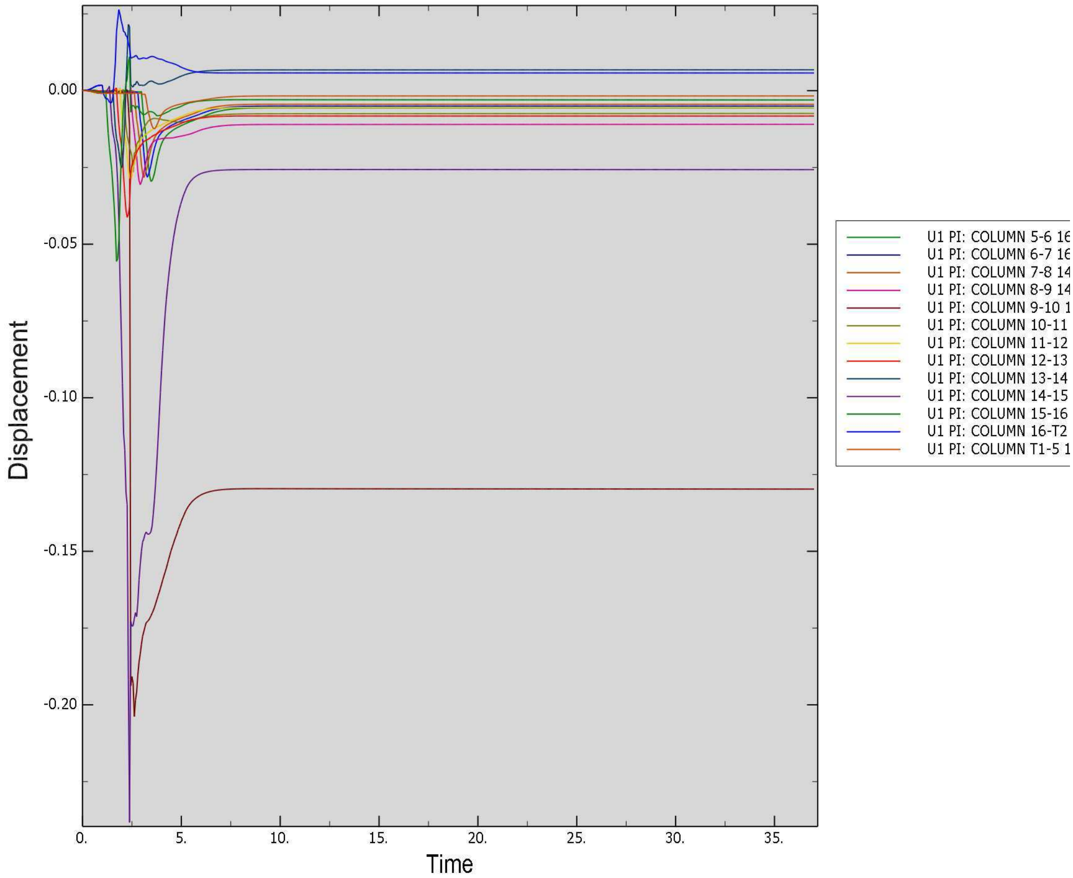

Figure 6.12 - Horizontal displacements, column 9 to 10 uninsulated, vertical restraint at top

Further analysis of the results produced by the fully insulted column model demonstrate the forces and displacements acting within the concrete floor slabs. Figure 6.13 shows the vertical displacement of the 9th floor slab during the early phase of heating.

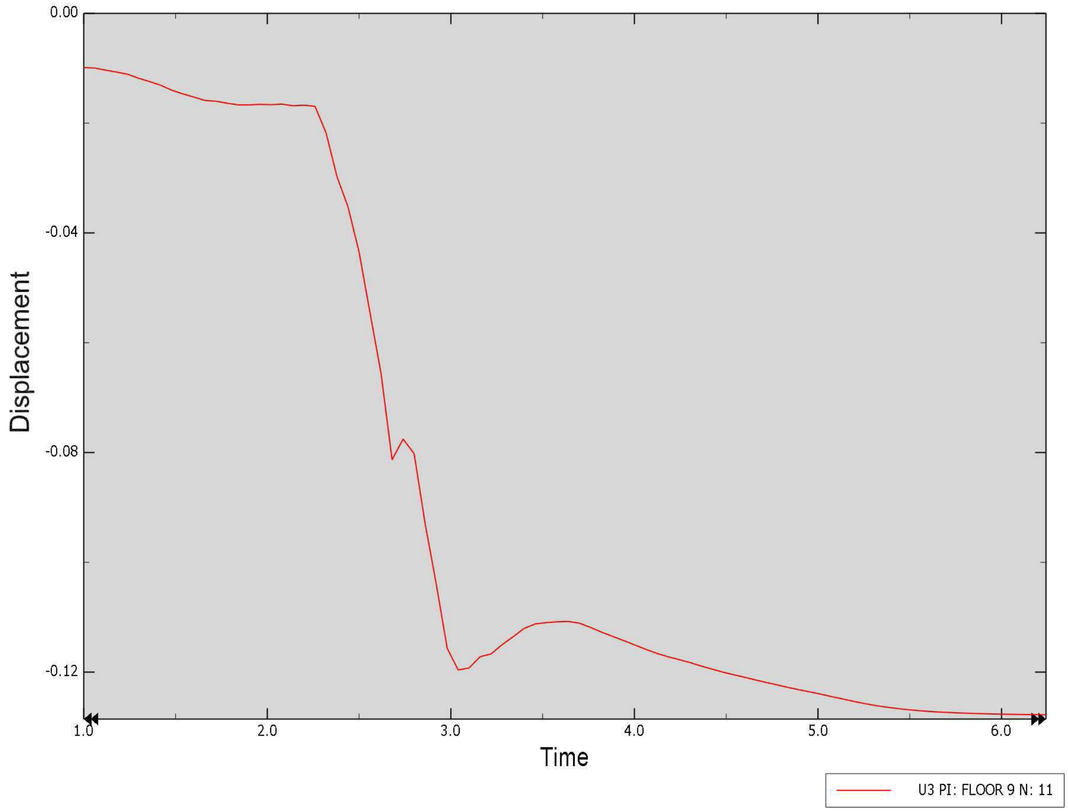

Figure 6.13 - Vertical Displacement, mid point of 9th Floor. Note that Time axis is scaled by a factor of 10,000

As can be seen from Figure 6.13, initial displacement is low and caused by heating of other areas of the structure. The fire reaches the top of Floor 9 at 12600s and reaches its peak temperature at 16200s. The fire reaches the floor below, i.e. the bottom of Floor 9, at 14400s and reaches its peak at 18000s. The heat is then conducted into the slab resulting in major deflection at around 25000s. Some recovery of deflection is exhibited during cooling.

The steel column between floors 9 and 10 also exhibits deflection during this time. (Figure 6.14). As can be seen, after an initial peak of outward displacement, the column recovers back towards the core leaving a residual displacement.

