PIT Project

Behaviour of steel framed structures under fire conditions

MAIN REPORT

June 2000

THE UNIVERSITY OF EDINBURGH
School of Civil & Environmental Engineering
King's Buildings
Edinburgh

Contributors

University of Edinburgh

Dr AS Usmani
Prof DD Drysdale
Prof JM Rotter
Dr AM Sanad
Mr M Gillie
Ms S Lamont

British Steel (now Corus)

Dr MA O'Connor
Dr D O'Callaghan

Imperial College of Science, Technology and Medicine

Dr AY Elghazouli
Dr BA Izzuddin
Mr AJ Richardson

Building Research Establishment

Dr CG Bailey

Steel Construction Institute

Mr GM Newman

EXECUTIVE SUMMARY

Steel beams in standard fire tests reach a state of deflections and *runaway* well below temperatures achieved in real fires. In a composite steel frame structure these beams are designed to support the composite deck slab. It is therefore quite understandable that they are fire protected to avoid *runaway* failures. The fire at Broadgate showed that this didn't actually happen in a real structure. Subsequently, six full-scale fire tests on a real composite frame structure at Cardington showed that despite large deflections of structural members affected by fire, *runaway* type failures did not occur in real frame structures when subjected to realistic fires in a variety of compartments.

This project was the first major effort to *understand* this behaviour using computational models of the Cardington fire tests. A full explanation of the mechanics that are responsible for the robust behaviour of unprotected composite frames in fire has been achieved and will be presented in detail in this report. Reaching this new understanding has been a laborious process, and numerous blind alleys had to be investigated along the way, however *obvious* the answer may now seem to the researchers involved in this project. It is possible that the conclusions will not seem obvious to others who have not been directly involved, however considerable effort has gone in to presenting the results of the project to provide as much detail as possible. Approximately 40 supplementary reports and over 10 technical papers have been written and appear as an appendix to this report. This amount of work has ensured that the conclusions presented have been verified by a number of independent approaches. Mutually reinforcing arguments were developed from the results of different computational models, application of fundamental mechanics and the analysis of test data. It is therefore with a great deal of confidence that these findings have been presented for close scrutiny by the profession. Once this new understanding of structural behaviour in fire is widely disseminated, discussed and understood, the way will be clear for completing all the other tasks which are necessary for full exploitation of the knowledge gained. This will lead to safer, more economic and rational design of steel frame structures for fire resistance.

The main objective of the project can be stated as :

> *'To understand and exploit the results of the large scale fire tests at Cardington so that rational design guidance can be developed for composite steel frameworks at the fire limit state'*

This objective was achieved by:

- Developing rigorous, robust finite element models using state of the art, commercial finite element modelling programmes. These models were thoroughly validated using the results of the major Cardington fire tests.
- Developing properly validated, computationally efficient, simplified models to accurately predict the behaviour of multi-storey steel frame structures under fire conditions. These models were developed

using both commercial and research finite element packages by an iterative process of comparison with rigorous models and experimental data.

- Developing different models of the same phenomena independently, in parallel to test model sensitivities and modelling assumptions.
- Developing methods of post-processing results so that the underlying structural mechanics can be more easily understood.
- Using the simplified models to conduct parametric studies to explore changes of structural behaviour.
- Checking the consistency of model results with the fundamental principles of structural mechanics by developing appropriate theory.

The key findings of the project can be summarised as:

- The composite steel framed building tested exhibited inherently stable behaviour under the tested fire scenarios due to the highly redundant nature of the structural form.
- This behaviour is characterised by several thermo-mechanical phenomena, which interact. This complex interaction is highly dependent upon the structural layout and the thermal regime of the fire compartment considered.

Identified thermo-mechanical phenomena include:

- restrained thermal expansion leading to buckling in both beam and slab structural elements
- thermal bowing due to differential temperatures in the main structural elements initially and due to through depth thermal gradients in slab later.
- induced P-δ moments in highly restrained compartments, due to high axial forces and large deflections due to large thermal straining.
- material degradation leading to reducing forces in the steel members.
- alternative load carrying mechanisms due to membrane stiffness of the concrete slab
- the behaviour indicates a number of competing effects in real structures which counter conventional understanding of structural behaviour in fire based upon traditional design methods

The main implications of this new understanding are:

- As the behaviour of composite structures is radically different from the present design philosophy, a new design philosophy is required based on a new definition of the fire limit state for this class of highly redundant structure.
- The complex nature of the structural behaviour will move the onus for the fire design from the architect to the structural engineer.
- Unprotected steel beams for composite steel structures are attainable provided that the robustness of the beneficial mechanisms and the quantification of the detrimental mechanisms can be ensured. This robustness can be ensured by a combination of good detailing practice to enable beneficial mechanisms to materialise and further work to identify worst case scenarios for structural layout and thermal regime.

It is recommended that:

- Worst case scenarios are identified by carrying out comprehensive parametric studies on composite structures under fire. This study should consider the effect of compartment size, structure geometry and fire scenario including the effect of whole compartment fires (post flashover) and spreading fires.

TABLE OF CONTENTS

1 INTRODUCTION

1.1 Background to the problem (see report ED1)

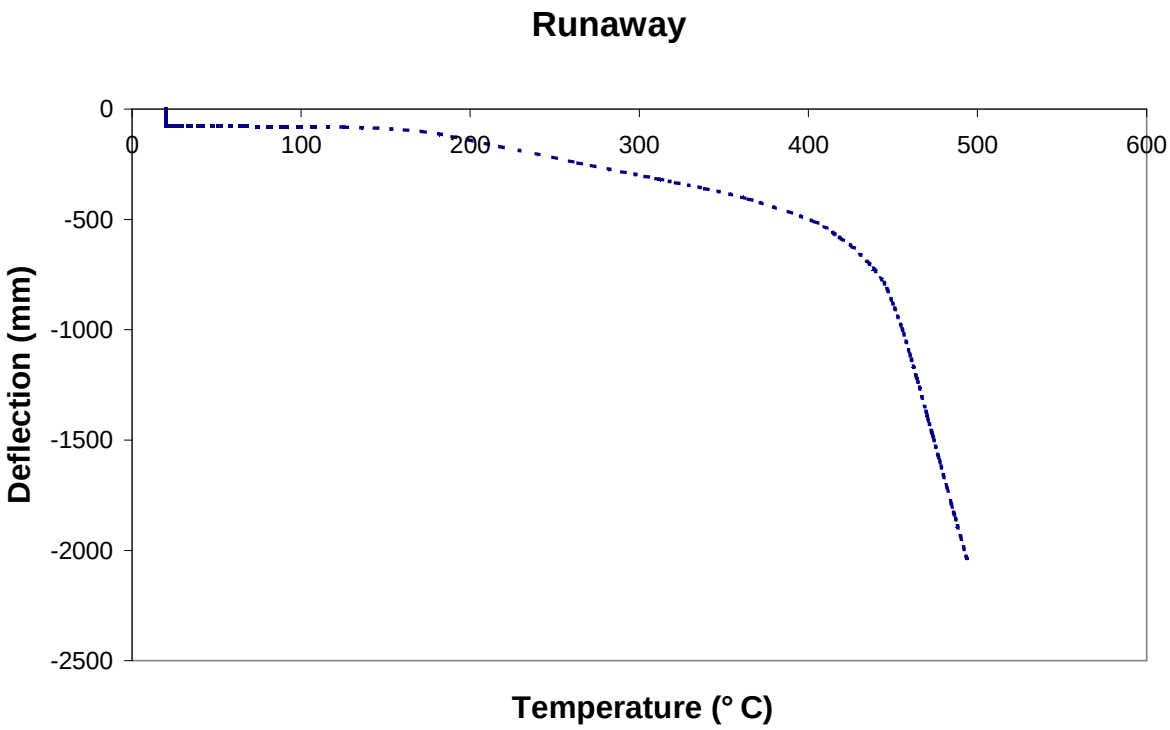
The motivation for this project stemmed from the widespread realisation in the structural engineering community that fire resistance of structures based upon single element behaviour in standard fire tests was a gross simplification of the real behaviour of these elements, when they acted as part of a whole structural system. It is accepted that conditions in standard fire tests have little relevance to the actual conditions prevailing in real fires. These are simply proving tests for regulatory purposes. However, this is not the key reason for the lack of relevance of standard fire tests to real structural behaviour. The concept of isolated structural elements may seem appropriate where fire in a compartment effectively attacks only the individual structural members nearby. However, in this model no account is taken of the interactions that inevitably occur with the surrounding structure. Where the complete structure is large and redundant, these interactions can completely change the structural response and invalidate the design assumptions.

The distinctions between determinate and indeterminate structures go back to the basics of a structural engineers education (and therefore apologies to readers who may find this too elementary, but it is felt that in the current context these distinctions are worth restating). A determinate structure is one in which the pattern of internal forces and stresses can be determined using equilibrium considerations alone and there is only one load path available for transfer of loads from the point of loading to the supports. An isolated beam between simple supports is an example of such structures. An indeterminate (or redundant) structure is defined as one in which the pattern of forces and stresses within the structure cannot be determined by the conditions of equilibrium alone, but depend in addition on the relative stiffness of parts of the structure and the conditions of displacement compatibility. These structures typically have multiple load paths available, and considerable redistribution of loads can take place even after several of the load paths reach capacity.

Under collapse conditions, determinate and redundant structures are more sharply differentiated. In determinate structures when the most highly stressed region reaches the local strength or, in other words, when the "single" load path reaches capacity the structure can no longer sustain any further loading and collapse occurs. Fire tests on isolated members match this condition and are characterised by ‘runaway’ failures (see **Figure 1.1**) as there are no means of arresting the runaway deflections (means that are quite evidently present in real structures). The main difference in the case of standard fire tests is that the load remains constant while the capacity of the load path reduces to a point where it is exceeded by the load. This leads to a displacement criterion being chosen to determine failure in fire tests as increasing displacements coincide with the overall failure of the member. By contrast, provided it has adequate ductility and does not suffer from instability, the redundant structure can find different load paths and different load carrying mechanisms by which to support additional load when its local strength is reached at a single location. Where a structure is highly redundant (as most modern steel framed structures indeed are), there are many alternative load paths and large deformations can develop without necessarily a loss of strength, and failure must be defined in a different way. This problem in not unique to fire: researchers in pressure vessels and rectangular storage structures are also trying to find new failure definitions.

Complex structural interactions taking place in framed structures during fire lead to extensive redistribution of loads. This phenomenon creates sufficient reserve capacity to allow most structures to survive fires with little structural damage. As this behaviour results from the

interaction of all or most of the structural elements acting as a unit, it is therefore necessary to consider a structure as an integrated whole when its fire resistance is evaluated. Although this fact has been recognised for some time (as demonstrated by the Broadgate Phase 8 fire of June 1990 [1] it was in the mid-90s that full-scale fire tests on realistic structural configurations were carried out at the large building test facility (LBTF) at Cardington. British Steel (now CORUS) completed a programme of four tests from September 1995 to June 1996 (see report ED1) and BRE carried out two further tests. These tests were designed to be complementary. The British Steel experimental programme is outlined in **Figure 1.2**. In terms of structural behaviour, the aforementioned tests provided unique data on the performance of steel frame structures when individual elements and combinations of elements are exposed to fire.

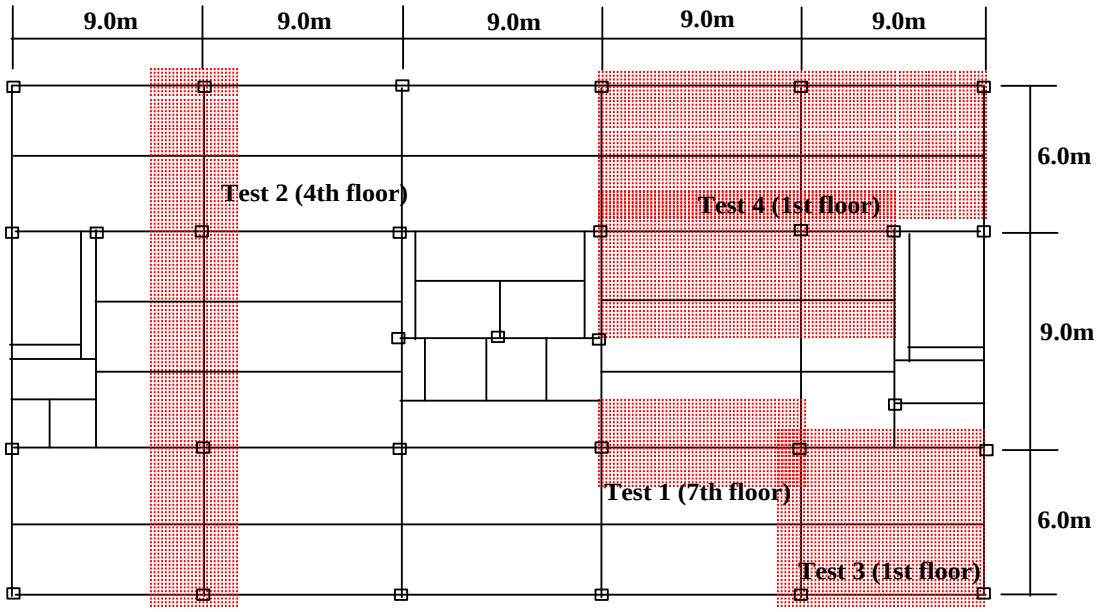


Runaway in a simple beam model
Figure 1.1

The response of structures to fire depends upon the materials used in construction, the response of individual structural elements and the structural system used to connect the elements. In addition to the above, the response of structures naturally depends upon the fire itself: its sequence, severity, spread and rate of development. The common assumption in structural fire resistance design is that fires will be localised through effective compartmentation. In the absence of this assumption, it becomes very difficult to predict the behaviour of the structure because the fire behaviour in such cases is very uncertain, therefore the heating regime and sequence in the structure is unknown.

A prime objective of the tests conducted at Cardington on the BRE eight-storey frame was to provide data to aid the development of understanding of the interactions between different structural mechanisms which determine the overall behaviour of composite steel frames in fire. For this reason, the mode of heating in the first three tests was not important other than for developing a uniform and controllable heating regime. To achieve this, in the first and second tests, gas fired furnaces were specifically designed and constructed around parts of the frame of particular interest. In the third test, heating the compartment using gas was not possible and therefore a natural fire

using wooden cribs was adopted as the heating medium. The temperatures within the compartment were controlled by an adjustable ventilation shutter. For the fourth test, a fire in a simulated office environment was created in order to impart upon the structure a fire, which could be related to a real scenario.



Layout of the British Steel Cardington Frame Fire Tests
Figure 1.2

The development of computer models is of fundamental importance to the successful application of the Cardington test results. The reason for this is that only six tests were performed on only one structural form. In order to formulate general recommendations for steel framed structures in fire it was essential to develop a reliable model, which could extend the database of test results to different structural forms and configurations together with a consideration of various fire scenarios.

Prior to this project, a variety of proprietary and commercial software packages had been used to model the Cardington tests, often with insufficient levels of independent verification. It was considered at the start of the project that none of the existing models had been developed sufficiently to encompass the key structural behaviour observed at Cardington. Consequently it was not possible to use the models then available to develop the understanding required, which was necessary to allow a full exploitation of the Cardington test data for meaningful and practical design guidance to be developed. The proposal for this project was therefore aimed at addressing the urgent need for a comprehensive and thorough numerical model of steel frame composite structures in fire in the wake of the Cardington tests.

1.2 The proposal

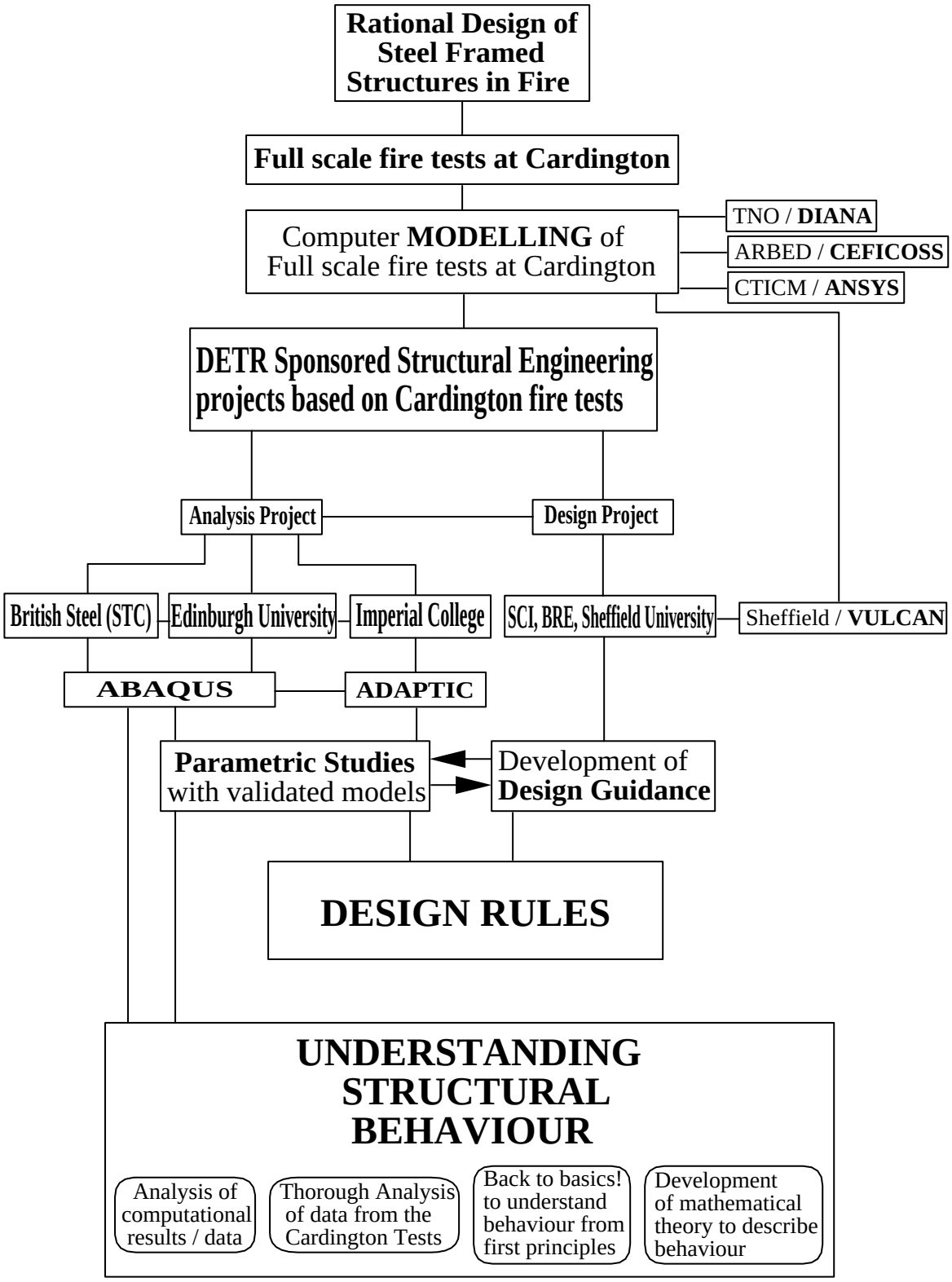
It was proposed to develop analytical tools that adequately modelled the structural behaviour of composite steel frames in fire. The main research team (EU, BS-STC, IC) would be able to offer a synergy of first rate expertise in the areas of structural behaviour and design, fire development, fire resistance of steel structures, numerical analysis techniques and the management of large projects. The project also involved a strong input from SCI and BRE who were to bring their experience in this field to bear by advising the modellers at frequent meetings. SCI was also responsible for a direct contribution to the project by posing relevant questions to the modellers about specific design

issues. Subsequently, SCI became the leaders of the 'Design Group' (with EU, BS-STC and IC being the 'Science Group') in an associated project on developing design guidance. BRE became responsible for performing an experiment on a slab to observe tensile membrane behaviour. **Figure 1.3** shows in a flowchart the interconnections between the groups involved and the broad strategy that was adopted in executing the project.