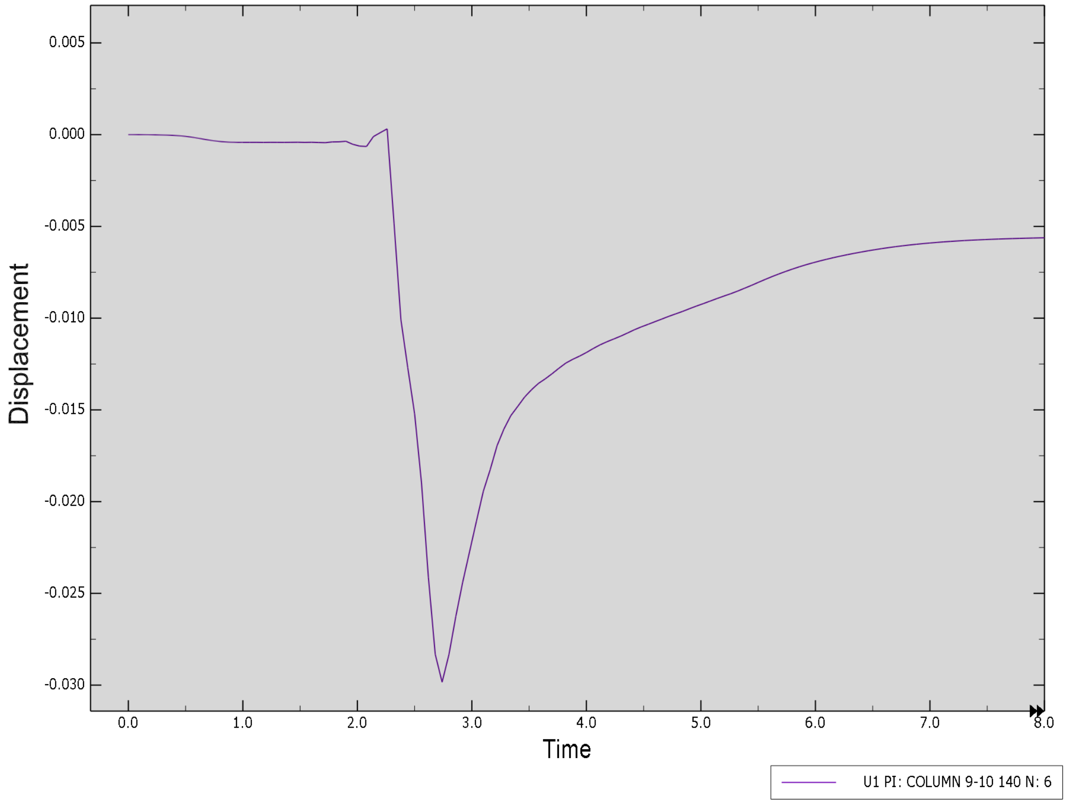

Figure 6.14 - Horizontal displacement, mid point of Column 9-10, negative away from core.

The reaction force between the end of the concrete floor slab and the core of the building is shown in Figure 6.15. Note that the reaction shown is per node, and that there are two nodes at the end of the slab – therefore the total reaction will be double this value. The graph demonstrates that after an initial period of compression (a negative reaction towards the core) the floor slab goes into tension, which gradually reduces. In conjunction with Figure 6.14 and Figure 6.15, this indicates that the initial expansion of the floor slab pushes out the exterior beams before weakening, deflecting further downwards and developing a degree of Tensile Membrane Action to support the deflected slab.

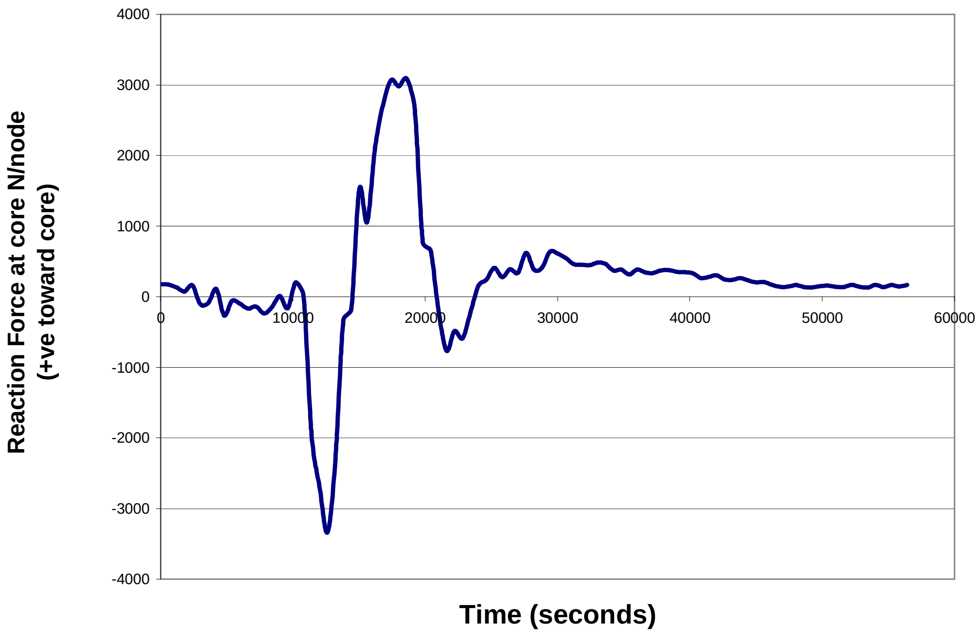

Figure 6.15 – Reaction between core and concrete beam

6.6.2 Variation in Top Support condition

As mentioned in Section 6.2, the member connecting the 16th floor to the upper transfer floor is unknown, as is the capacity of this connection. In order to examine these two variations, and to allow study of the mechanisms of redistribution within the structure, two potential restraint conditions are used for the top of the steel column.

- Fully restrained against rotation and displacement. (**A**)

This models the case where a significant structural member is directly attached to the upper transfer floor, and can transfer load to it.

- Restrained against rotation and horizontal displacement, but free in vertical displacement (**B**)

This represents a case where either no top transfer floor is present, or the member connecting the 16th floor to the top transfer floor is unable to resist significant vertical loading. This method was chosen rather than removing the entire topmost section of the column for ease of modelling.

Using these two modelling cases, it is possible to determine if there is a degree of 'cantilever action' due to the stiffness of the floor members and beams, allowing load from fire weakened members to be redistributed to the core. Alternatively it may be necessary to have a connection to a substantial and highly fire resistant structural member to carry the redistributed load.

As can be seen from Table 6.4, the model is capable of withstanding the fire if the steel is fully insulated regardless of the top support condition.

In the case of support condition **A**, the model does not collapse when any column is uninsulated, as the floors above the uninsulated column are supported by hanging from the upper transfer floor.

This also holds for two adjacent column sections being uninsulated. This result led to investigation of a case where all columns were uninsulated. Again this did not lead to collapse. This will be discussed further in Section 6.6.3.

Where support condition **B** is specified and there are one or more uninsulated column sections, the model exhibits collapse in all but one case (Figure 6.16). As can be seen from Table 6.4 and Figure 6.17, collapse is prevented when the column from Floor 15 to Floor 16 is uninsulated.