The bulk of the work in this project was devoted to developing computer models that would model the 'real' behaviour in the Cardington tests. In this project it was proposed at the start to use the commercial general-purpose solid and structural mechanics code ABAQUS. The choice of ABAQUS was made after careful reflection and consultation which highlighted the most important considerations as listed below.

- The analysis of a large composite frame structure for fire is a highly non-linear and complex problem in computational thermo-mechanics, and the task of writing ones own thermo-mechanical computer code single-handedly is very onerous. The expertise required to achieve this goal free of errors is not likely to exist in a single institution. The results of such endeavours can at best result in software that may be suitable for a limited combination of structural configurations and loading.
- The single most important reason why writing such codes is wholly unnecessary, is that there are several widely available commercial structural mechanics codes (such as ABAQUS), which have been very extensively verified against a huge range of problems, which should handle this task quite adequately. The use of powerful commercial codes allows one to have confidence that the chief programming challenges (geometrically and materially non-linear analyses with rigorous and reliable formulation) have been completed by others, freeing one to concentrate on the more critical engineering task of understanding the behaviour of the structure and the mechanics of the problem. Thus time and effort is not wasted on repeating work that has been done many times before by others in developing reliable computational methodologies.
- As mentioned earlier, models developed using these programs can be exchanged openly between research groups and all the assumptions and simplifications made are available for general scrutiny. This is not the case with most proprietary codes with limited access to the general community for scrutinising the code, notwithstanding the fact that this is a pointless exercise anyway given the existence of well-documented commercial codes.
- Most commercial codes (including ABAQUS) allow users to add to the software using user defined subroutines where this is desirable allowing any number of special cases to be handled (as has been done extensively in this project).
- The modelling skills developed using commercial codes are much more readily transferred to other researchers or to industry (if required). This is because it is much more likely for expertise to exist in the use of the most commonly used commercial codes than for using special purpose proprietary software (without an adequately well staffed user support structure).

There are however, a number of existing research codes which have been developed over many years and have been thoroughly tested and used by the authors themselves and other users. This was thought of as a very useful mechanism to provide an independent verification of the ABAQUS calculations and therefore the non-linear structural mechanics code developed at Imperial College (ADAPTIC) was used in this manner. Working independently, the team at Imperial College identified the same key structural responses and broadly similar behaviour using ADAPTIC as discovered by the teams at Edinburgh and British Steel (Swinden Technology Centre) using ABAQUS.



Flowchart showing interconnections between research groups

Figure 1.3

1.3 Summary of objectives, methodology and deliverables as proposed

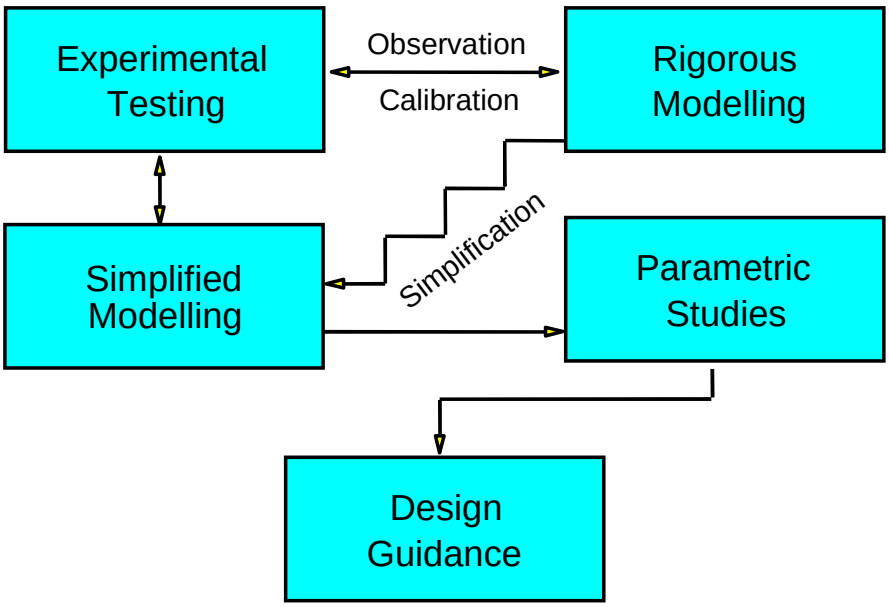
The prime objective of this project can be stated as:

> 'To understand and exploit the results of the large scale fire tests at Cardington so that rational design guidance can be developed for composite steel frameworks at the fire limit state'

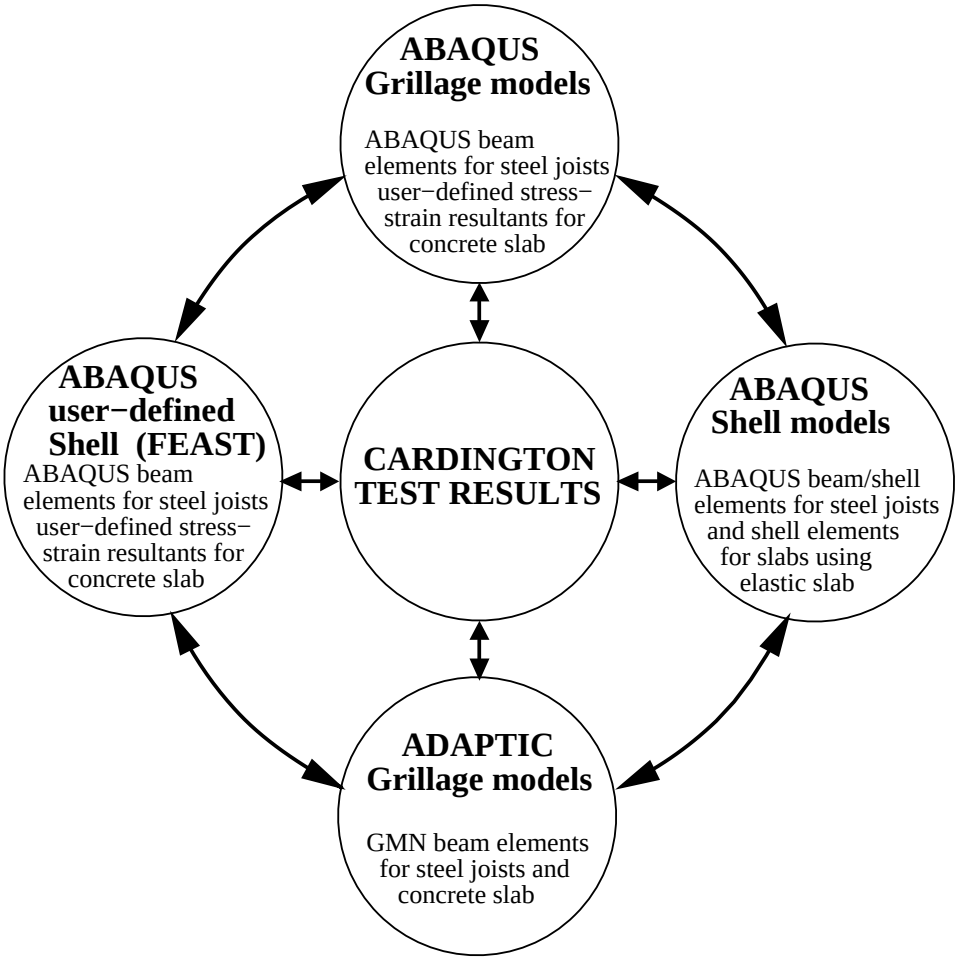
The methodology proposed to achieve the stated objective was to develop rigorous models of the British Steel fire tests using the commercial finite element package ABAQUS. These results were to be thoroughly validated against the data from the Cardington fire tests. Firstly, these models were to be used to develop a thorough understanding of structural behaviour and to identify the key features of structural response to fire in the Cardington tests. Secondly, the rigorous models were to be simplified in an step by step manner to produce models that were suitable for use in performing a large number of parametric studies as shown in **Figure 1.4**. An independent comparison of results from ABAQUS simplified models was to be made with computations made using ADAPTIC (software developed over the last decade at Imperial College). **Figure 1.5** shows the validation strategy adopted.

The deliverables originally proposed in this project may be summarised as follows (and may also be used as a measure against which the outputs are judged).

- Full details of BS data from the Cardington tests
- Documentation of all modelling and interpretation work carried out in the form of reports
- Design recommendations
- Technical journal publications disseminating the main results
- General seminar to structural engineering community on the findings



Methodology
Figure 1.4



Validation strategy
Figure 1.5

1.4 The scope and limitations of the research undertaken

The research results to be presented in this report are based upon the modelling of the BS fire tests at Cardington and other theoretical and computational work done in support of the primary modelling work. The modelling work clearly is limited to compartment fires in the Cardington structure. Strictly speaking, therefore, the interpretation of the model results and the conclusions drawn from them about structural understanding apply only to Cardington like structures and compartment fires. This may however be too strict an interpretation and we are confident that much of the knowledge gained on composite structure behaviour in fire should be more generally applicable. When discussing the results from particular tests and their modelling we will endeavour to enlarge upon the scope of their validity.

Most of the theory that will also be presented in this report or in one of the many appendices is indeed generally applicable to structures in fire and any limitations will be inherent in the stated assumptions made in the derivations.

2 MODELLING

2.1 Introduction

This chapter describes the modelling procedures used in the project. Firstly, the key structural aspects to be modelled are discussed before the modelling principles adopted to capture these aspects are outlined. Examples of the models developed to analyse each of the Cardington tests are then given along with the validation techniques used to ensure that the key structural behaviour is captured.

2.2 The Cardington Structure

This description of the Cardington structure tries to draw the distinction between the frame as it is assumed to act and the way that it actually acts. This will be done, at first, under static loading with the implications for behaviour under fire loading given later. The distinction is important because 'actual' behaviour needs to be modelled but it often needs to be compared with 'assumed' (design) behaviour.

The Cardington floor layout is given in **Figure 2.6**, the key points to note are:

- Braced cores provide lateral stability
- The floor slab carries load to secondary beams at 3m spacing
- Composite secondary beams[1] span 9m between columns and primary beams
- Composite primary beams predominantly carry secondary beam concentrated loads to columns.

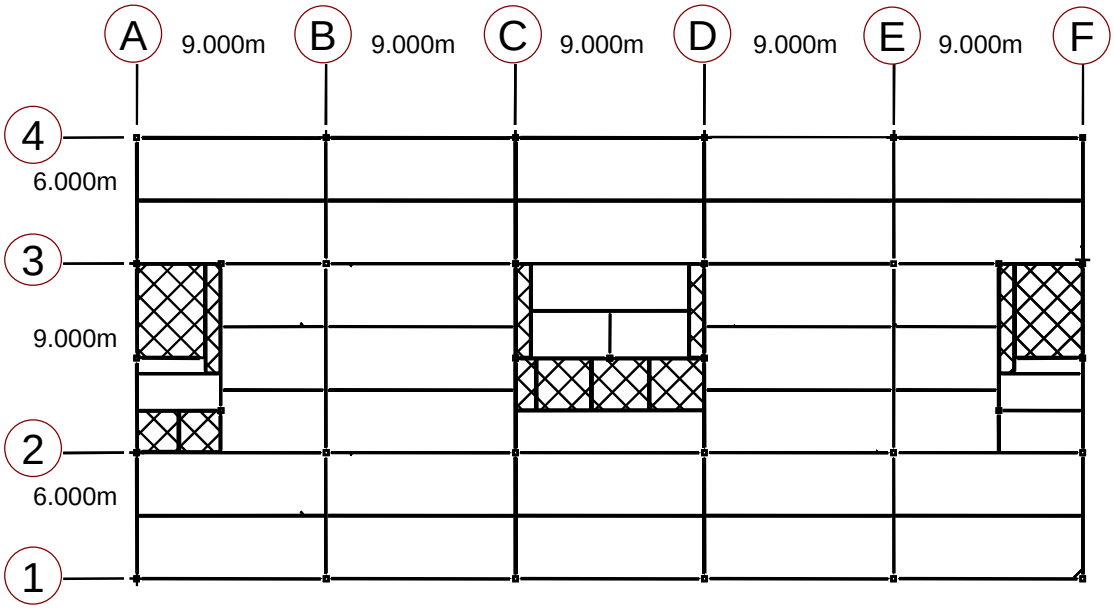
For illustration of the design assumptions, the steelwork representing the test 3 compartment (British Steel corner test) is shown in **Figure 2.7**. The structural details are representative of the rest of the structure. The assumptions are:

- The beams are designed as isolated members.
- The slab is assumed to carry the floor load as a continuous or simply supported beam spanning between the secondary beams.
- Secondary beams are assumed to be simply supported and to carry half of the slab load either side of the beam.
- Secondary beams are assumed to have an effective width of slab acting compositely with the steel joist[1]. This is the half span of the slab either side of the beam or a quarter of the secondary beam span, whichever is the less. In this case, the effective width would be assumed to be 2.25m.
- The secondary beam loading translates to a concentrated load on the primary beam, which is assumed to be simply supported.
- The primary beam also acts as a composite member with the assumed effective width of slab determined in an identical manner to the secondary beams.

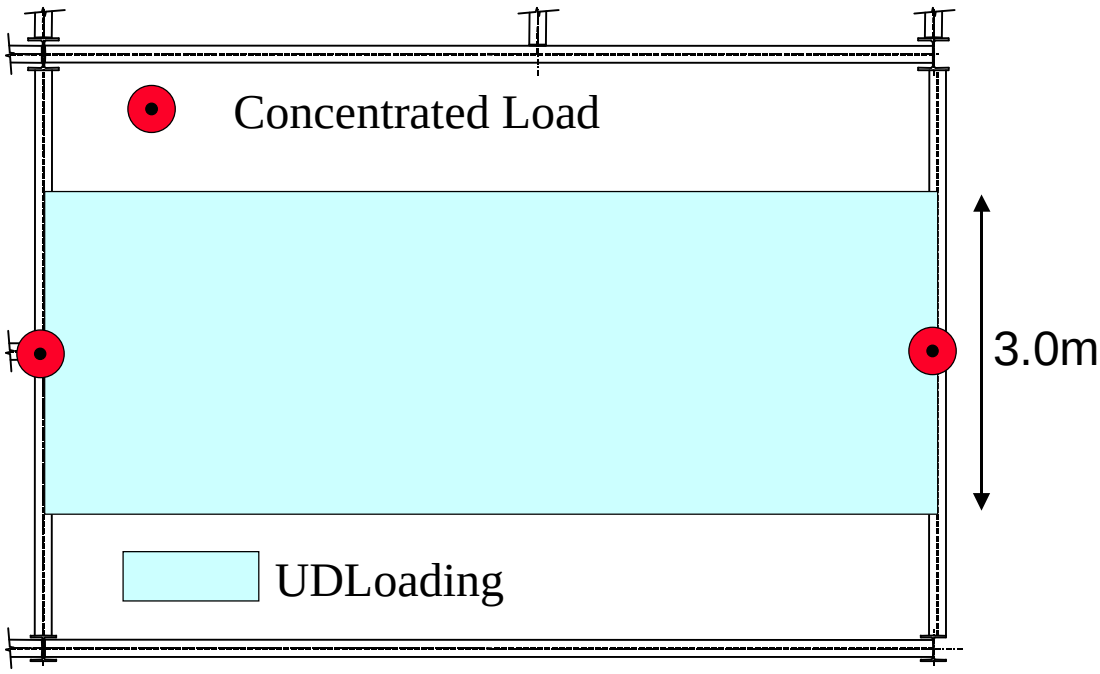
These assumptions idealise the floor system as two independent and determinate load-carrying mechanisms. One is the slab spanning between the secondary beams and the

[1] Within this chapter, the term 'beam' will be used to refer to the concrete slab and steel universal beam acting as a composite member. In referring to the steel universal beams, the term 'joist' will be used for clarity.

other is the secondary beams carrying the slab loads back to the columns and primary beams. In fact these two mechanisms are neither determinate nor independent. To examine this we must look at the key structural details.



Floor Layout of Cardington Structure
Figure 2.6



Steelwork in British Steel Corner Test – Test 3
Figure 2.7

2.2.1 Beams

The connection detail at the internal end of a 9m secondary beam spanning between primary beams is illustrated in **Figure 2.8**. The figure shows a fin plate connection on either side of the primary beam. This connection is assumed to be a simple connection i.e. shear transfer takes place but no moment transfer. However, because of composite action with the slab, force transfer will take place in tension through the slab and in compression through the steel joist. The compression force can be generated due to the stiffness of the beam on the other side of the connection. This gives rise to a moment being transferred at the connection, meaning that in reality the connection is really semi-rigid, at least initially. Moreover, and particularly important for behaviour in fire, the end of the beam is heavily restrained against translation.