The top of the column is still restrained against rotation and horizontal deflection. Therefore some degree of cantilever action is taking place, with the stiffness of the column above the 16th floor and the 16th floor itself being sufficient to support the limited load associated with one floor and preventing collapse.

This is an unrealistic structural arrangement, as it relies on a structural member above the 16th floor that is unlikely to be present with the specified restraint conditions. As it is unclear from the Windsor Tower plans what member was actually present in this location, this system of load redistribution can be regarded as very unlikely to occur.

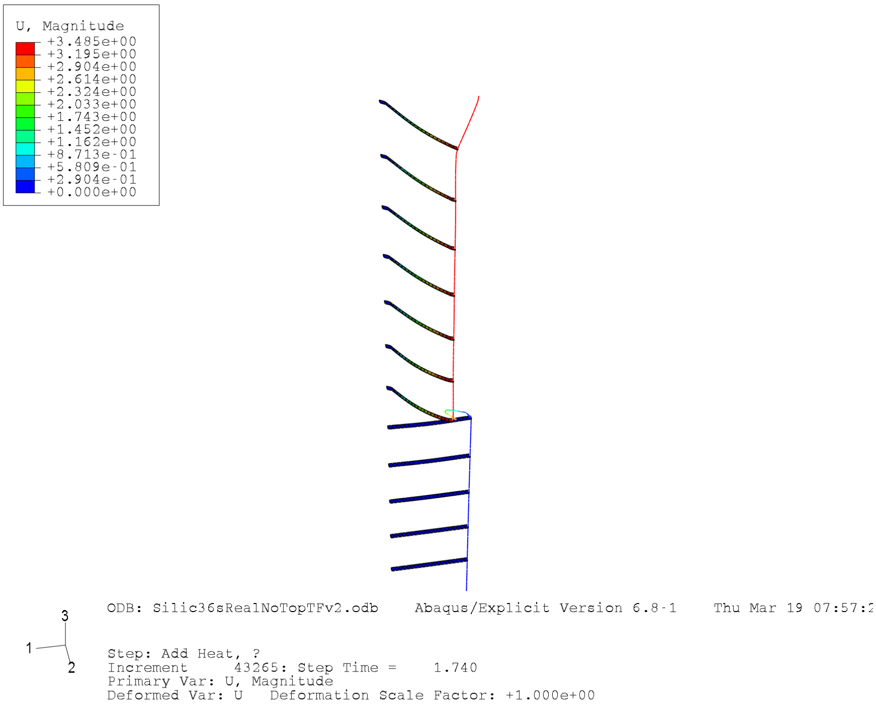

Figure 6.16 - Collapse, column 9 to 10 uninsulated, No vertical restraint at top

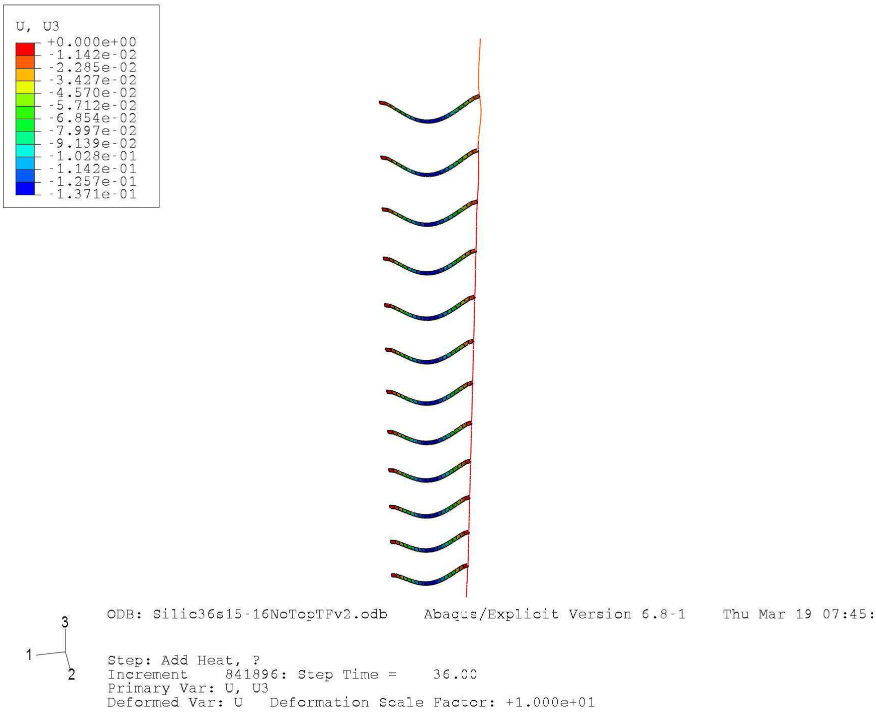

Figure 6.17 – Final state, column 15 to 16 uninsulated, No vertical restraint at top

The general indication is that, in a case where fire protection is limited or not present but load redistribution to a major structural member such as a transfer floor is possible, collapse will not occur.

This may have been the case in the Windsor Tower, however the unknown properties of the topmost section of column also indicate that further redistribution methods may be important, for example through the floor to either side of the section examined rather than by cantilever action.

6.6.3 Variation in fire

As shown in Table 6.4, various fire cases were considered.

The first case studied assumes that fire compartmentation restricted the fire to the 9th floor, and hence the column between Floors 9 and 10. Fire was also applied to the top of Floor 9 and the bottom of Floor 10.

The fire protection on the column and the top restraint conditions were varied. It was found that fire protection prevented collapse regardless of top restraint condition.

Where no fire protection was present on the column between the 9th and 10th floors, collapse did not occur in support condition **A**, as would be expected. When the top of the column was defined to have support condition **B**, collapse occurred (Figure 6.18).

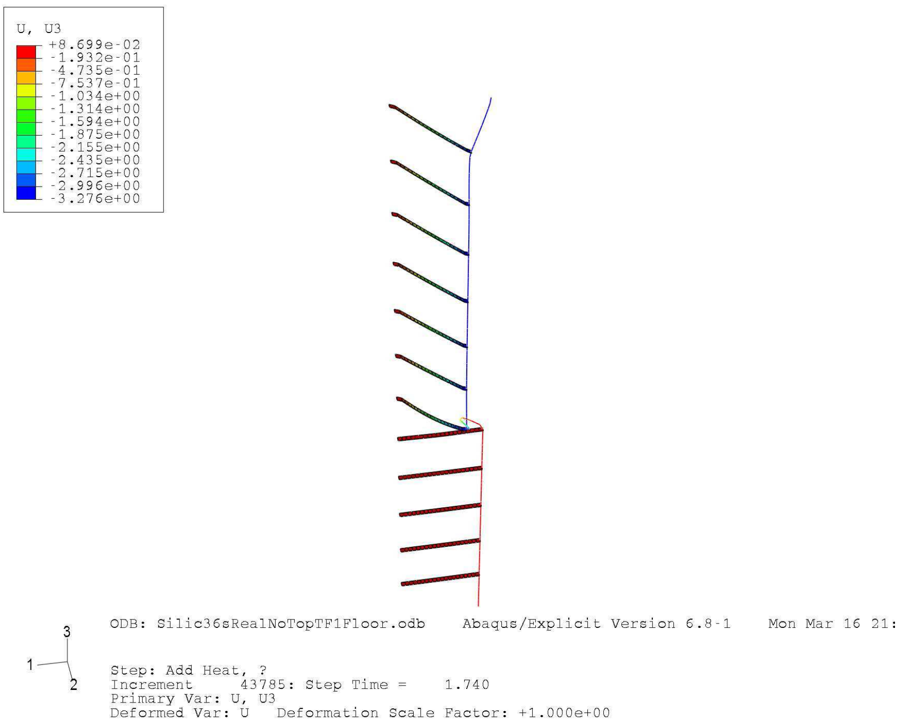

Figure 6.18 - Collapse, column 9 to 10 uninsulated, No vertical restraint at top, fire on 9th floor only.

This demonstrates that regardless of the success of fire compartmentation failure will occur if an uninsulated column is exposed to fire and no major structural member is available for load transfer.

The remaining cases varied the rate of fire spread through the building. In all of these cases support condition **A**, fixed at the top of the column, was used as it has been shown that this is necessary for models with uninsulated column members to withstand collapse. All columns were uninsulated. In the case of downward spread, the fire was first applied to the top of Floor 16 and then progressed downward. In the case of upwards spread, the fire was first applied to the bottom of Floor 5.

In the case of fire spread down the building at the rate of either 15 or 30 minutes per floor, no collapse was observed (Figure 6.19).

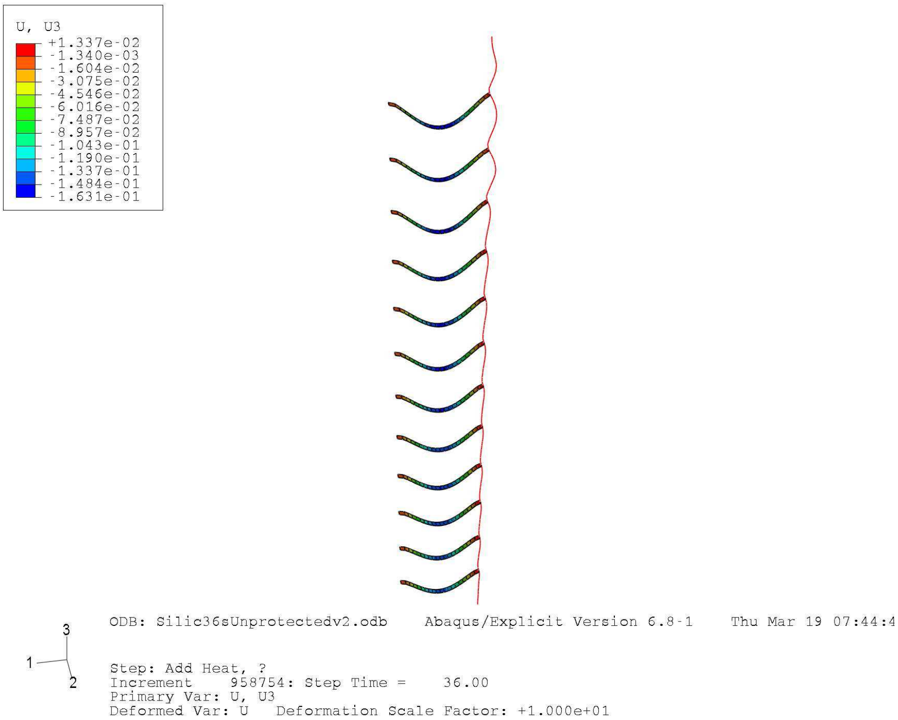

Figure 6.19 - Final state, all columns uninsulated, vertical restraint at top, 30 minute downward spread

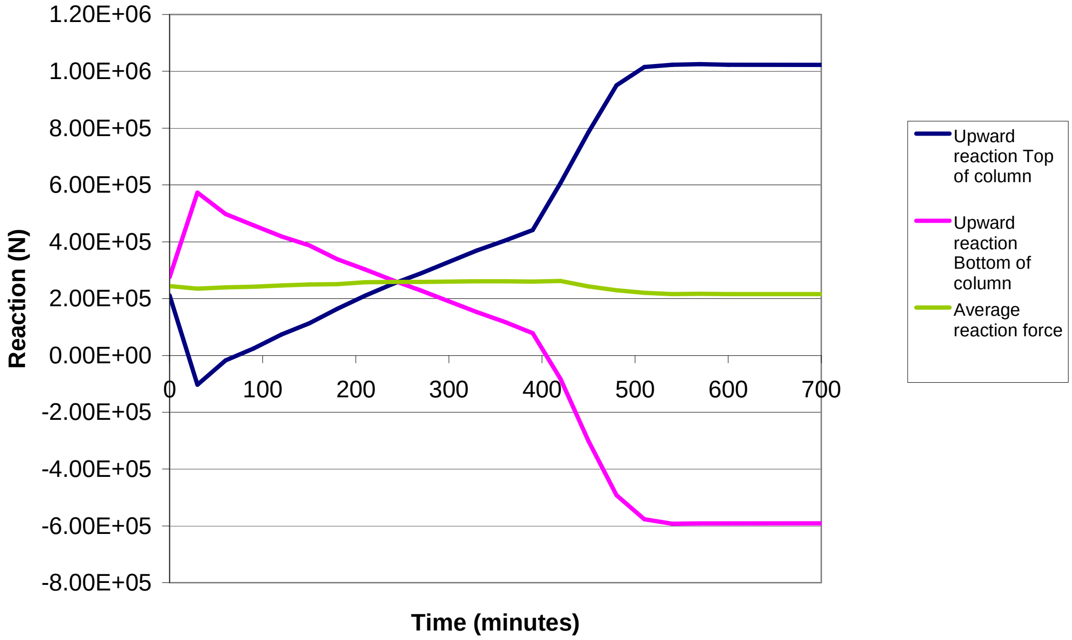

Figure 6.20 - Reaction forces at column ends, 30 minute downward spread

Analysis of the reaction forces at the top and bottom of the column (Figure 6.20) shows an initial increase in upward reaction at the bottom and downward reaction at the top. This can be attributed to the expansion of the steel members increasing the compression on the column. From this point, the upward reaction of the bottom support begins to decrease at the same rate as the upward reaction of the top support increases.

This is attributed to the steel members above the current fire floor recovering structural strength due to cooling, and therefore being able to support the load from the floors below in tension. As the uninsulated steel cools to ambient temperature rapidly, it will increase further in strength as the fire progresses down the building and therefore be able to support the load from an increasing number of floors. This is shown in Figure 6.21 and Figure 6.22.