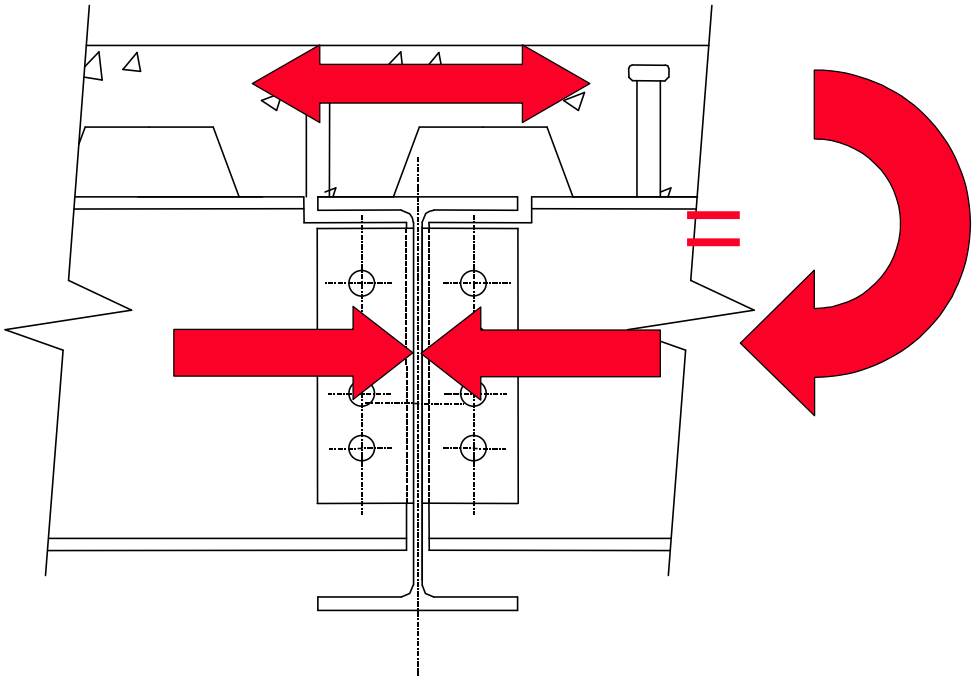
Figure 2.9 shows the connection detail at the external end of the 9m secondary beam spanning between the primary beams. It again shows the fin plate connection on the side of the primary edge beam. Therefore composite action will again produce tension through the slab and compression through the steel joist. The actual compression force generated depends upon the lateral stiffness and torsional stiffness of the primary beam. This gives rise to a limited moment being transferred at the connection depending on the degree of restraint.

By representing these end conditions in a schematic diagram of the beam, as shown in **Figure 2.10**, it can be seen that far from being simply supported, the beam is actually:

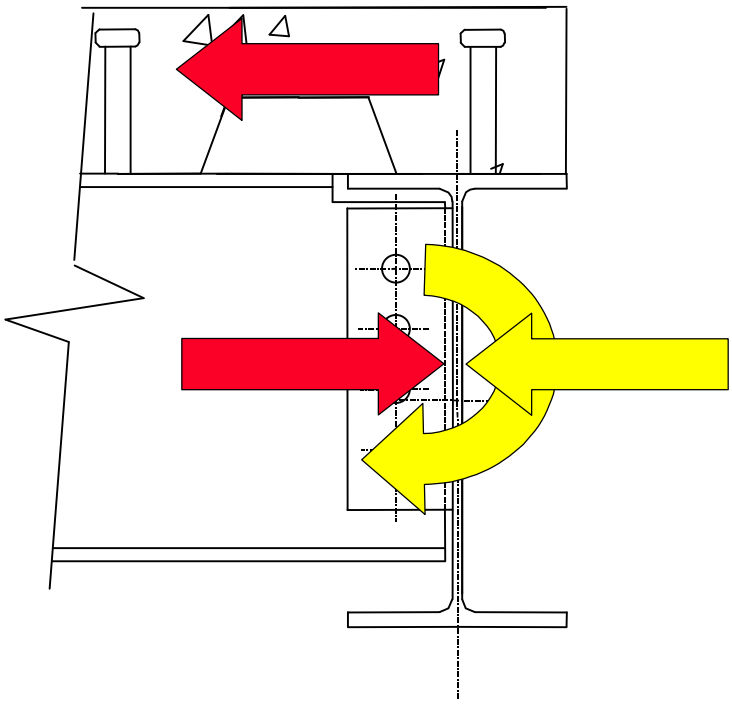
- heavily restrained against end translation at the internal connection
- restrained against end translation to some degree at the edge connection
- transfers some moments at both connections.

These effects are not considered important for conventional design, however, under fire conditions this will not be the case.

A similar line of reasoning may be followed for primary beams (and secondary beams between columns) as shown in **Figure 2.11**. The connections to the columns are partial depth endplates. This connection is also assumed to be a simple connection i.e. shear transfer but no moment transfer. For reasons discussed earlier, this is also a semi-rigid connection, and some moment transfer can take place. The beams are also highly restrained by the columns against end translation. **Figure 2.12** shows a schematic of beams between columns, showing all the different reactions available.

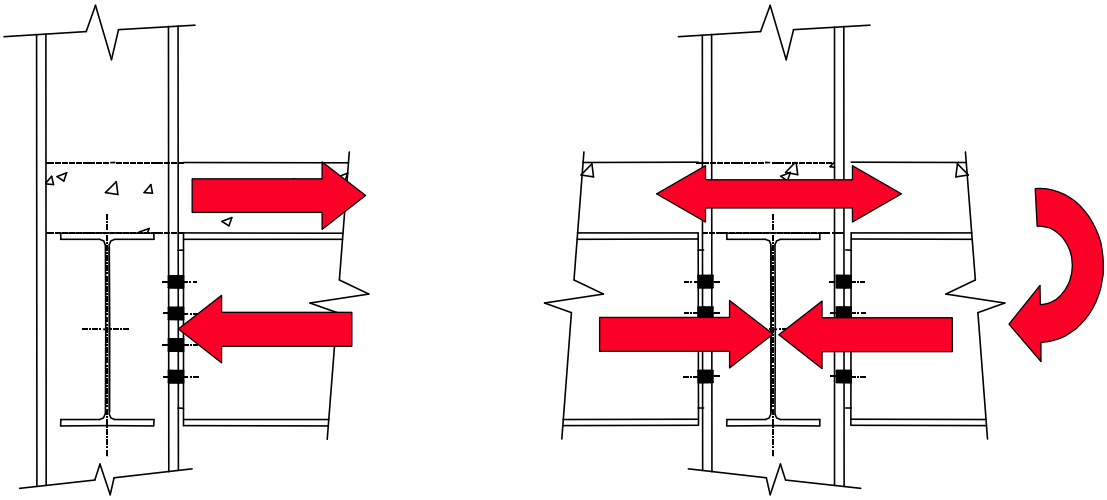


Internal connection detail at the end of secondary beam
Figure 2.8

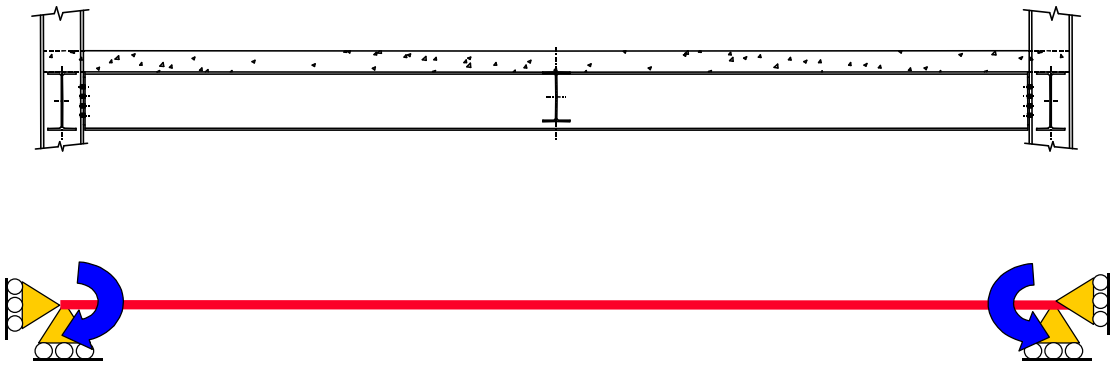


External connection detail at end of secondary beam
Figure 2.9

Actual structural representation of secondary beams
Figure 2.10



Connection details at the end of the primary beams
Figure 2.11

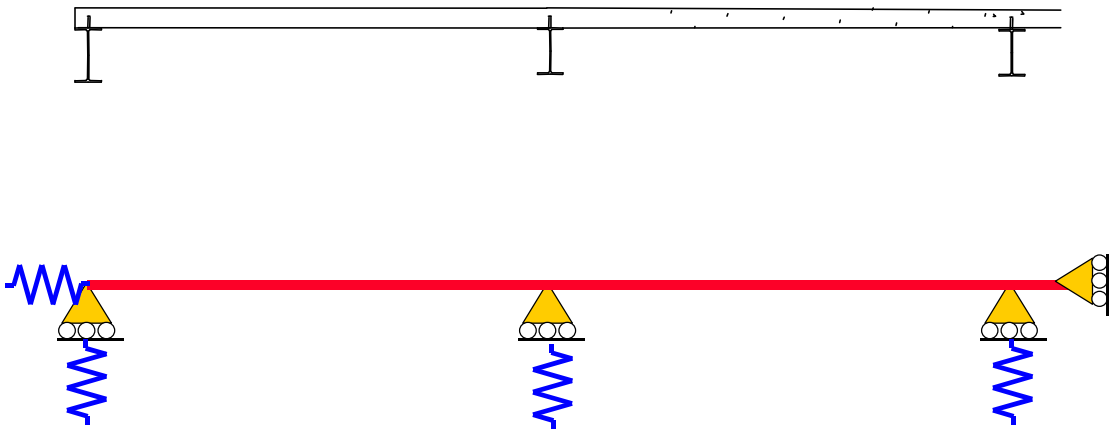


Actual structural representation of primary beams
Figure 2.12

2.2.2 Concrete slab

The concrete slab is anisotropic due to the presence of the downstands (ribs) which make the slab predominantly one way spanning. A cross-section through the slab at the centreline of the secondary beam is given in **Figure 2.13**, which also shows a schematic representation of the support conditions. In addition to the restraint to vertical deflection provided by the secondary beams, the slab is also translationally restrained by the rest of the structure internally, and by the composite beam at the

edge of the structure. The degree of restraint will depend upon the stiffness of the restraining mechanisms.



Actual structural representation of concrete slab
Figure 2.13

2.2.3 Whole structure behaviour

The above view of the structural system tries to isolate individual elements. However, the structural elements do not act in isolation. Under static conditions, the simplifying assumptions of isolated determinate members leads to safe structures under both vertical and lateral loading. In reality, the structure will have a high degree of redundancy arising from numerous interactions between members. Under fire conditions, the predominant loads are due to thermal expansion of the main structural elements. This will give rise to further complex interacting mechanisms due to the thermal response of structural members.

2.3 Modelling Principles

This section outlines some of the primary considerations in modelling the behaviour of composite structures in fire and the steps taken in building models which represent the key structural behaviour. Building models that adequately reflect the key structural behaviour is dependent upon making the right choices and assumptions at each level of the finite element analysis. The section seeks to illustrate some of these choices and options.

The basic modelling aims and principles adopted in the project were:

- The identification of the main load carrying mechanisms
- Adequate representation of these load carrying mechanisms
- Accurate representation of the stiffness of the structure and stiffness of restraints
- An assessment of the sensitivity of the models developed to key variables
- Qualitative, as well as quantitative, validation against experimental results
- Consistency with the fundamental principles of structural mechanics

In determining the response of a structure in terms of internal forces and displacements it is vitally important to represent the stiffnesses accurately. If the stiffness of the model is correct then the response of the model to the various applied loads (increasing temperature is one) will also be correct. This point is important as

most engineers tend to concentrate on strength rather than stiffness due to the discipline of limit state design and the use of codes.

The list of phenomena to be included in the models, from the structural considerations above and from observations of the Cardington fire tests, is given below.

- Beam local buckling behaviour
- Concrete slab behaviour – including anisotropy
- Geometric nonlinearity in all members
- Material nonlinearity in all members
- Connection behaviour
- Column behaviour
- The effect of different temperature profiles through structural members
- The assumed boundary conditions to the test compartment
- The shear interaction between beam and concrete slab

The actual contribution of each to the overall structural behaviour is not known in advance. It is important that any models that are developed include all the above effects to gain an understanding of which ones are the most important.

The basic steps in building a finite element model are listed below.

- Meshing of the structure
- Finite element selection
- Material modelling
- Boundary conditions
- Applying the loads
- Choice of analysis procedure

The steps will be discussed individually and the most important choices made at each stage illustrated.

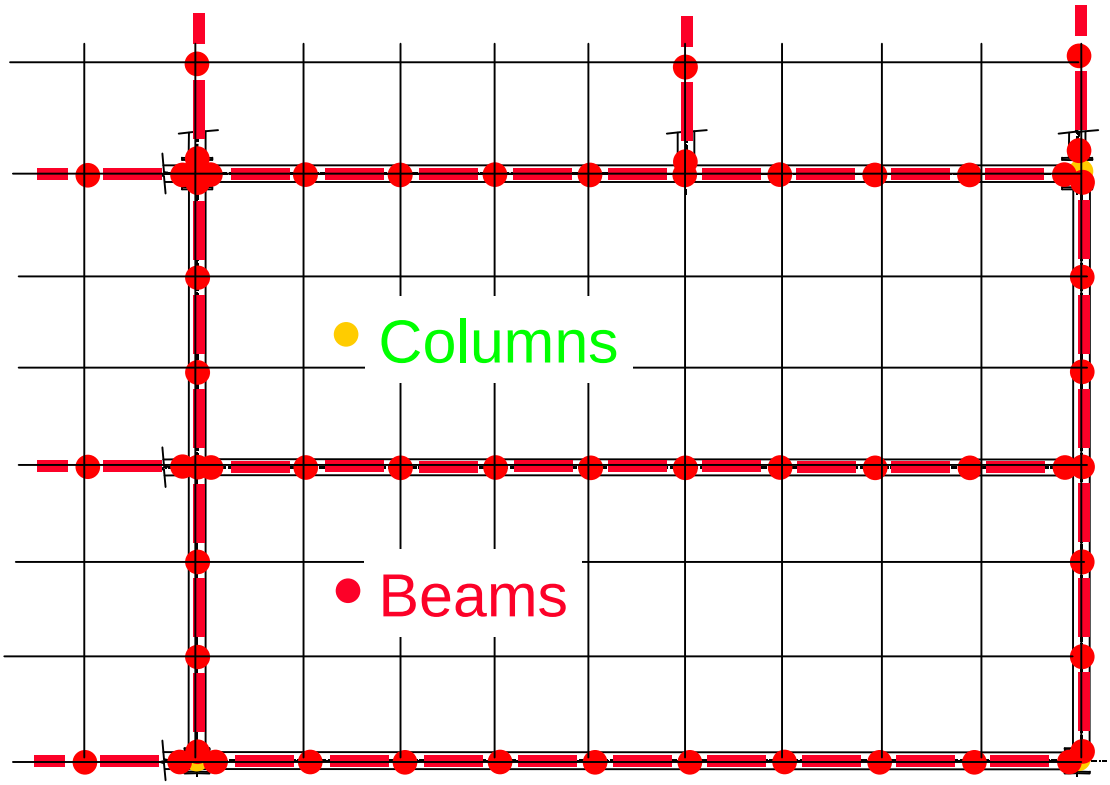
2.3.1 Meshing of the structure

Meshing is illustrated by reference to the corner test as it includes most of the structural details. A plan view of the corner test with a schematic representation of the mesh is given in **Figure 2.14**.

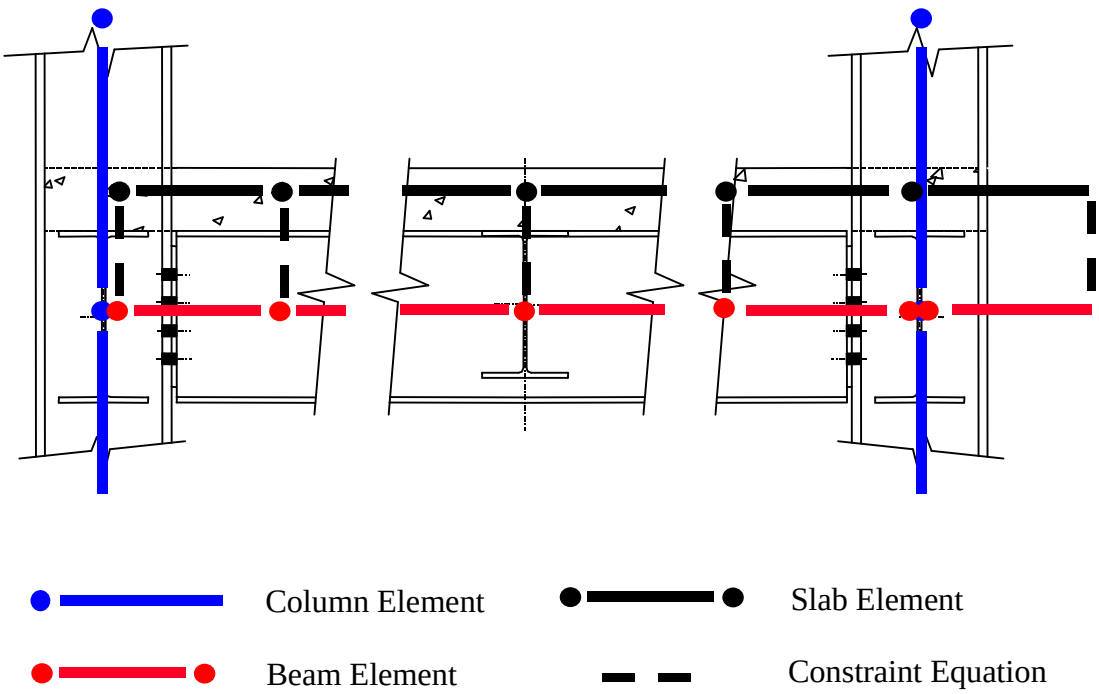
The columns are meshed first and are usually represented with beam elements. The columns are meshed so that a column node exists at the level of the main structural steelwork (see **Figure 2.15**). The rest of the steelwork is then built up. Joists are modelled using beam or shell elements; beams are assumed in this discussion. Where joists and columns meet, duplicate nodes are specified. This allows spring representations (rotational and translational) for the connections or constraint equations between the beams and columns to be specified. A similar approach is taken for secondary joists spanning between primary joists where separate nodes for the two members are used. After the steelwork has been meshed, a representation of the slab is included, this can consist of either beam or shell elements. It will normally have the same mesh density as the steelwork.

The meshing considerations to achieve the correct degree of thermal restraint to the key structural elements are illustrated schematically in **Figure 2.15**. The key points to note are:

- Concrete slab elements are placed at the centre of the top 70mm of the slab i.e. 95mm above the top of the beam. The slab is represented by either shell elements which are 70mm thick acting in both directions or beam elements acting in the joist direction.
- Beam elements are usually placed at the centreline of the joist.
- Beam elements are connected to the columns with springs (semi-rigid) or constraint equations (usually pinned)
- Slab shell elements are not connected to internal columns. This represents the fact that stress can 'flow' around the column in achieving slab continuity.
- Slab elements are connected to joist elements using constraint equations which represent a rigid link between the joist and slab. This represents full composite action. Springs could be included instead of rigid links to represent partial shear transfer.



Plan view of Corner test showing meshing details
Figure 2.14



Elevation of Cardington structure showing meshing details
Figure 2.15

2.3.2 Finite element selection

The basic finite element sets used to represent the structural members in the composite frame are given below. Solid elements are omitted as they are numerically too expensive for representing large extents of frameworks.

- Shell elements are 2D planar elements which include both membrane and flexural terms. Through thickness properties are included by integrating through several layers. Each layer is assumed to be in a two dimensional plane stress state.
- Beam elements are 1D elements which include both axial and flexural terms. Plane sections are assumed to remain plane in bending and shear. Cross-sectional variations are included by integrating at several points through the appropriate cross-section, assuming a one-dimensional stress state.
- Spring elements represent the strength and stiffness between two points which are assumed to be nearly coincident.

An illustration of the joist behaviour that may need to be modelled is given in **Figure 2.16**. The Figure illustrates local buckling of the section. This behaviour can only be represented using shell elements. A representation of the joist section so that local buckling can be studied is given in **Figure 2.17**. The majority of the section - web and lower flange - is represented using shell elements. This enables the cross-section of the joist to distort locally in the manner illustrated. If beam elements were used to represent the section then a perfect I-shape would be maintained at every cross-section along the length of the joist. The effect of modelling the joist in both ways was studied within this project.

Representing the joist using shell elements also gives added flexibility in the representation of the connection behaviour at the end. For example, partial depth endplate connections can be represented as a series of spring and gap elements which ensure a good representation of the 'actual' local restraint condition to both the web and lower flange of the joist at the connection (**Figure 2.18**). This is important in examining the local buckling phenomena. A partial depth endplate, initially, provides no restraint to the lower flange of a joist as it is only connected to the web. However, the gap, nominally equal to the thickness of the endplate, can close up during the fire test due to beam thermal expansion. This effect can be examined using such joist representation.

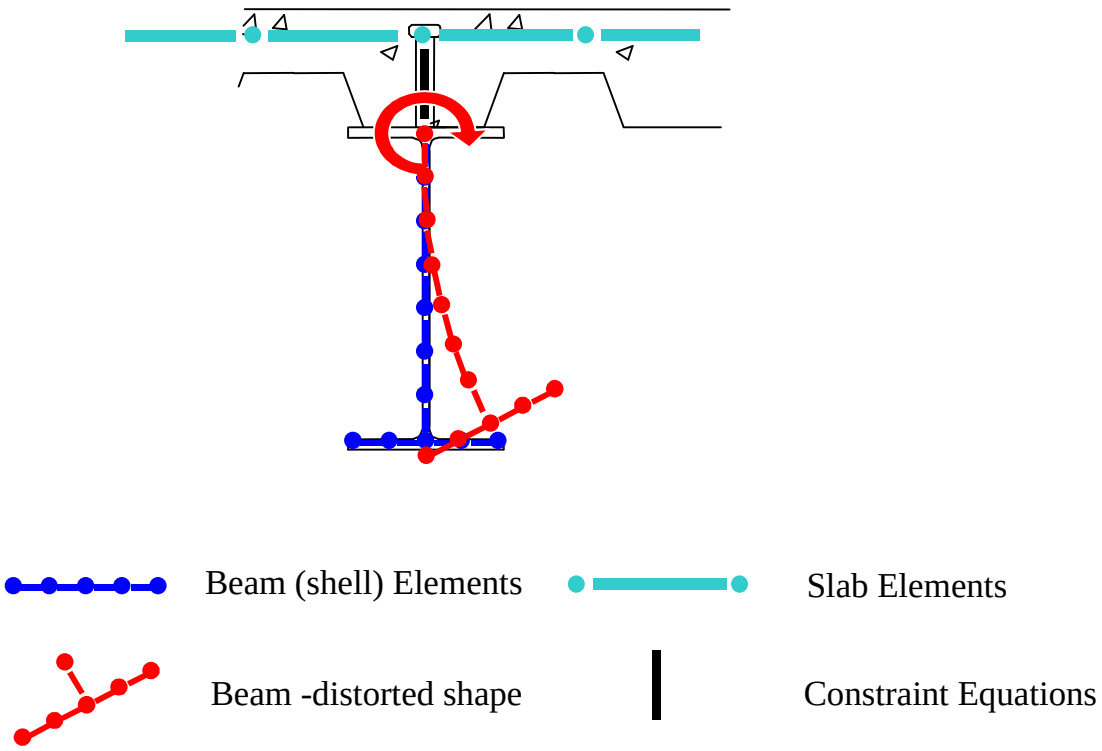
The concrete slab can be modelled using either shell elements, beam elements or a combination of the two approaches. All three methods have been used in this project.

Modelling the slab using only shell elements presents numerical convergence problems because of the discontinuous material behaviour of concrete. One method of avoiding these problems is to adopt a stress resultant approach to defining the behaviour of the elements. This can be done within ABAQUS using a user-defined subroutine written in FORTRAN. Alternatively, the anisotropic properties of the slab can be included by representing the downstand (bottom 60mm) of the slab using beam elements (**Figure 2.19**). These elements use the top slab nodes and are offset by the correct amount to achieve the desired slab properties.

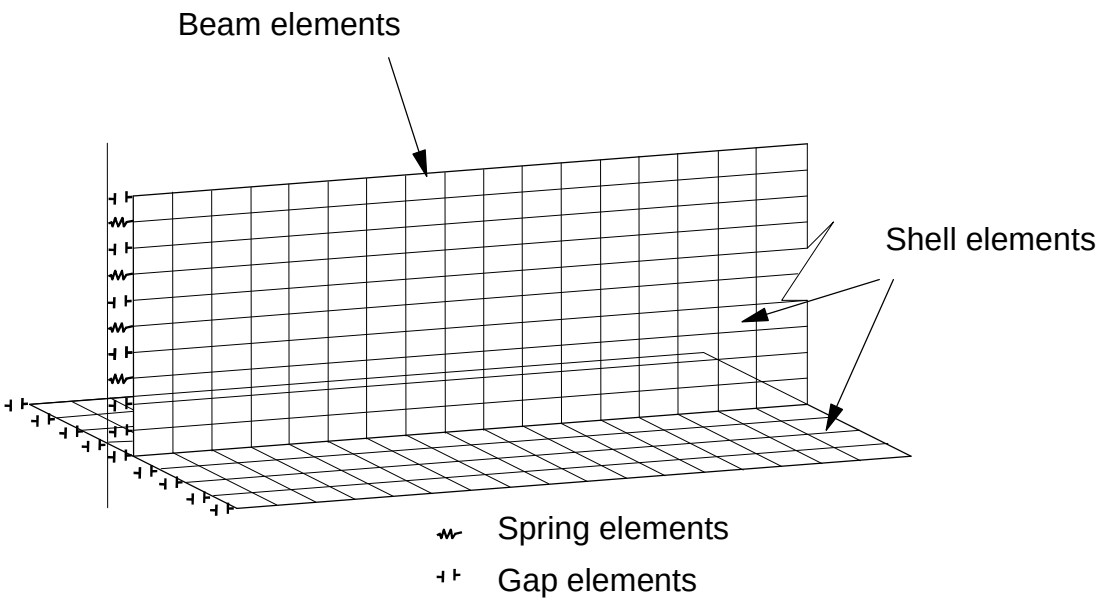
The slab can also be represented as a grillage model where the behaviour is split into two orthogonal load carrying mechanisms (**Figure 2.20**). This allows beam elements to be used to represent the slab behaviour. This gives advantages in terms of computational speed and also allows the use of stress-resultant models which smooth the effects of locally high stresses to give more stable forms of material nonlinearity. The accuracy of the representation will be dependent on the relative importance of the stiffness of the slab in the two directions. That is, if the slab is sufficiently anisotropic that the slab behaviour is dominated by behaviour in each of its decoupled load carrying directions then the grillage model should give an adequate representation.

It should be noted that representing the slab using shell elements means that a lot more post-processing work needs to be done to integrate stresses into forces and moments which are meaningful to engineers. Whatever sort of representation of the slab is chosen, a decision must be taken about the effective width that is assumed to act compositely with the joist. When using a grillage approach this decision is taken *a priori* when using a shell approach, *a posteriori*.

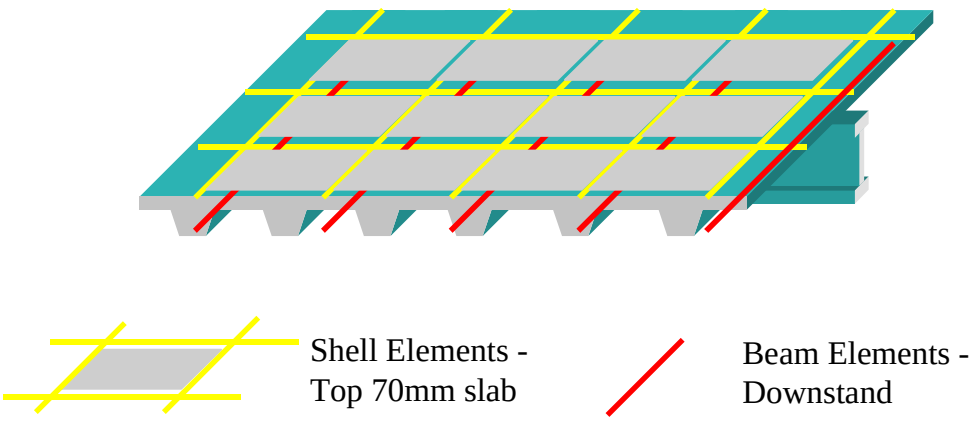
Beam local buckling behaviour (British Steel Test 1)
Figure 2.16



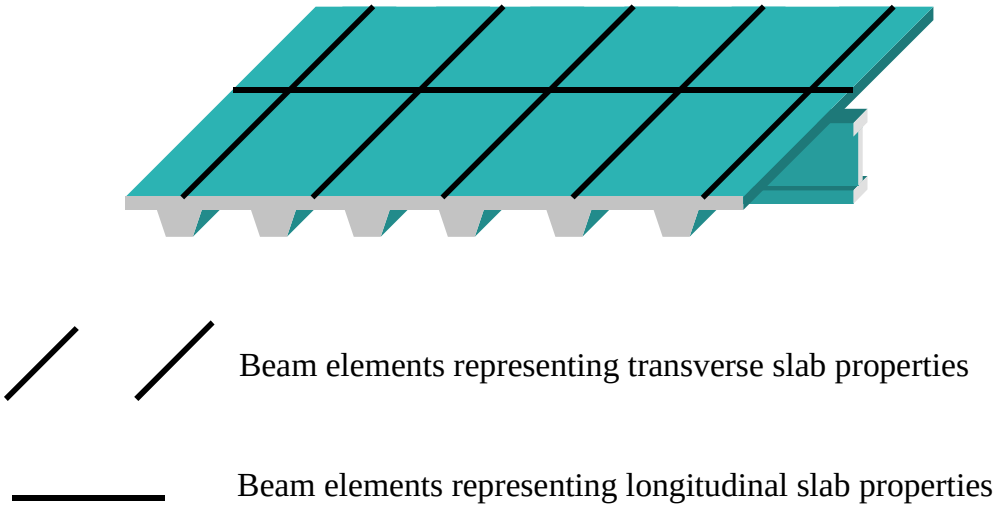
Mesh representation of steel beam using shell elements
Figure 2.17



Beam shell model showing connection modelling
Figure 2.18



Shell representation of concrete slab including anisotropic properties
Figure 2.19



Grillage representation of concrete slab
Figure 2.20

2.3.3 Material Modelling

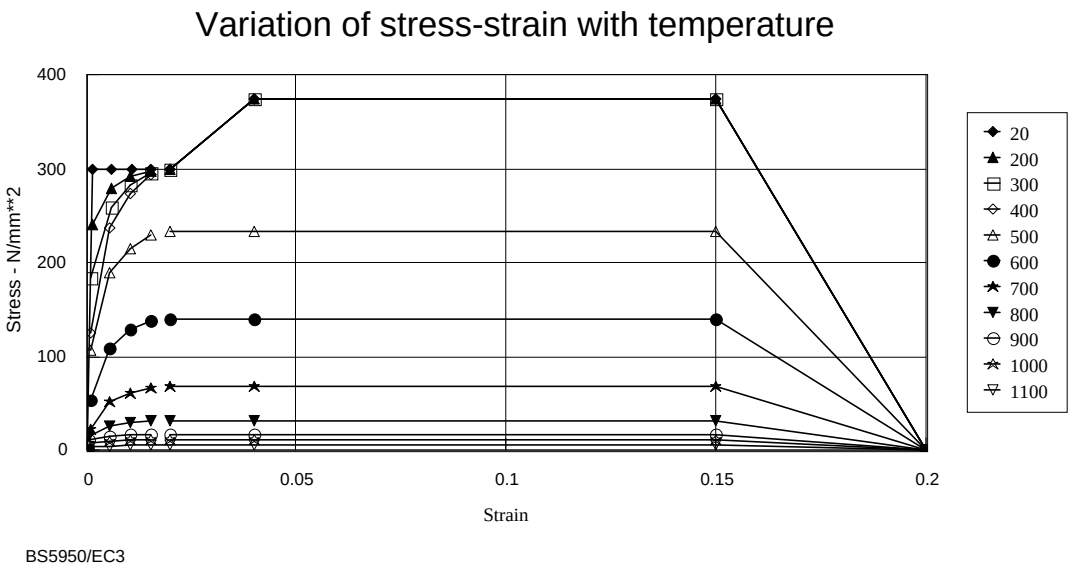
Steel is modelled using an elasto-plastic model. A von Mises yield criterion is used with associated plasticity. Full degradation of the stress-strain curves with temperature is allowed. The steel material properties are taken directly from the material definition in EC3: Part 1.2 [2] and include the enhanced strain hardening effect above 400 ^{o}C, **Figure 2.21**. It should be noted that there may be some degree of conservatism in the material properties assumed in the code.

Concrete modelling poses considerable numerical challenges. This is primarily due to the nature of concrete's low tensile (cracking) strength and brittle behaviour. Full degradation of concrete material properties with temperature is included. Reinforced concrete properties are included by modelling the rebars with a steel material model. This allows for the specification of layers or individual rebars of reinforcing steel to be included within concrete elements. The reinforcement layers are assumed to be one-dimensional, acting purely in tension or compression. Some approximations to the transfer of load from cracking concrete to reinforcement is allowed within the concrete cracking model itself. The concrete material properties are taken directly from the material definition in EC2:part 1 [3]. (**Figure 2.22**). As in the case of steel, there may be some degree of conservatism in the material properties assumed in the code.

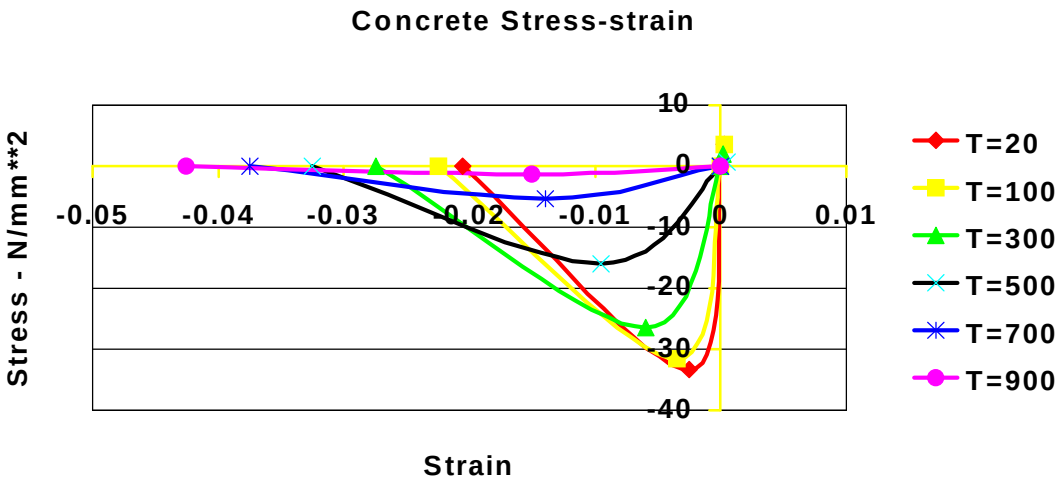
The concrete cracking model in ABAQUS met with limited success in modelling slab behaviour due to local cracking behaviour preventing overall numerical convergence. It is possible to introduce a more stable form of slab nonlinearity by working in terms of stress resultants (moment and forces per unit width), rather than numerically integrating stresses across a particular section.

If an idealised strain distribution across the slab section is considered, it can be split into axial strain and curvature. The force-strain and moment curvature relationships

for any given concrete slab and temperature profile can be calculated [4] and directly input into the finite element model. It should be noted the stress-resultant beam representation available in ABAQUS assumes that axial and bending strains are completely decoupled, although the effects of geometric nonlinearity are included. This limitation is not present in the user-defined, stress-resultant shell model available in ABAQUS



Steel material properties used in ABAQUS model
Figure 2.21



Concrete material model used in ABAQUS modelling
Figure 2.22

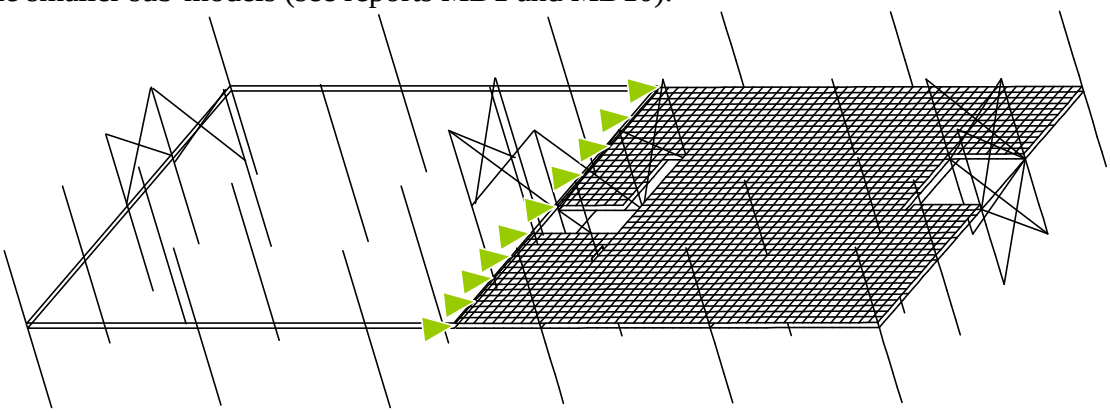
2.3.4 Boundary conditions

As the predominant response of the structure is restraint to thermal expansion, it is important that close attention is paid to applying the correct boundary conditions to the model. This becomes doubly important when analysing sub-models of the complete frame as in this case the restraint provided by the rest of the structure to the expanding members of the fire compartment is clearly altered. The degree to which the restraint conditions change depends upon individual structural configurations and the type of model used. In most of the tests modelled it was found that sub-models could be used with very little impact on the main pattern of behaviour and relatively insignificant differences in the internal forces (for instance Sanad 99). However the boundary conditions for the sub-models must be chosen with care, especially if the sub-model is not much bigger than the compartment itself, as in this case the potential for significant change in behaviour will be much greater. The types of boundary conditions used in this work are discussed next.