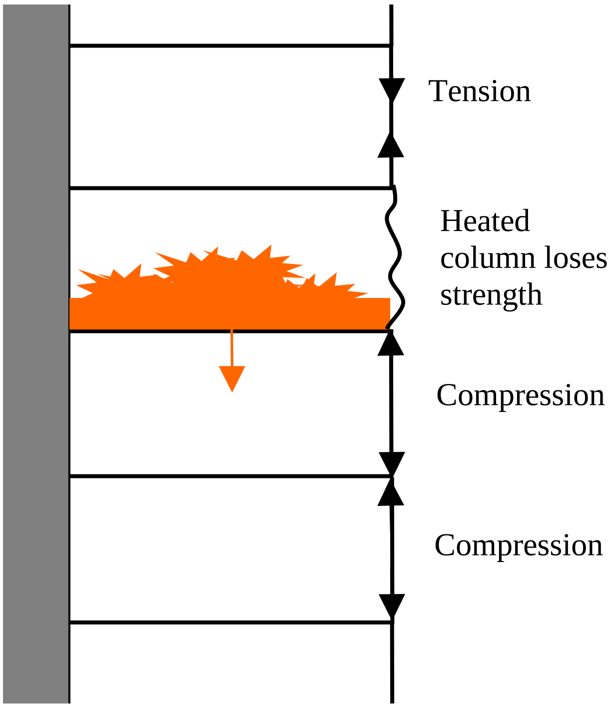

Figure 6.21 Fire begins downward spread

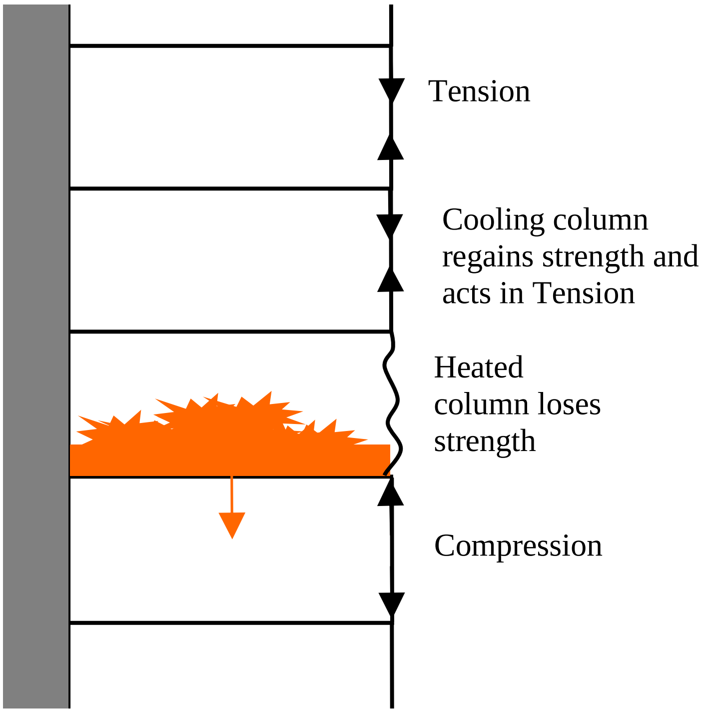

Figure 6.22 Load path transferring from Compression to Tension

The fire reaches its peak temperature between the bottom transfer floor and Floor 5 at 420 minutes, the same time that the bottom support reaction begins to act downward (i.e. in tension). Immediately before this, virtually all load has been transferred to the top support as the bottom column will have little strength. As the steel begins to cool, it contracts. This results in an increase in the downward force from the bottom support, matched by an increase in the upward force from the top support.

By the end of the fire, the structure is hanging from the top transfer floor rather than the majority of the load being supported by upward reaction from the bottom transfer floor. The mechanism for a downward spread rate of 15 mins is similar (Figure 6.23).

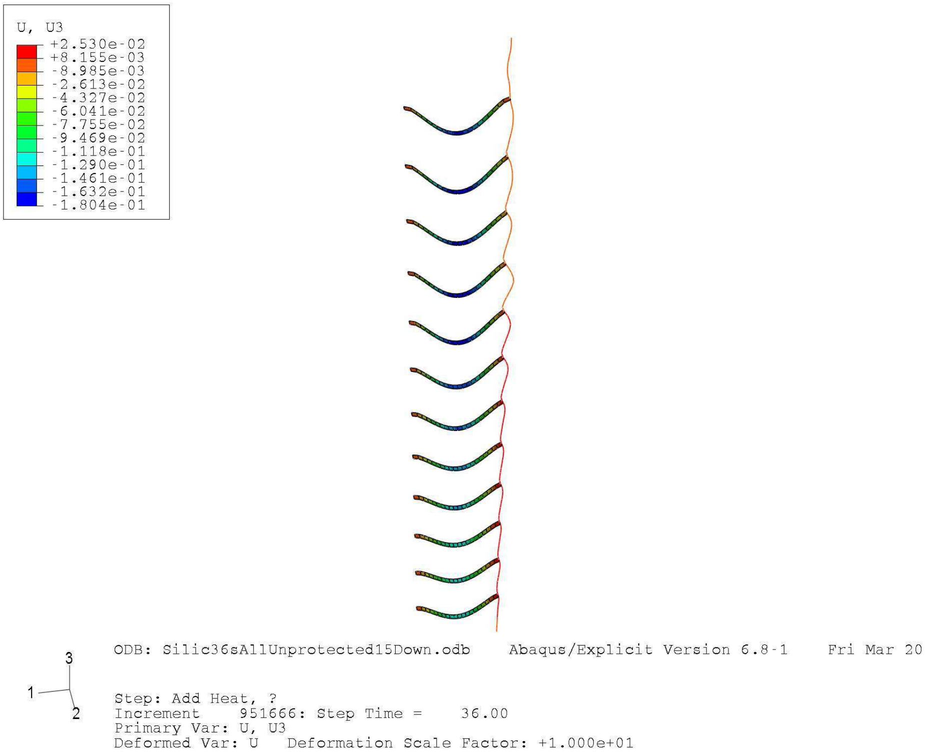

Figure 6.23 - Final state, all columns uninsulated, vertical restraint at top, 15 minute downward spread

The final two analyses are used to examine the effects of upward spread and extremely fast fire spread. The rates considered are 15 minutes per floor upward, which was identified as the most severe fire spread rate in Chapter 5, and applying the fire to all floors at time zero (the 'instantaneous' model).

In both cases, the model collapsed even though it was restrained at the top. In the case of the 'instantaneous' model, this happened very quickly after the model started.