The bases of the columns in the floor structure are fully restrained in all directions and that the tops of the columns are translationally restrained in the horizontal directions. These boundary conditions simulate the continuity of the columns at the base of the structure and at the top of the columns.

A typical symmetry boundary condition for a half floor model is given in **Figure 2.23**. The symmetry boundary in this case is restraint to translation across the symmetry boundary and rotational restraint about the two major axes on the plane of symmetry. Care does need to be exercised in the application of symmetry boundary conditions particularly when the test compartment is close to the model boundary. In these cases two fire tests on either side of the symmetry boundary will be simulated with the consequent effect on the degree of restraint provided to the compartment of interest. When this occurs, superelements for the other half of the floor, representing the 'stiffness' of the other half of the floor can be used.

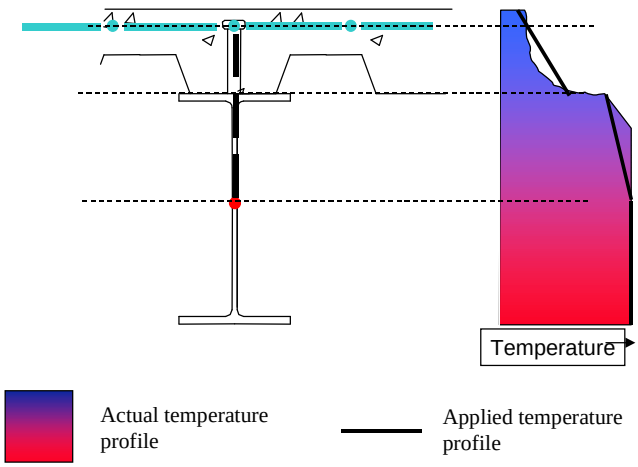
In case of smaller compartments it has been found sufficient to include one or a half of a bay beyond the compartment boundaries with either restrained lateral translations or symmetry. The boundary conditions for grillage type models can follow the same rules as discussed here, however because of the discrete nature of grillage models, the exact boundary conditions applied need even more detailed attention, especially for the smaller sub-models (see reports MD1 and MD10).



Symmetry boundary condition applied to Cardington half floor model
Figure 2.23

2.3.5 Loading

There are two types of load that need to be applied to heated structures. The first type is gravity loading, which is normally straightforward to apply. The second type consists of the temperatures of the main structural elements. Sometimes approximations need to be made in applying temperature profiles through the various elements in ABAQUS. A schematic of a typical temperature profile through the slab and joist is given **Figure 2.24**. This needs to be represented through two elements, one for the slab and one for the joist. The beam element temperature profiles can be input by applying temperatures at the top and bottom flanges and at the centre of the web. This gives a reasonable approximation to the actual temperature profile through this element. However, the actual temperature profile through the slab is less uniform. Slab temperatures can sometimes only be input by a reference axis temperature and a constant thermal gradient especially for stress resultant models. The sensitivity to assumed and actual temperature profiles of the various models has been investigated in the project (see report SM5).



Comparison of actual and applied temperature profiles
Figure 2.24

2.3.6 Analysis Procedure

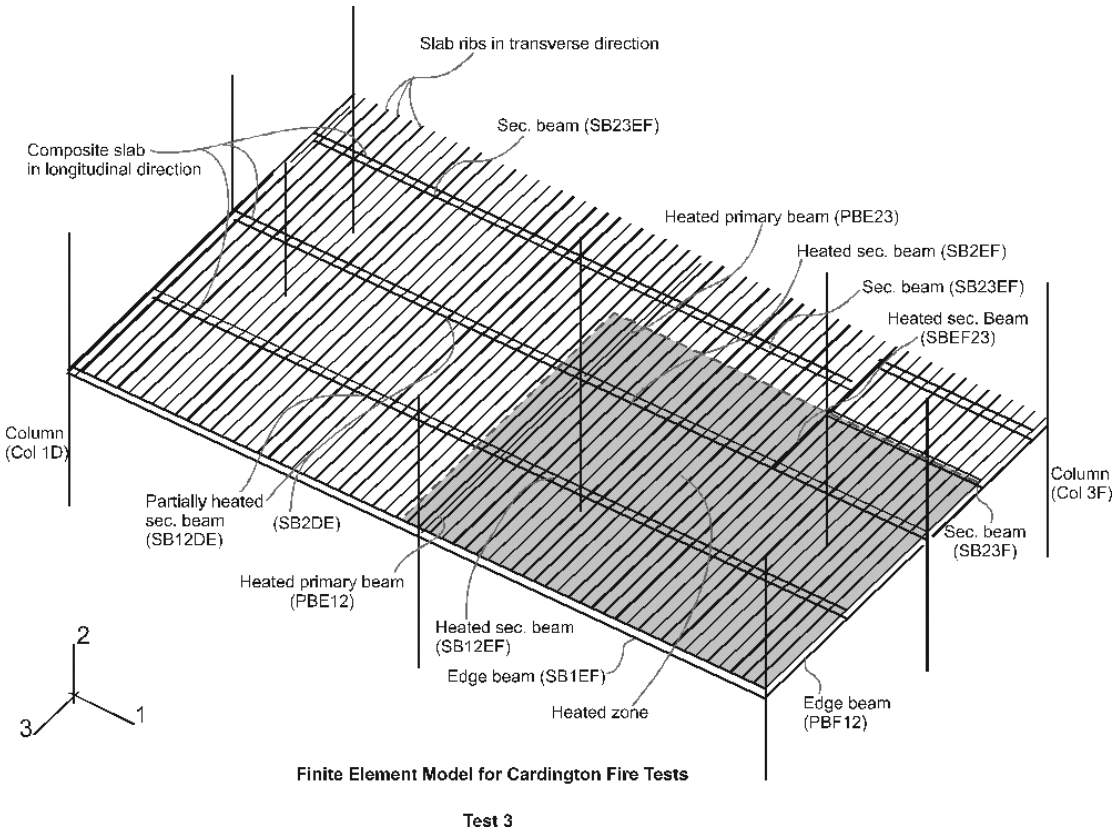
Various analytical techniques can be used within FE packages to obtain solutions to a variety of structural problems. Many of these techniques, such as arc length methods, have been derived to deal with buckling type problems where instabilities in structural behaviour lead to unloading of parts of the structure. All these techniques assume that the program has control of the loading so that unloading can take place when instabilities occur. However, in the case of heated structures such analysis techniques do not work well. This is unfortunate because in this case the restraint to thermal expansion gives rise to quite complex bifurcation type problems in both the slab and beam elements. The only technique the analysis package has available is the standard Newton-Raphson solution method. Using this technique it is necessary to reduce the solution increment size when instabilities occur, leading to quite lengthy solution times.

The most important part of the analytical technique for heated structures is accounting for geometric nonlinearity. High axial forces due to restraint to thermal expansion, coupled with large deflections due to thermal strains, mean that geometric nonlinearity is extremely important. Any package that does not take this into account will not be able to represent the correct structural behaviour.

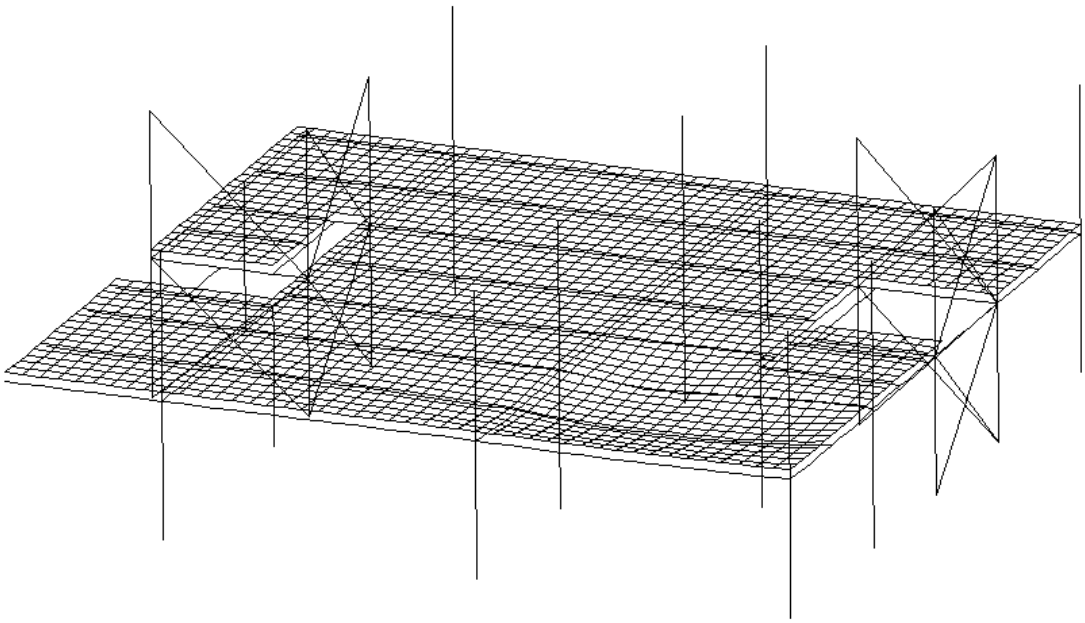
2.4 Examples of Developed Models

The modelling principles outlined above were used to develop rigorous, robust finite element models using the state of the art, commercial finite element package, ABAQUS and the research software, ADAPTIC. The models used to analyse each of the Cardington tests are described in detail in reports MD1 to MD15. These models can be split into five broad classes:

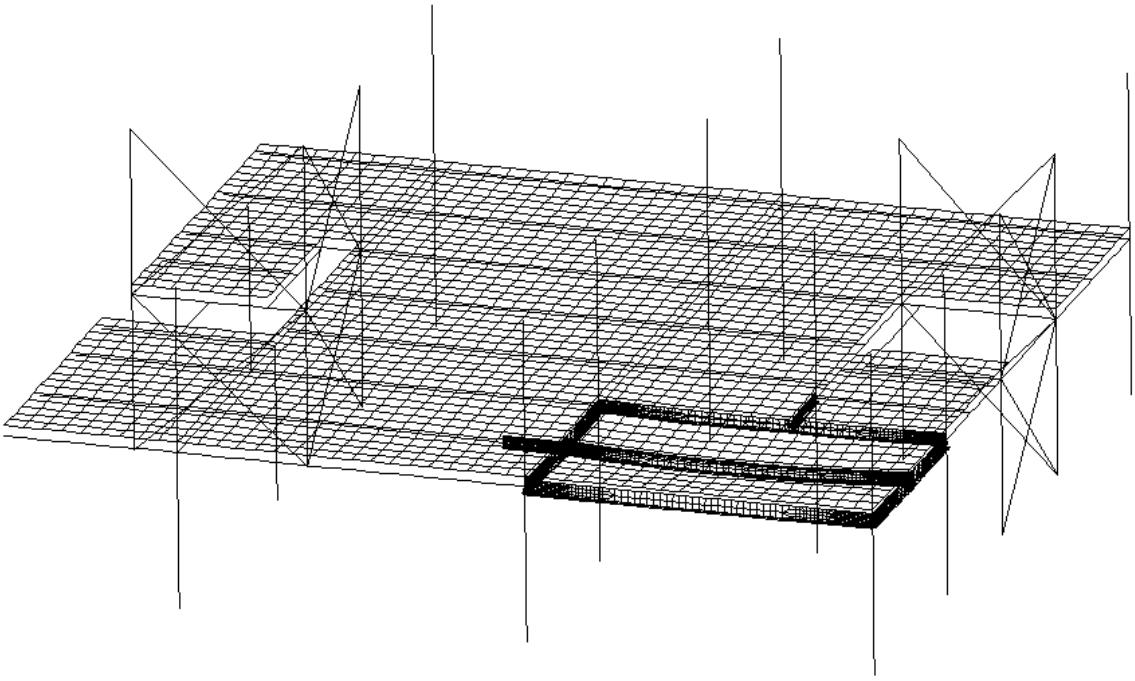
- ABAQUS models with steel joists modelled using beam elements and concrete slab modelled using a grillage representation (**Figure 2.25**).
- ABAQUS models with steel joists modelled using beam elements and concrete slab modelled using a shell representation (**Figure 2.26**).
- ABAQUS models with steel joists modelled using shell elements and concrete slab modelled using a shell representation (**Figure 2.27**).
- ABAQUS models with steel joists modelled using beam elements and concrete slab modelled using a coupled stress-resultant shell representation (FEAST) (**Figure 2.28**).
- ADAPTIC models with steel joists modelled using beam elements and concrete slab modelled using a grillage representation (**Figure 2.29**).



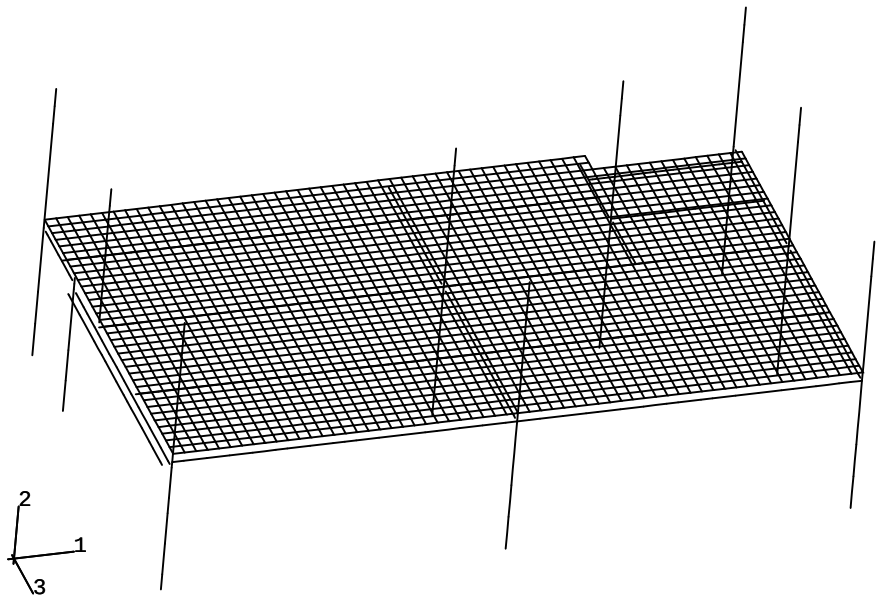
ABAQUS beam model with grillage representation of slab- Corner test (test 3)
Figure 2.25



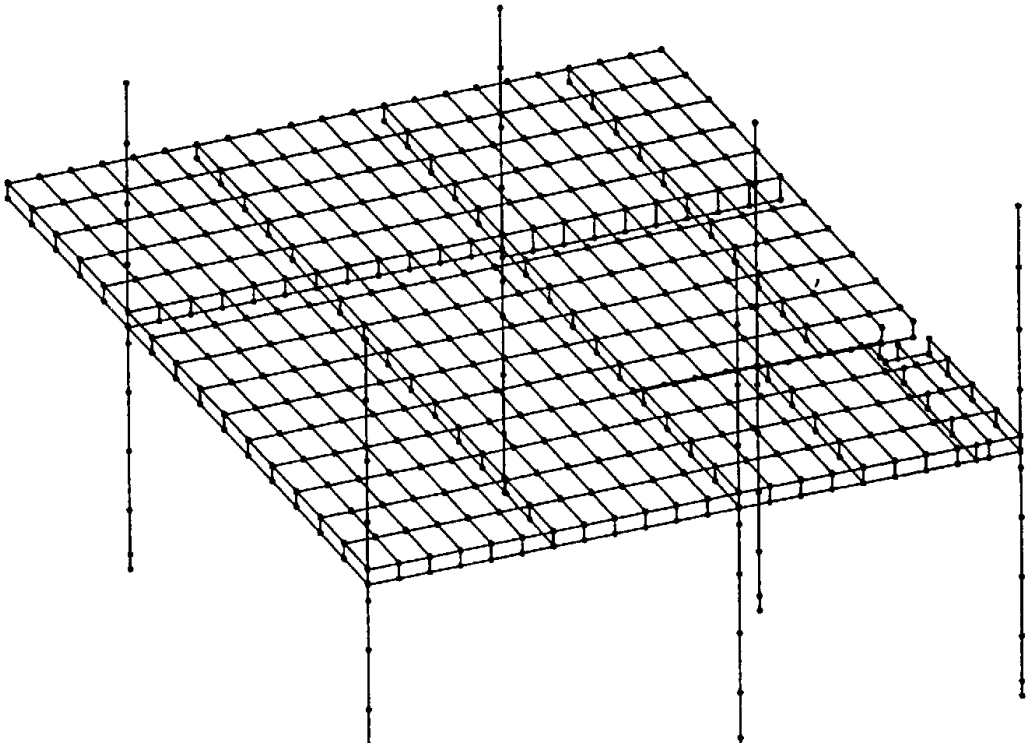
ABAQUS beam model with shell representation of slab- Corner test (test 3)
Figure 2.26



ABAQUS shell representation of beam and slab – Corner test (test 3)
Figure 2.27



ABAQUS beam model with stress resultant shell (FEAST) representation of slab – Corner test (test 3)
Figure 2.28



ADAPTIC beam model with grillage representation of slab – Corner test (test 3)
Figure 2.29

2.4.1 Validation procedure

The models were thoroughly validated by:

- Producing models using two different software packages (ABAQUS and ADAPTIC), one commercial and one research
- An iterative process of comparison of simplified models with more rigorous models and experimental data
- Developing different models of the same phenomena independently, in parallel to test model sensitivities and modelling assumptions
- Developing methods of post-processing results so that the underlying structural mechanics could be more easily understood
- Developing the fundamental theory from fundamental principles of structural mechanics to examine the consistency of model results with the theory
- Utilising the simplified models to conduct parametric studies to explore the sensitivity of the structure by monitoring changes of structural behaviour with changes in key modelling parameters

2.4.2 Validation examples

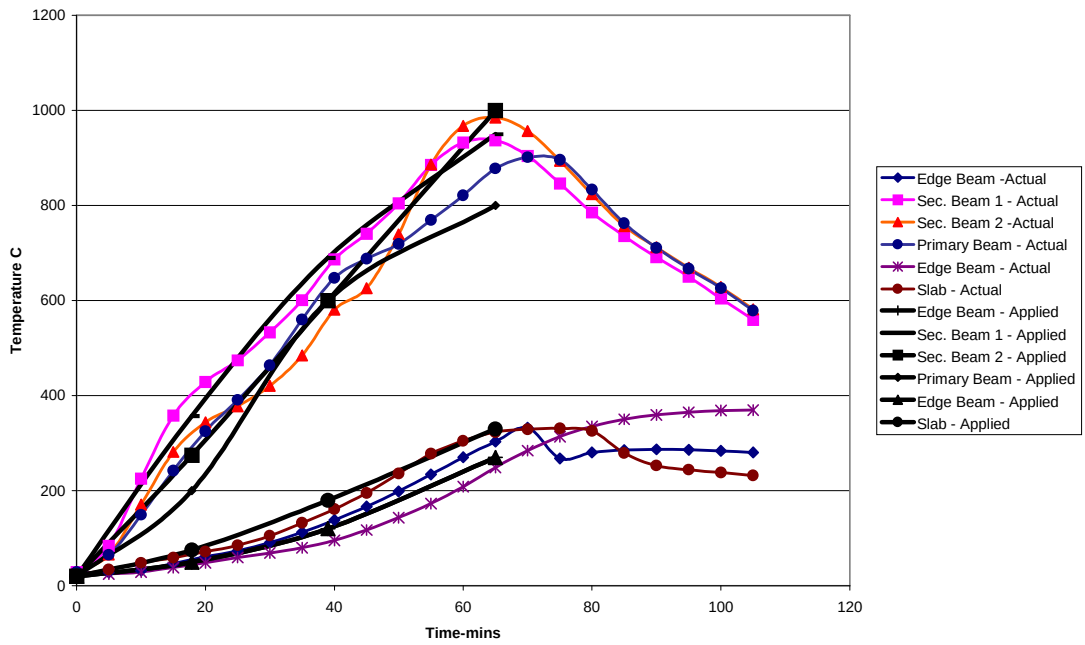
Test 3

The models were compared with as much of the experimental data as possible. The experimental data are analysed in report AE1. The temperature in each of the tests was input in as simplified a manner as possible. This entailed making some assumptions about the rise in temperature of each structural element in relation to each other for all the tests. A typical comparison of applied temperature to actual temperature is given for the corner test (Test 3) in **Figure 2.30**. The models were analysed up to peak temperature in each of the tests.

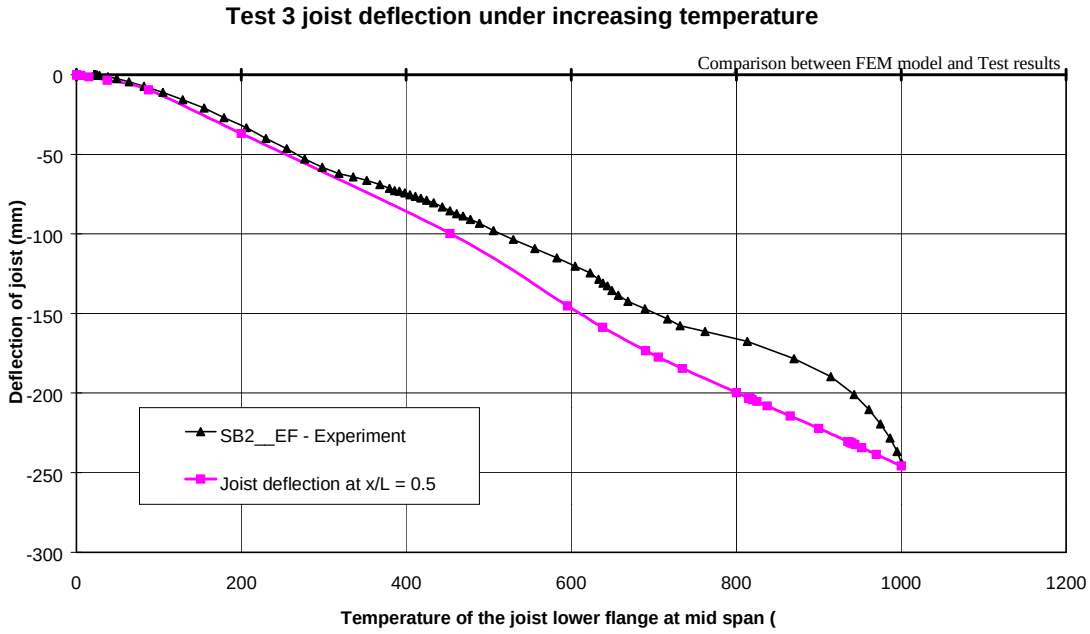
Key experimental data for comparison of the models were:

- Vertical deflections across the whole of the compartment to measure compatibility and thermal response
- Lateral deflections, where available, to measure degree of restraint provided by the structure
- Strain gauge readings to provide an indication of mechanical response of the structure to thermal loading and redistribution of forces.

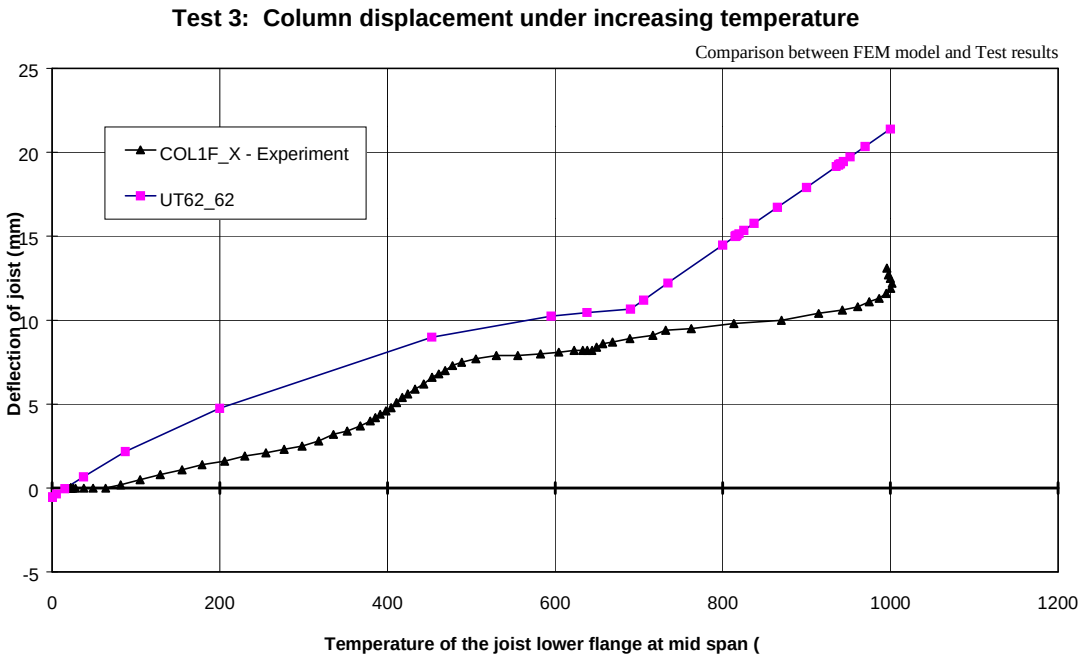
Full validation of each of the models against the key experimental data for each test is given in the reports MD1-15. Examples of validation of models against vertical deflection, lateral deflection and strain gauge readings are given in **Figure 2.31** to **Figure 2.33** for various models of Test 3.



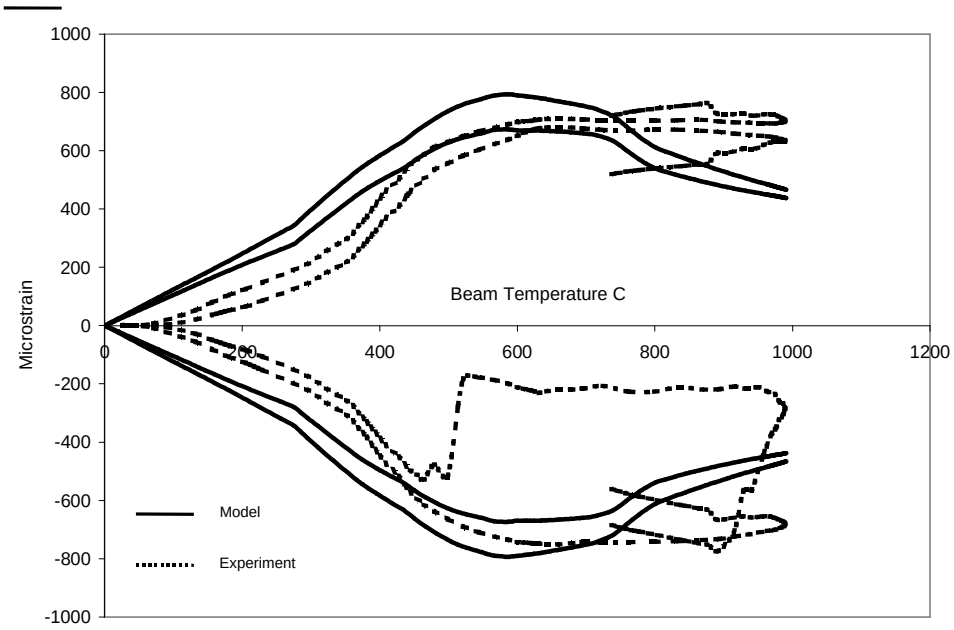
Comparison of actual and applied temperatures for Corner test (test 3)
Figure 2.30



Comparison of vertical displacement versus temperature – Corner test
Figure 2.31



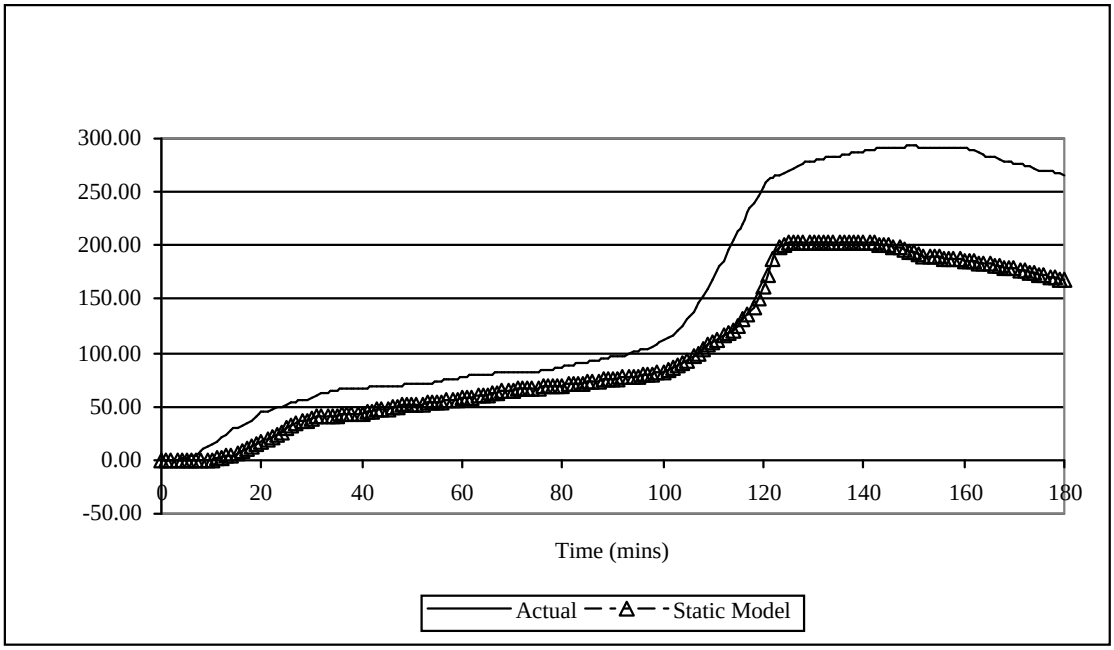
Comparison of column lateral deflection versus temperature – Corner test
Figure 2.32



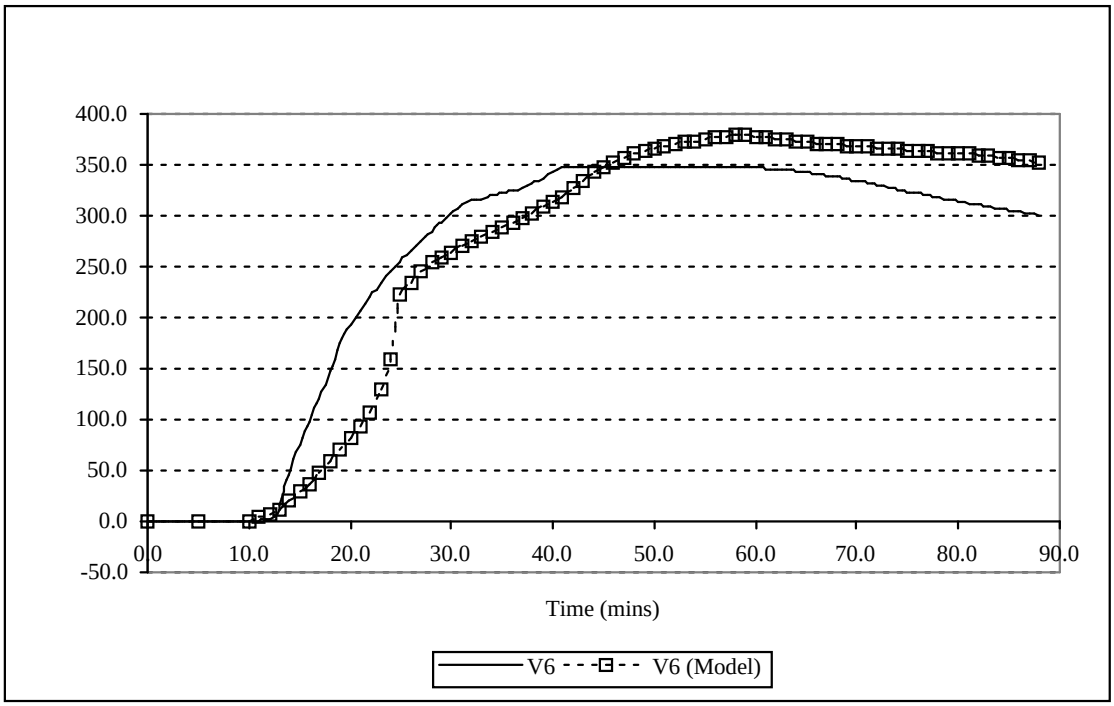
Comparison of column strains versus beam temperature – Corner test
Figure 2.33

Test 2/4

Extensive models were also constructed for Tests 2 and 4 using ADAPTIC. These are described in more detail elsewhere (MD15, AM7). Examples of the comparison between experimental response and numerical predictions for these two tests are illustrated in **Figure 2.34 and Figure 2.35**. Although reasonable comparisons were observed in these models, most of the effort on interpretation and examination of the results was directed to Tests 1 and 3 rather than Tests 2 and 4. In Test 2, i.e. the plane frame, in which the connections and parts of the columns were not protected, has specific features which make it a special, and possibly unrealistic situation. On the other hand, the fourth test, i.e. the demonstration test, did not include sufficient measurements to enable direct comparison with numerical results nor to justify detailed examination of the results.



Comparison of experimental vertical deflection and numerical prediction in Test 2
Figure 2.34



Comparison of experimental vertical deflection and numerical prediction in Test 4
Figure 2.35

3 INTERPRETATION

A degree of quantitative validation of the predictions of a model against test measurements can be said to have been achieved once individual quantities such as maximum beam and slab deflections, strains and column displacements have 'all' been compared with test data and shown to agree within reasonable margins. Qualitative validation may also be achieved by the prediction of various events that are known to have occurred in the tests at particular temperatures or times. Once a satisfactory level of validation has been achieved the data from the models can be processed and analysed much more thoroughly to determine more detailed information about the structural response that cannot be obtained from the test results. This 'additional' information is the main purpose of the modelling as it is only through this process that the detailed structural behaviour can be interpreted. The interpretations of structural behaviour can provide further validation of the model by assessing the consistency of the results with the fundamental principles of structural mechanics. Completion of this step by step process leads to a much deeper understanding of structural behaviour. The following sections provide an example of such a process applied to the interpretation of the results from various models of the Cardington tests developed in this project.

3.1 Interrogation of computational models to develop understanding (AM1-7)

The large amount of data available from the modelling results was thoroughly processed to obtain detailed information on the behaviour of each individual structural element for the whole duration of the fire. This was done by downloading the analysis output data to graph plotting packages (such as Excel and Gnuplot) and then manipulating the numbers to derive a much larger variety of information on forces and displacements at many locations and for the whole range of temperatures. This was done for all the different models developed. The following list gives the types of quantities that were derived from interrogating the output data (with reference to the grillage models).

- Axial forces in all the members (primary and secondary steel joists, longitudinal slab and transverse ribs)
- Moments in all the members
- Shear forces in selected members
- Total axial forces in the composite (steel joist + slab) by adding the individual axial forces
- Composite moments (by adding the individual moments and moments of individual axial forces at a reference plane)
- P-Δ moments by integrating axial force times midspan deflections over the length of the beams
- Differences between end moments and midspan moments to obtain a measure of the notional equivalent uniformly distributed load on a composite beam

This data was plotted in a variety of ways, including

- Variation of a force quantity at particular locations (points) against reference temperatures (normally chosen as the temperature of the bottom flange of the hottest steel joist)
- Variation of a force quantity at particular temperatures over the whole length of the member

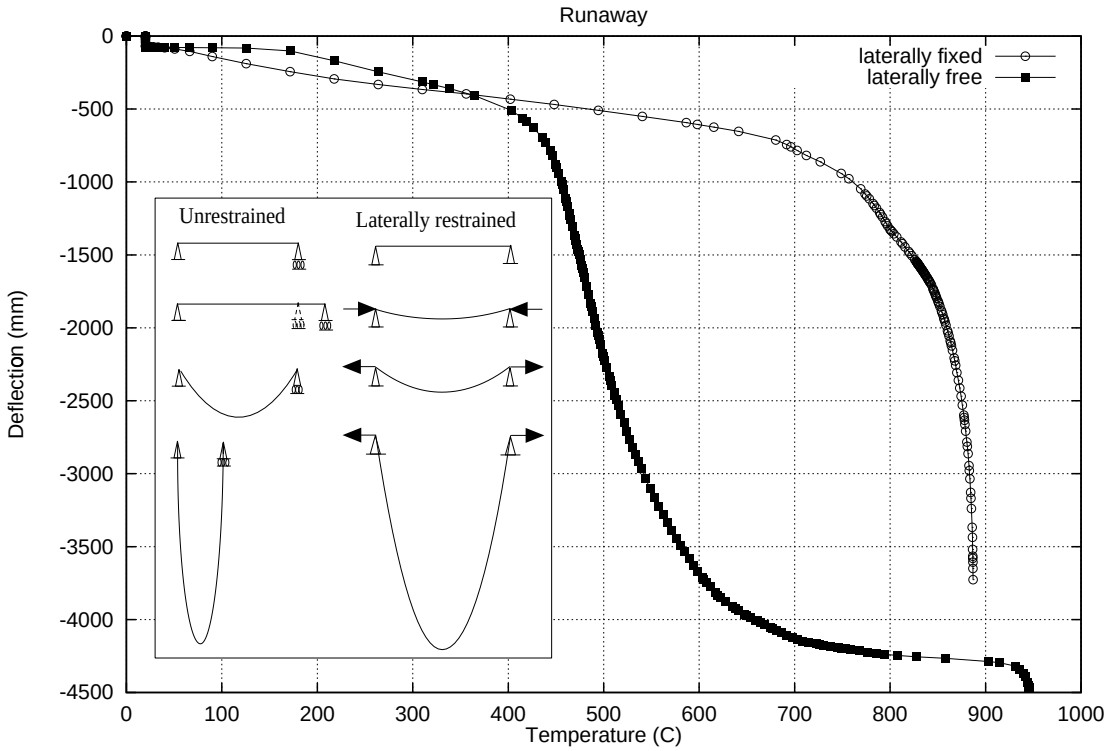
The above processing was reasonably straightforward to perform for the results of the grillage models. As mentioned in the previous chapter, the ease with which output data can be presented is one of the biggest advantages of the grillage models. These models allowed the interpretation of their results to be carried out in a simple and tractable manner over the course of heating. Considerable understanding was achieved in terms of the development of forces in the various members, their interactions and redistribution mechanisms. Most of the above types of processing were also performed for the results of the more rigorous shell models to allow direct comparisons to be made between the two types of model in terms of the predictions of structural response. This was, predictably, a much more painstaking task and equivalent graphs as developed for the grillage models were produced by integrating quantities within finite strips of slabs. A cross-section of the results appears later in the discussion of the main findings.