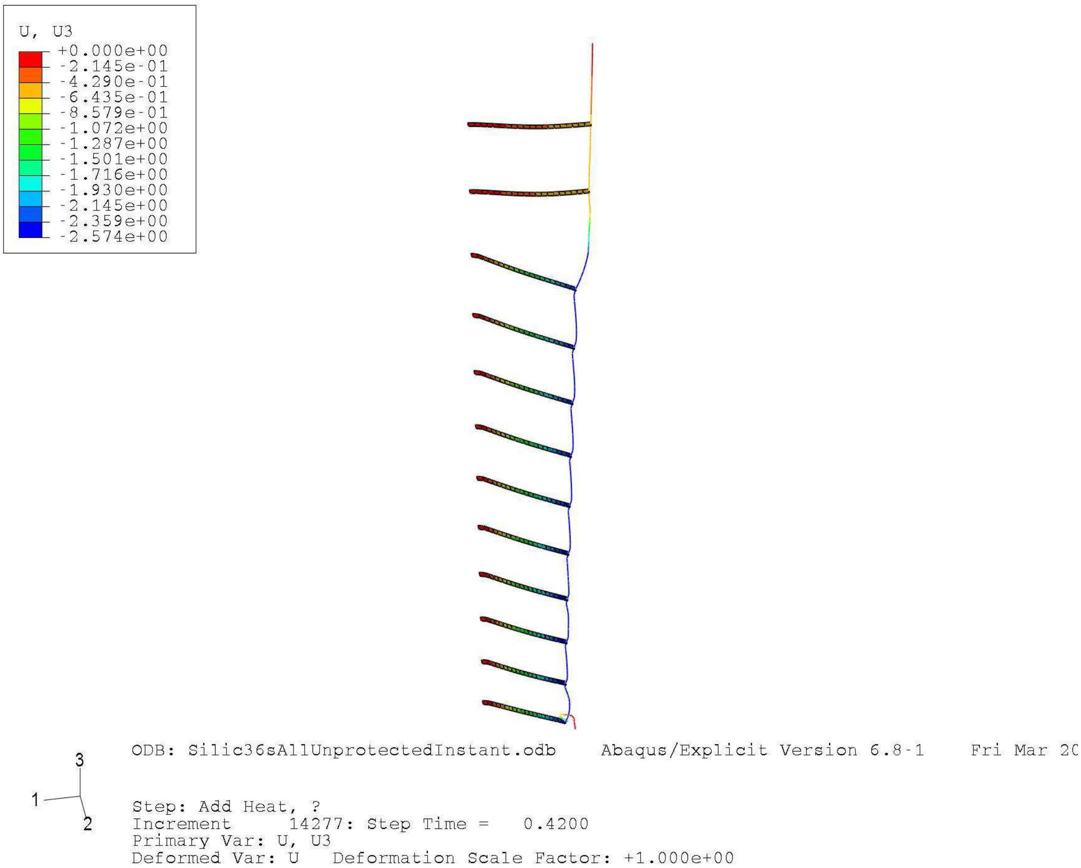

Figure 6.24 - Collapse, all columns uninsulated, vertical restraint at top, fire on all floors simultaneously

As can be seen from Figure 6.24, the two uppermost floors have suffered less deflection while the lower section of the model has collapsed as a unit. This may be due to some small degree of 'propped cantilever action' due to the stiffness of the top column section and the top two beams interacting with the rotational and translational restraints on the top of the column, in a manner similar to that discussed in Section 6.6.2. The collapse appears to have initiated at the point of highest loading, i.e. the column between the bottom transfer floor and Floor 5.

The model using an upward spread rate of 15 minutes per floor also undergoes collapse (Figure 6.25).

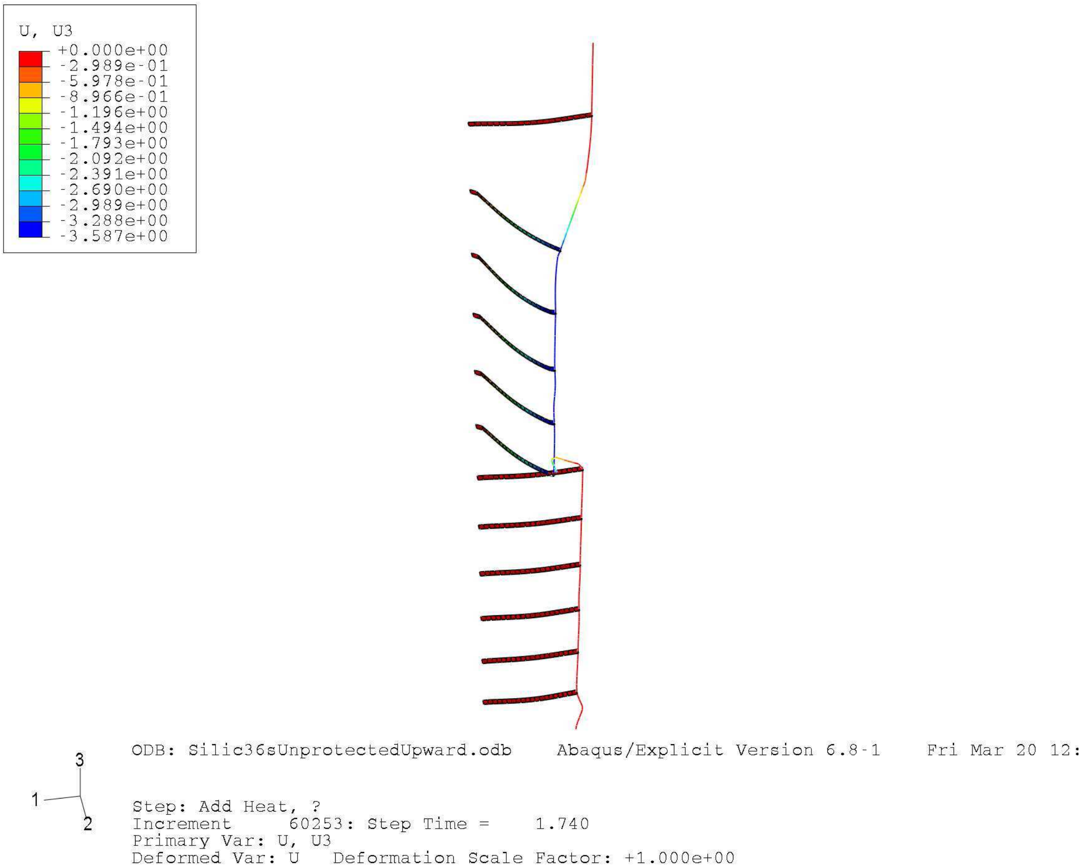

Figure 6.25 -Collapse, all columns uninsulated, vertical restraint at top, 15 minute upward spread

The mechanism involved in this collapse is obviously more complicated than in the 'instantaneous' model, as failure does not occur at the lowest steel column and the fire continues for a substantial length of time.

As can be seen in Figure 6.28, immediately after the fire between the lower transfer floor and Floor 5 breaks out, much of the load is transferred to the top support. The amount of load imposed on the top support gradually decreases as the fire travels up the building and the steel columns lose strength and then return to a compression mode supported by the bottom support (Figure 6.26, Figure 6.27).

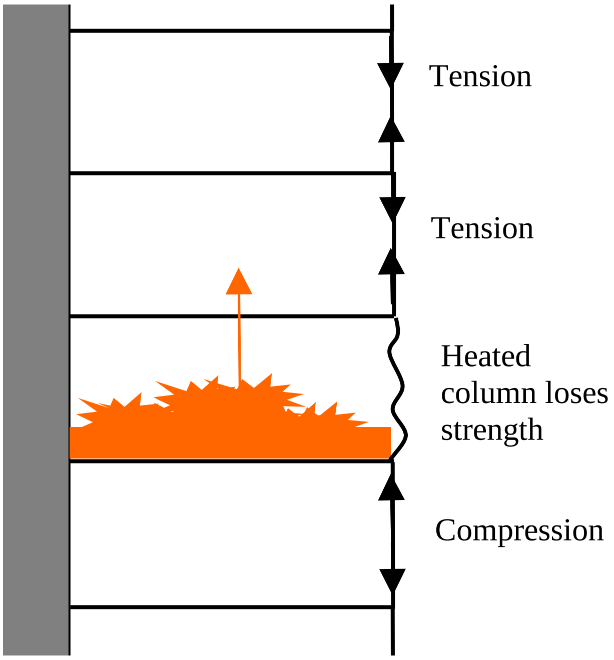

Figure 6.26 Fire begins upward spread

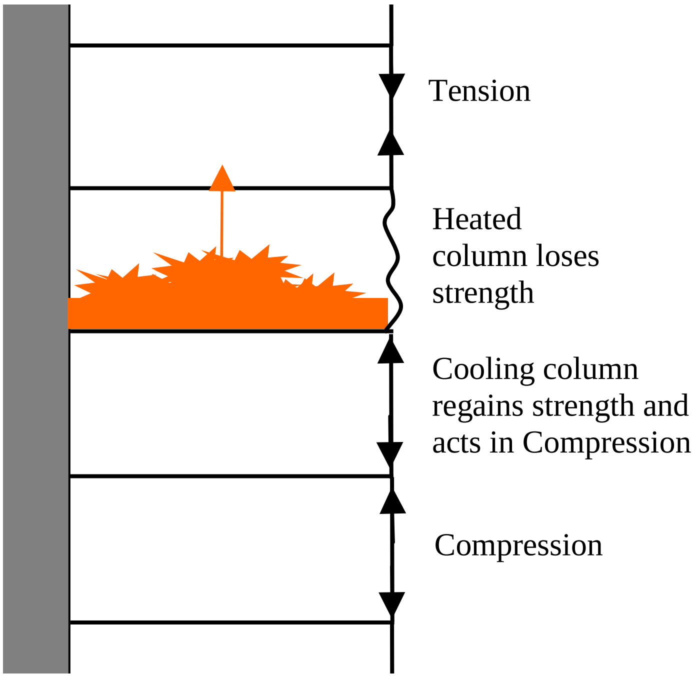

Figure 6.27 Load path transferring from Tension to Compression

The force on the bottom support continues to decrease after the initial point as the steel strength reduces, until a time of 90 minutes at which the support reaction reaches its lowest value. The lowest column begins to cool and can support an increasing amount of load, in line with the load reduction to the top support.

The exact mechanism by which these reactions are interacting is not known, nor is the degree of influence of steel expansion. This would be a useful area for further study. The concrete beams may support some of the load which has been shed from the top support but has not yet passed to the bottom support.

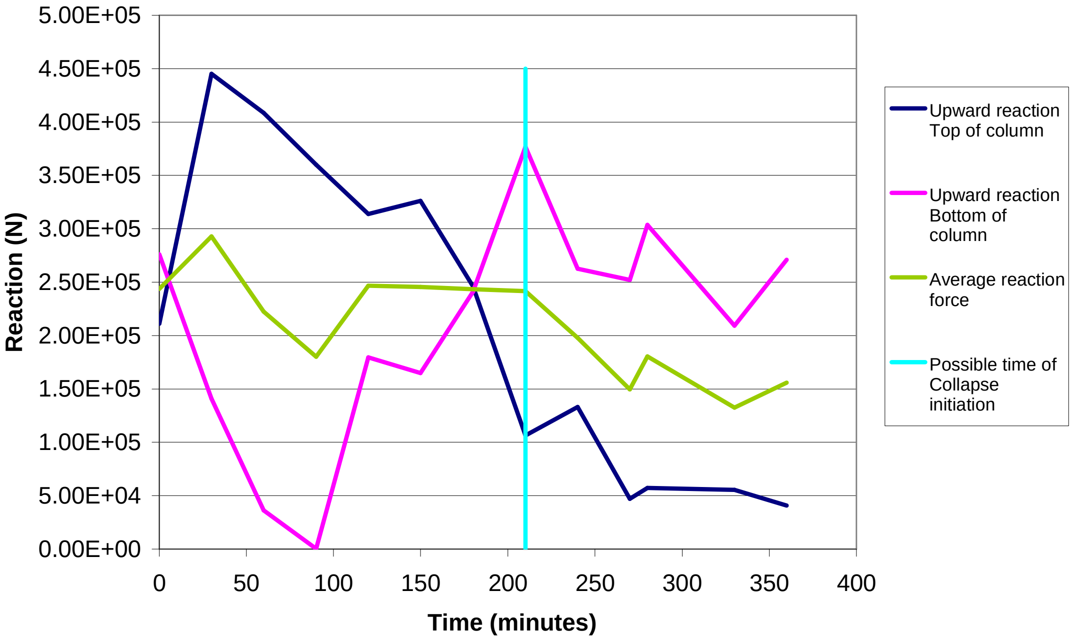

Figure 6.28 Reaction forces at column ends, 15 minute upward spread

The time of collapse initiation is around time 210 minutes from the sudden loss of reaction force at the bottom support. From this point, Floors 11-15 undergo collapse as can be seen in Figure 6.25.

Around 210 minutes, the fire on Floor 15 reaches its peak temperature and heats the column between Floor 15 and Floor 16 reducing its strength significantly. The fire on Floor 10 has just reached extinction and the steel between Floors 10 and 11 will still be cooling, though much of its structural strength will have restored.

As a proportion of the load is always held by the top support acting in tension, it is notable that collapse is initiated from the middle column of the building (Floors 10 to 11) upward. This implies that the majority of the load below this point is usually held by the bottom support, while the load in the upper half is held by the top support. The column between Floors 14 and 15 will therefore be one of the most highly loaded in the building.

When the structural capacity of the column between Floors 14 and 15 is reduced by heating, the lower columns are still weakened by fire and therefore appear not to be able to withstand the additional loading. This initiates collapse.

Again, some degree of 'propped cantilever action' between the topmost floor and the topmost beam appears to take place.

In summary:

- While fire is spreading *down* the building, the progressive conversion of columns from compression to tension, with associated increase in loading, acts at the same time as the columns above have their strength restored by cooling.
- While fire is spreading *up* the building, although the load also converts from tension to compression a significant proportion of it is still supported by tension from the top. This may be due to the initial increase in tensile loading at the outbreak of the fire. As the fire travels up the building conversion from a tensile mode to a compressive mode does not occur sufficiently to prevent collapse.

It can be seen that there are fire scenarios which will lead to collapse of a building which does not have sufficient fire protection, regardless of the support conditions. It appears that the resistance of the building is dependent on allowing the structure sufficient time to restore its strength during cooling in order to allow load transfer. Therefore fire compartmentation is necessary to either totally prevent the spread of fire, or to slow it sufficiently that the structure will remain stable.

6.7 Conclusions

It can be seen that, with the strength of concrete used and the spans involved, negligible cantilever action takes place to transfer load to the core of the building.

If the steel members of a building are sufficiently fire protected, the modelled structure is capable of resisting the fire load applied to it. It cannot be verified with certainty that the modelled structure behaved in the same fashion that the Windsor Tower did, as it is unclear what the top support condition of the Windsor Tower was. If the topmost column section of the Windsor Tower was not structural, it seems that a mechanism other than suspension from the upper transfer floor was responsible for the Tower’s stability. This would be a useful subject for further research.