3.2 Theoretical models (TM1-4)

As mentioned earlier, in parallel to the modelling and data interrogation work, theoretical work on setting out the fundamental principles of structural responses to thermal actions took place. This was so that the interpretations derived from processing the model outputs could be thoroughly tested for consistency with the principles of mechanics. In developing the theory and analysing simple test cases, many counter-intuitive phenomena emerged, which were also later discovered in the model results. Most of the theory and principles developed are reported in the Appendices (reports TM1 and TM4). A brief discussion of these principles is presented in the following paragraphs.

Figure 3.36 shows a simple comparison of two geometrically non-linear analyses. The first case is heated, simply supported (laterally unrestrained) steel beam with a uniformly distributed load and the second is a laterally restrained (but rotationally unrestrained) beam with the same uniformly distributed load. Initial deflections are lower in the first beam because the supports are able translate outwards upon expansion. However, ‘runaway’ occurs at around 450 °C (even though considerable steel strength remains) mainly because of pulling in of the supports when the flexural stiffness of the beam reduces to a point where it cannot sustain the imposed load and there is nothing to restrain the growing deflections. In the second beam larger initial deflections occur because the beam ‘buckles’ due to the restraining forces very early on (70 °C) and further increases in length due to thermal expansion can only be accomodated in deflection. But runaway does not occur until much later (900 °C) when the steel properties are completely lost. This illustrates that the presence of restraints to end translation delays ‘runaway’ to much higher tempratures because of development of catenary action to replace the highly depleted flexural stiffness. The second beam is a much more appropriate model for beams in large redundant structures. In real structures not only is this restraint available but the steel beam is in

composite action with the concrete slab which produces a much stronger and more robust structure. This strength and robustness is enhanced by the redistribution mechanisms present in redundant structures (for instance the load may be carried by the transverse slab supported in tensile membrane action which retains its strength for much longer than the steel beams). The question may now be asked, what about the large deflections seen in real structures? Are those not a clear sign that 'runaway' was occuring? **Figure 3.36** clearly shows that for temperatures below 300 °C, the deflections for the restrained beam are much larger than that for the simply supported



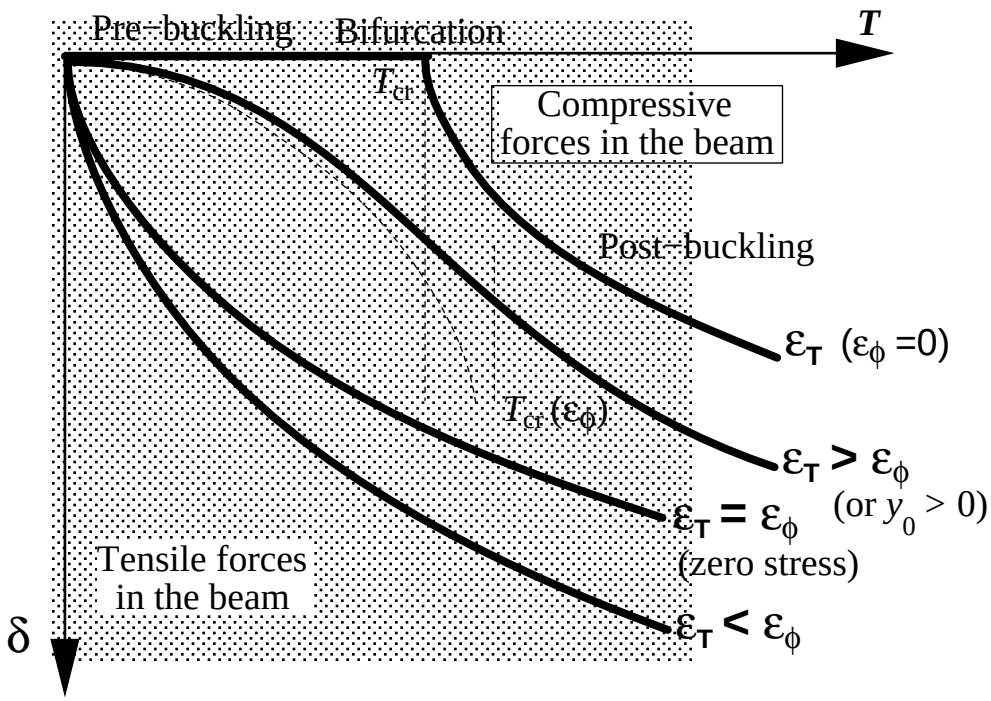
Runaway in a simple beam model
Figure 3.36

beam, however they have nothing to do with 'runaway'. These deflections are caused entirely by the increased length of the beam through thermal expansion and are not a sign of loss of 'strength' or 'stiffness' in the beam until much later. In fact approximately 90% of the defelection at 500°C and 75% at 600°C is explained by thermal expansion alone. Most of the rest is explained by increased strains due to reduced modulus of elasticity. However the behaviour remains stable until about 700°C when the first signs of runaway begin to appear.

The main purpose of the above exercise was to indicate that strength reduction and loads are secondary phenomena when analysing redundant composite structures in fires (assuming realistic maximum temperatures). The response of such structures is overwhelmingly controlled by the displacements imposed by thermal actions of expansion and bowing, and the compatibility of displacements. The degree of restraint (translational and rotational) to displacements determines the forces and moments that will occur in the structure. A whole range of responses exist for combinations of thermal expansion (caused by rise in average temperature) and thermal bowing (caused by transverse temperature gradients). **Figure 3.37** illustrates

these interactions graphically. Reports TM1-3 provide considerable detail of these effects, however a few points that may be used to analyse model results can be stated as:

1. Unrestrained thermal expansion caused by a rise in mean temperature causes ends to move apart. The thermal strain producing this expansion is $\varepsilon_T = \alpha\Delta T$ (where α is the coefficient of thermal expansion and ΔT is the average temperature increase).
2. Thermal expansion in the presence of restraint to lateral translation from the surrounding structure produces compression forces leading to yielding or buckling (both the restraint and the temperature rise do not have to be large for buckling or yielding to occur).
3. Thermal bowing caused by the through depth thermal gradient leads to curvature resulting in the pulling in of the ends in a simply supported beam (Let the reduction in distance between the ends be written as a "contraction" strain ε_ϕ to understand the interactions between thermal expansion and thermal bowing as shown in **Figure 3.37**).
4. Restraint to end translation produces tensions in the beam which grow with growth in thermal gradients.
5. Restraint to end rotation produces hogging moments (and no curvature).
6. Compatibility of displacements in compartments with orthogonal stiffness distribution and orthogonal temperature distribution (steel only in one direction) also influences the forces in the members.



Thermal bowing and thermal expansion interaction illustrated
Figure 3.37

3.3 Main findings from modelling (AE1, AM1-7, SM1-5)

In this section a summary of the main findings from the modelling and interpretation work undertaken are presented.

3.3.1 Description of structural behaviour from Test 1 and Test 3

The majority of the detailed processing work was undertaken on the models of Tests 1 and 3. These two tests adequately cover two of the most important cases for small compartment fires. The main features of Test 1 in terms of the principles of behaviour are as follows:

- Highly restrained internal compartment
- The behaviour of the single composite beam (secondary steel joist and longitudinal slab) in terms of its interaction with the end restraints and the transverse ribs can be studied in detail without more complex interactions masking it in larger compartments
- Rectangular compartment (3m x 8m) with unequal thermal expansion in the two directions and therefore compatibility issues have a major influence on behaviour

Test 3 has different features and covers most of the issues that are not covered by Test 1, such as:

- Exterior compartment with two sides on the structure boundary resulting in high restraint to interior ends of members and much lower restraint to the exterior ends
- More 'square' in plan (10m x 8m), compatibility still important because of the highly orthogonal distribution of stiffness and thermal properties. Steel joist acts compositely with the longitudinal slab in the long direction and slab ribs in the other (so higher average temperature on the composite beam than in the ribs)
- Variety of different beams to study; unprotected secondary composite beams between columns and between primary beams, fire protected edge beams and unprotected primary beam.
- Over two continuous rib-spans interact with two heated secondary beams, a protected edge beam and the cooler and stiffer interior slab.

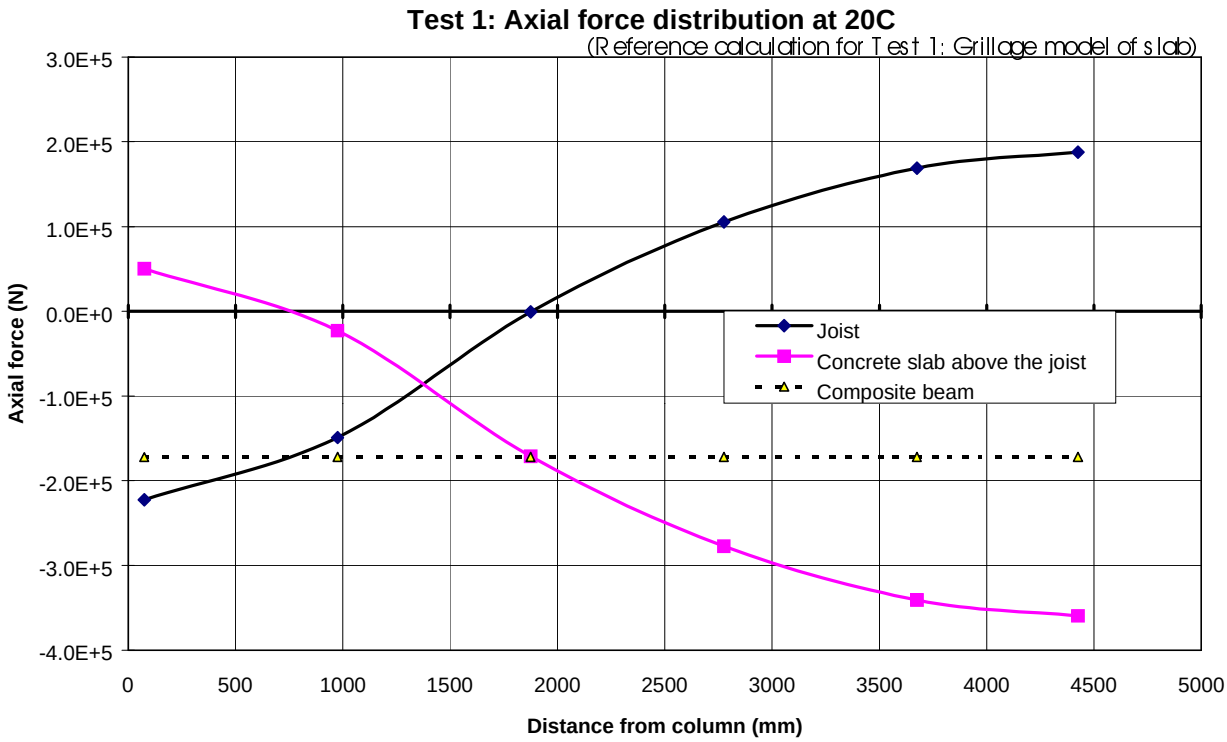
Therefore it was possible to examine a large number of different features of behaviour by concentrating on the analysis of model results from these two tests.

3.3.1.1 Discussion of Test 1

Descriptions of the structural behaviour in Test 1 can be found in a number of reports in the Appendices (MD1-6, MD15, AM1-3, AM7 and SM1-4). Further descriptions can be found in technical papers (see list of publications [4-6] in the appendix). The amount of information processed out of the results of Test 1 models is too voluminous to be presented here, even in summary, however a brief description of the most important features is presented with the help of a few selected figures over the full range of temperatures. Most of the results presented are from the reference grillage model (MD1). There are clear differences between the results from other more accurate models such as shell models, however these differences do not show a fundamentally different pattern of behaviour.

Ambient Temperature

Figure 3.38 shows the axial forces in the steel joist and longitudinal slab (acting compositely) over the whole length at ambient temperature (from the reference grillage model – MD1), which are as expected. The most interesting thing to note is the 'net compression' in the composite (sum of the two forces). This demonstrates the model picking up the *compressive membrane action.*



Axial forces along the joist and in slab at 20°C
Figure 3.38

Ambient -150 °C

Figure 3.39 shows the development of the axial forces in the joist and slab over the whole heating range for the ends (x/l=0.0) and the midspan (x/l=0.5). **Figure 3.40** shows the composite axial forces. The compressions at all locations grow until the first kink in the Figure, which corresponds to the yield of the steel joist at its ends (coincides with the local buckling observed Test 1). This is caused by a combination of restrained thermal expansion (at very low deflections) and the increasing hogging moment caused by the thermal gradients (because the ends of the composite are rotationally very stiff before the buckling event) seen clearly in **Figure 3.42** and **Figure 3.43**.

150 °C - 200 °C

The deflections begin to grow at a higher rate and **Figure 3.44** shows the P-Δ moments increasing rapidly as a consequence. The rise in hogging moments at the ends reachs approaches its peak value (slab tensile capacity is reached) and the end

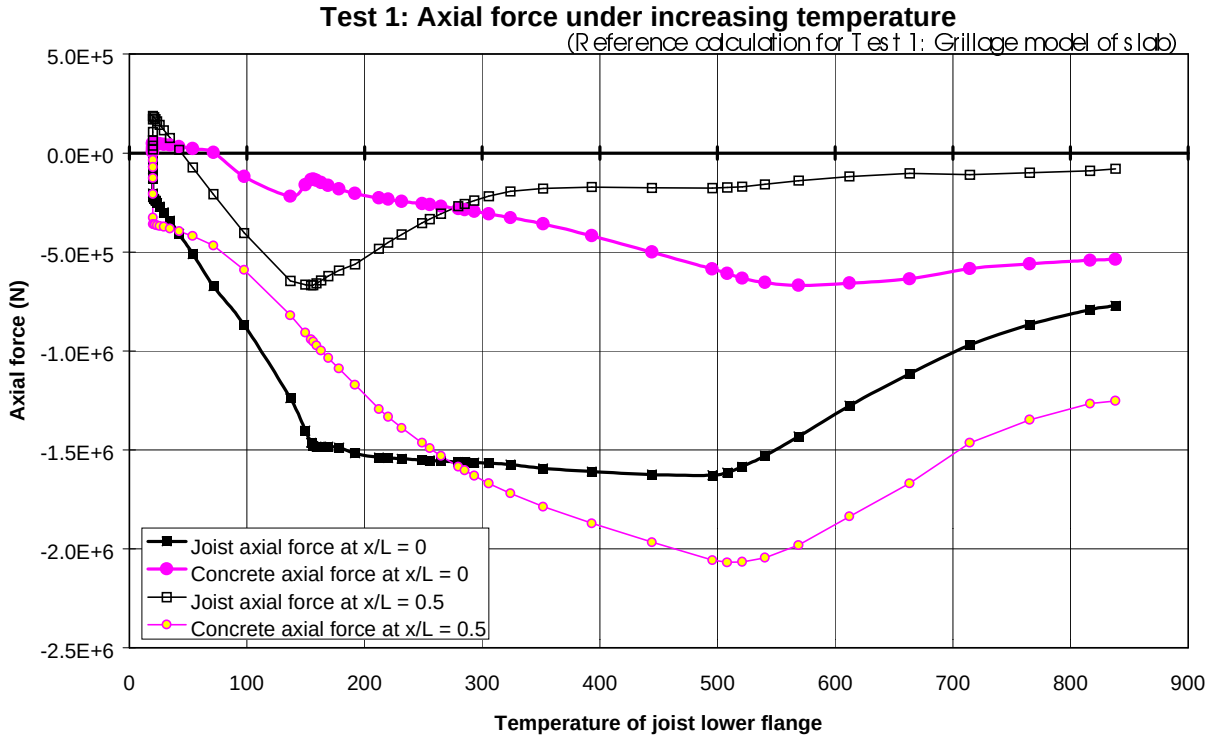
rotational restraint is completely lost (see **Figure 3.42**). Moment redistribution to midspan begins (**Figure 3.42** and **Figure 3.43**). Concrete participates in the composite moment carrying capacity only through developing axial forces (see **Figure 3.41)** showing negligible moments in concrete, therefore absence of axial-flexural interaction in the model does not have serious consequences). The effect of hogging moment growth is also reflected in the continued increase in compression in the ribs near the ends (due to very low deflection caused by the hogging curvature until about 200 °C) as shown in **Figure 3.45**. The same figure shows that the axial compressions in the central ribs begins to decrease after 150 °C as the deflections begin to grow near the midspan.

200 °C - 500 °C

The next major event is caused by the steel joist reaching its ultimate axial capacity at approximately 500 °C, beyond which the compressions follow the path determined by steel capacity. **Figure 3.42** shows the end and midspan moments in the composite. The difference between these moments gives a measure of a notional uniformly distributed load carried by the composite beam. This difference shows that over twice the imposed load is actually experienced by the beam because of thermal effects, also shown in **Figure 3.42** (as end-to-centre difference). However this capacity is seriously eroded when the steel joist begins to degrade in strength after 500 °C.