The deflection of the uninsulated steel column between the Floors 9 and 10 in the model agrees with the observations from the 9th floor in the real building. This indicates that the model behaves in a similar fashion to the real structure when examining a key indicator.

When using the mechanism of load transfer to a strong structure which is not affected by fire, it is clear that the rate and direction of the fire spread are an important factor in allowing the overall structure time to restore its strength through cooling.

Therefore, the role of either full fire protection, or compartmentation which either totally stops or impedes the progress of a fire, is highly important in a structure of this type. Both are recommended.

It should be noted that in no case did the structure appear to fail due to any weakening of the concrete in the structure, although there were some large deflections and a large amount of repair work would be required.

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CHAPTER 7: CONCLUSIONS AND FURTHER RESEARCH

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7.1 Introduction

This chapter gathers the conclusions drawn during the course of this research project and suggests avenues for future research.

7.2 Conclusions

- A wealth of data is available on the performance of concrete as a material in fire, and the performance of individual structural members. However the performance of a largely concrete structure as a whole is less well understood.

- Given the relative lack of information available from large scale fire tests on concrete structures, a case study on the effects of fire on a real concrete structure is of particular interest.

- A significant quantity of data relating to the construction details, post fire photographs, investigative and eye-witness reports of the Windsor Tower Fire has been assessed in collaboration with the University of Cantabria (Spain). The Windsor Tower was therefore used as the basis of the case study to determine:

    - Fire movement within the building.
    - Behaviour of the structure during the fire.

- The Windsor Tower, in those areas where fire protection was installed, performed remarkably well when subjected to a prolonged fire, however some areas did undergo collapse. Two collapse mechanisms are proposed, along with potential methods of fire spread up and down the building.

- Due to the level of assumptions required for both the fire and structural models, it was impractical to carry out a totally forensic model of the Windsor Tower. However, the performance of a ‘Windsor Tower-like’ structure can be investigated using modelling of both fire and structural behaviour. The multiple floor nature of the fire is of particular interest.

- An intuitive research method, ‘the worst credible case’ was used to estimate unknown concrete material properties.

- The fire modelling results are shown to approximate the real world fire.

- The 'simplified ISO-based curves' provide a reasonably accurate temperature representation of the fire in the Windsor Tower, comparable to the results from CFD and real world observations, while reducing the erroneous impression that it is possible to establish a 'definitive' fire that occurred.

- Shell elements are used in the Finite Element package to model concrete members as they allow accurate representation of deflected shapes and the modelling of reinforcement. They also have multiple temperature definition points through their depth facilitating representation of the thermal response.

- The specific collapse mechanisms examined all involved the perimeter area of slab acting as a cantilever due to the weakening of exterior steel columns by heating. The area of the tower studied in detail was between two concrete 'Transfer Floors'.

- When a section of the steel column is unprotected, it quickly loses its ability to support load. In the case where no top transfer floor is present, the load must then be supported by the stiffness of the concrete floor members and the steel columns above the weakened member. Even if the connection between the members is very stiff, the floor members themselves must be strong enough to resist the downward force at their end, acting as a 'multiple-floor cantilever'.

- The analysis was designed to demonstrate whether the stiffness of the concrete members together with surrounding cooler steel members was sufficient to transfer load to the core of the building. Alternatively a substantial transfer structure, unaffected by fire, would be required to prevent collapse by 'propping' the end of the 'multiple-floor cantilever'.

- It was concluded that, with the strength of concrete used and the spans involved, any 'multiple-floor cantilever' effect which takes place is insufficient to transfer load to the core of the building when unprotected steel members are present, and that a substantial structural element which remains unaffected by fire must be present to prevent collapse.

- If the steel members of a building similar to the Windsor Tower are sufficiently fire protected, the modelled structure is capable of withstanding the fire applied to it.

- The deflection of the uninsulated steel column between Floors 9 and 10 in the model agrees with the observations from the 9^{th} floor in the real building. This indicates that the model behaves in a similar fashion to the real structure when examining a key indicator.

- The Windsor Tower Fire spread to multiple floors, both up and down the building. This is of particular interest in those modelling scenarios where all structural steelwork is left uninsulated but supported at both the top and bottom by the 'Transfer Floors'.

- In a structure with uninsulated steel members, the rate and direction of the fire spread are an important factor in allowing the structure time to restore its strength through cooling. This allows load to be transferred to stronger structural members which are unaffected by fire, i.e. a transfer floor.

- Peak temperatures developed in the fire are to a degree uncertain but may not be of great importance in terms of failure mechanisms. A hotter fire would have a similar effect in reducing the strength of the steel structure compared with a cooler fire. It was shown that the concrete structural members were able to withstand a fire of higher temperature.

- A fire of longer duration might delay the cooling of steel structural members, and therefore prevent their strength recovery and redistribution of loading. Again the concrete structural members were shown to be able to withstand fires of greater duration.

- Therefore the strategies of either full fire protection, or sufficient compartmentation which either totally stops or impedes the progress of a fire, are vitally important in a structure of this type. Both are recommended.

- In no case did the structure appear to fail due to weakening of the concrete, although there were some large deflections and a large amount of repair work would be required.

7.3 Proposals for Further Work

Further analysis and refinement of the concrete models used would be a valuable area of further research, particularly in respect to the definition of concrete tensile strength.

The damage evolution parameters of the Concrete Damaged Plasticity model were excluded from these analyses, and would be useful to add to the model.

It cannot be verified with certainty that the modelled structure behaved in the same manner that the Windsor Tower did, as it is unclear what the top support condition of the Windsor Tower was. If the topmost column section of the Windsor Tower was not structural, it seems that a mechanism other than suspension from the upper transfer floor was responsible for the Tower's stability. Horizontal expansion of the model and therefore inclusion of the structural effects of the edge beam, secondary beams and floor slab would allow examination of other load redistribution mechanisms. Expansion of the 3D model would be of particular interest on the 9th floor where structural steel was fire protected on two sides of the building.

Variation in the concrete strength, reinforcement arrangement and quantity and steel member sizes could be used to determine the concrete elements that would be necessary to support action as a 'multi-floor cantilever' without support from the transfer structure.

Variation in member sizes can also be used to examine the effects of fire on other areas of the Windsor Tower, as the reinforcement arrangement in the Windsor Tower's concrete waffle slabs varied from area to area, as did the size of the external steel columns.

Fires which reach higher temperatures, longer durations, or both, could be applied to the structure to determine if a more severe fire scenario will result in a different collapse behaviour.

The fire could be applied to the members in a more localised manner, possibly directly coupled with the output of CFD models (Welch, Miles et al. 2008). This may lead to a more or less severe fire scenario due to localised extinction (Rein 2007b).

Further analysis of the collapsed area of the upper tower which included the concrete columns would be of great value (Figure 3.10) and would focus on identifying the mechanism by which collapse occurred.

When analysing a fire acting both above and below a concrete slab it will often be the case that different temperatures are imposed on either surface. Further analysis using temperature variation between the top and bottom of a slab would be valuable.

Further analysis of the mechanisms by which loads in the exterior steel columns switch from acting in compression to acting in tension, and vice-versa, would be valuable.

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