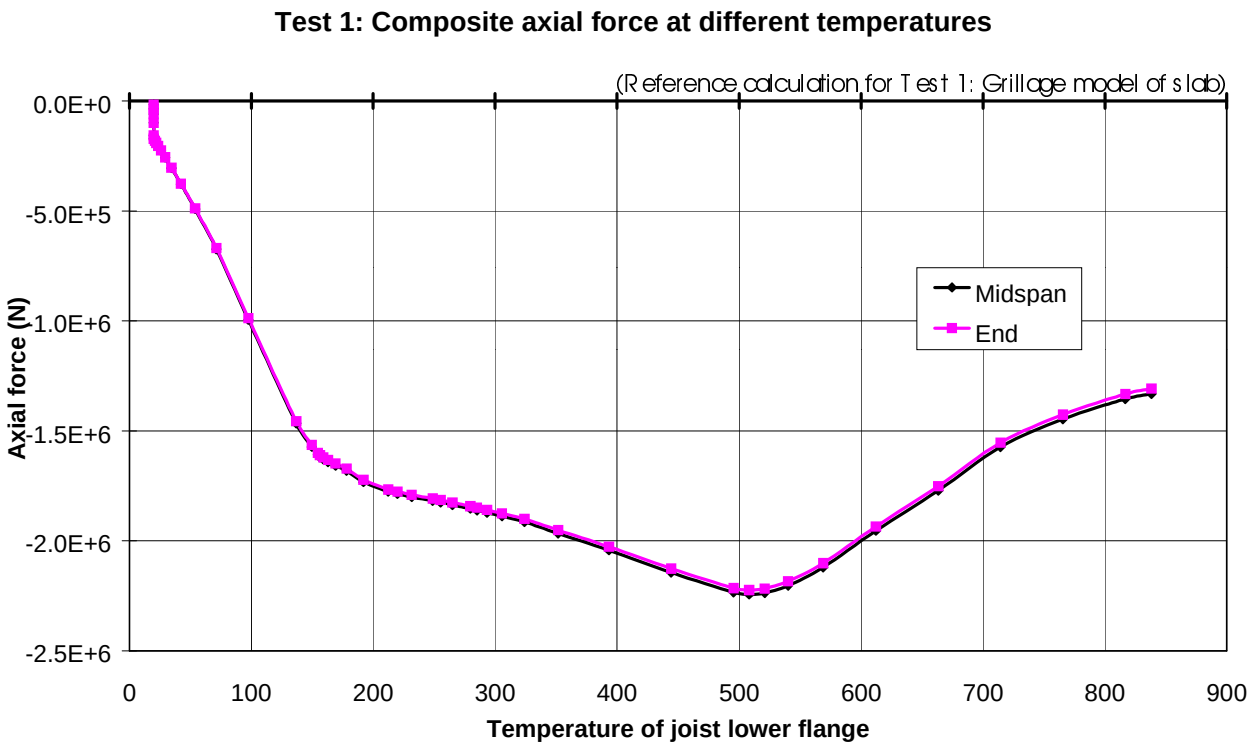
500C - 800 °C

The P-Δ moments continue to grow beyond 500 °C (mainly through increasing Δ) and peaks at 600 °C. This is extra loading in the face of rapidly reducing capacity as illustrated by **Figure 3.42** (where the end-to-centre difference crosses P-Δ moments). This point suggests that from here on the composite beam is carrying less than the moment applied on it at midspan. **Figure 3.45** shows that between 500 °C and 600 °C the ribs in the middle 20% of span move into tension and from thereon this tension increases until the tensile capacity of the ribs is reached and it stabilises. This is a clear illustration of tensile membrane redistribution from the longitudinal to the transverse direction. The ribs near the ends of the compartment remain in significant compression, due to much lower deflections, and also redistribute loads to the parallel secondary beams, albeit in compressive membrane action.



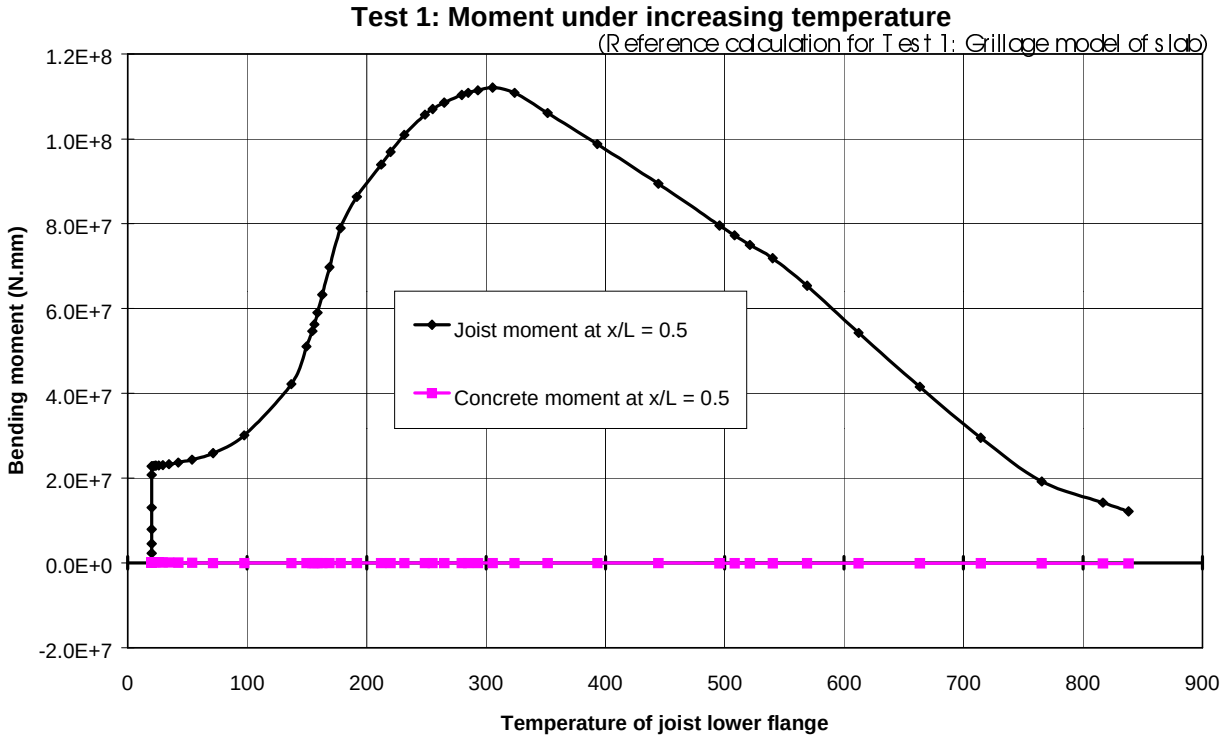
Axial forces along the joist and in slab at higher temperatures [at mid-span and ends one figure]

Figure 3.39



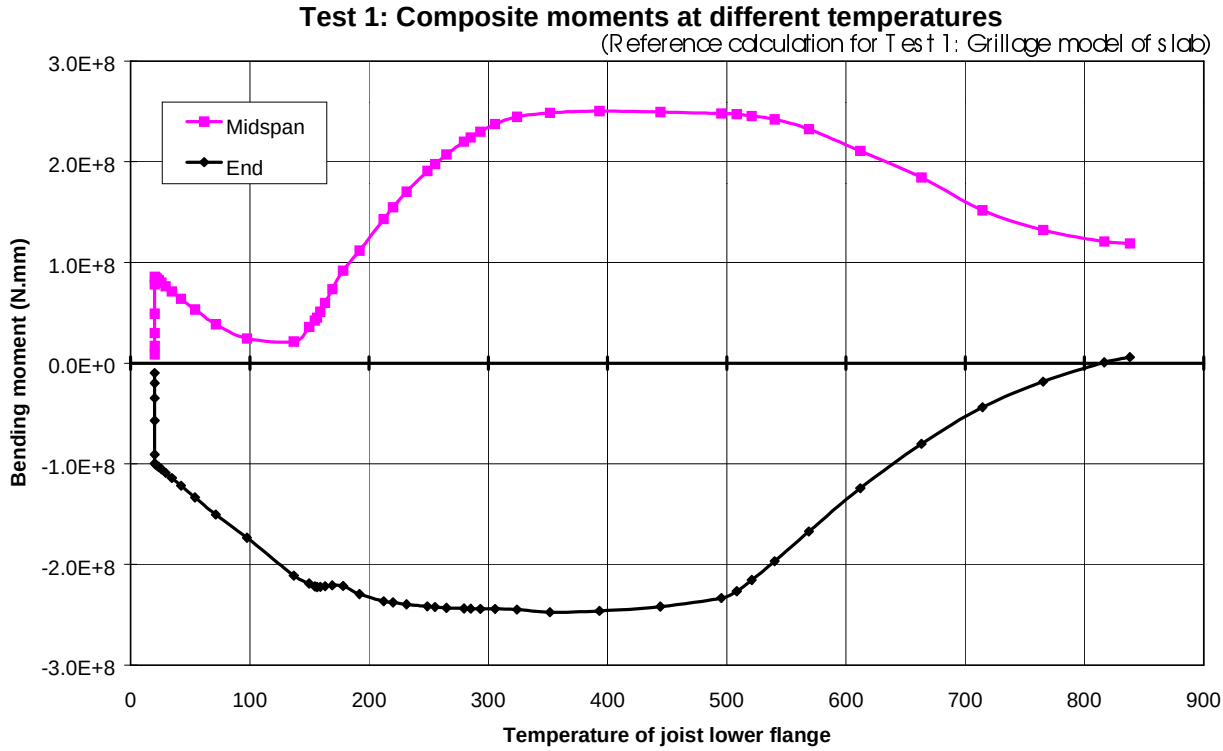
Composite axial force over the beam against temperature

Figure 3.40



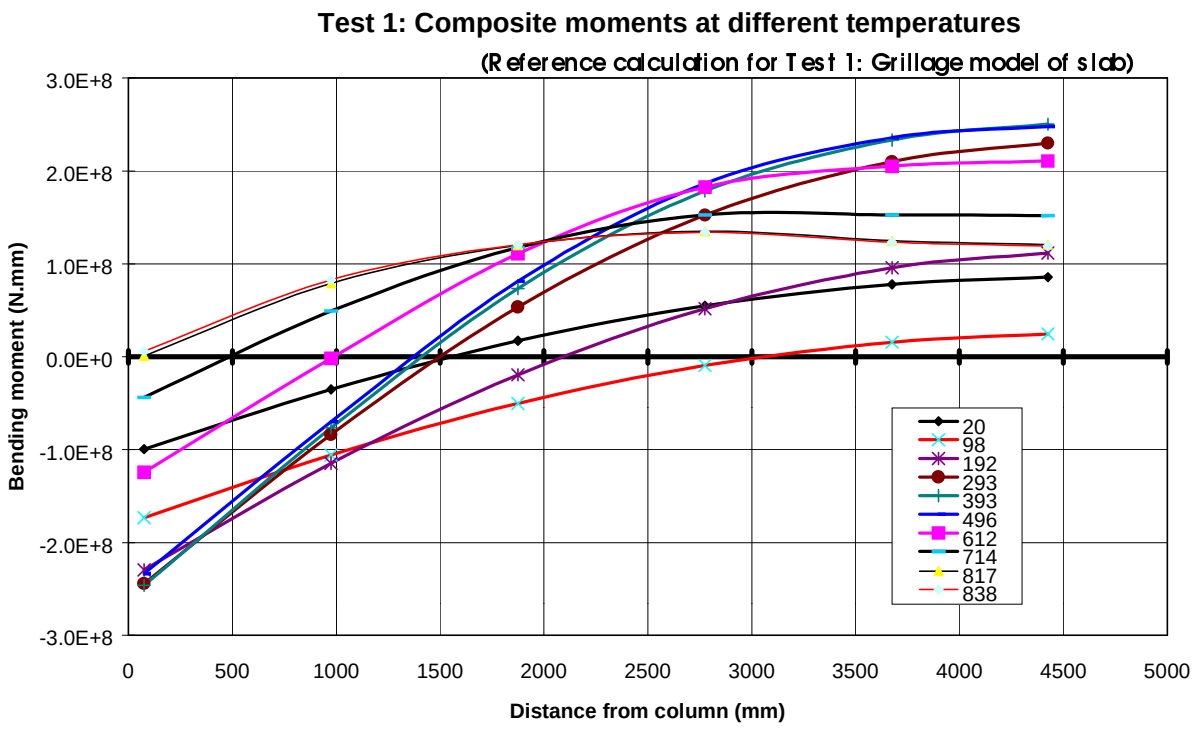
Joist and slab moment at mid-span

Figure 3.41



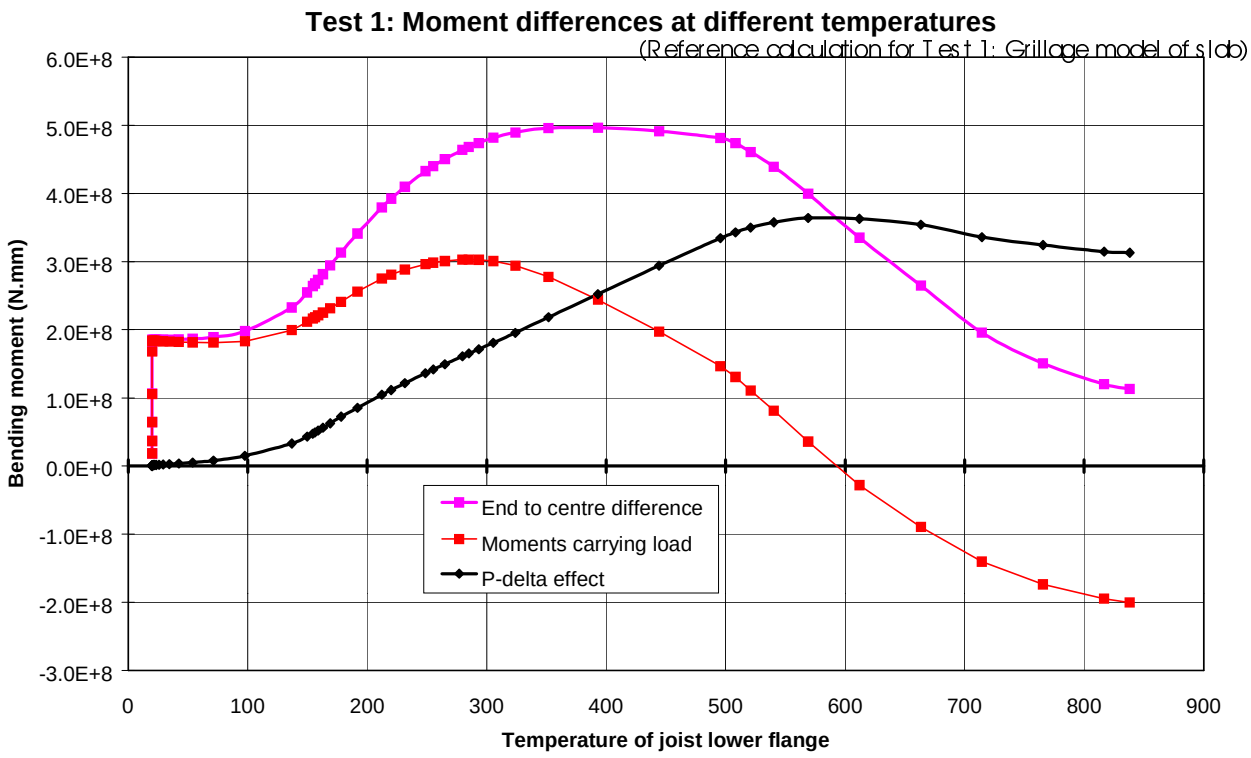
Composite moment at support and at mid-span against temperature

Figure 3.42



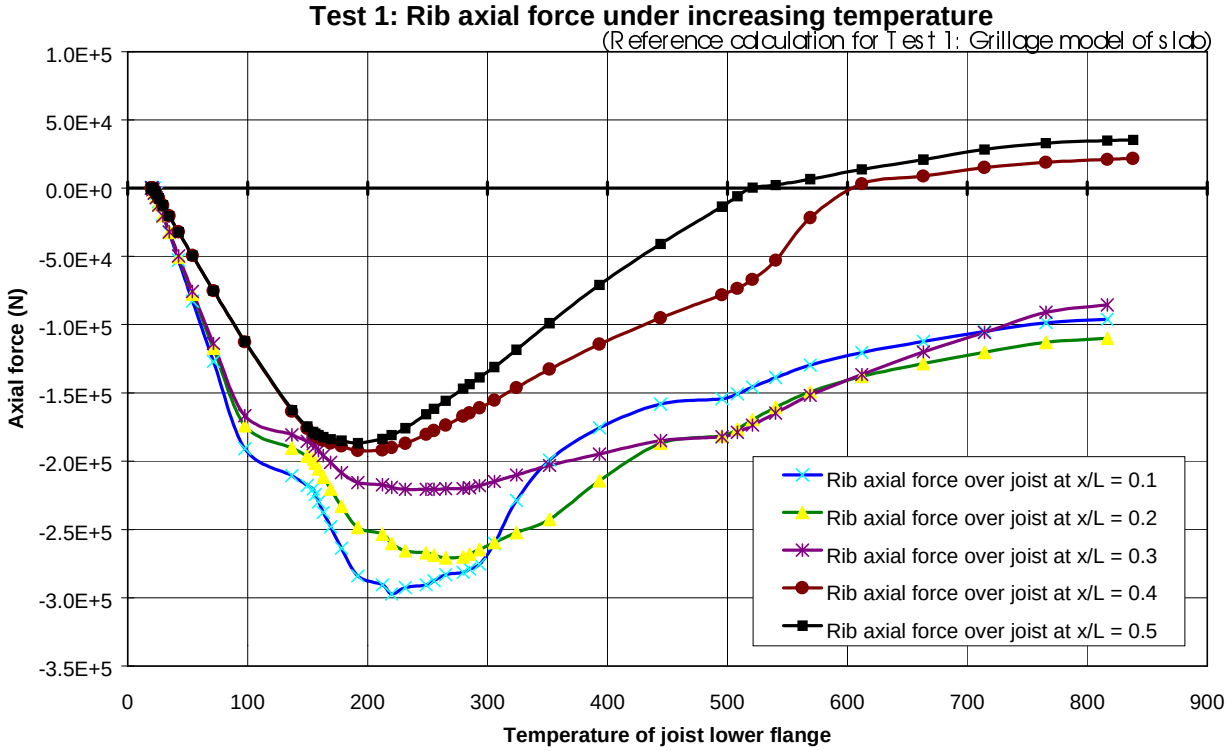
Composite moment along the beam

Figure 3.43



Moment carried by the beam and moment applied (P-Δ)

Figure 3.44



Forces developed in the transverse direction

Figure 3.45

Comparisons with other models

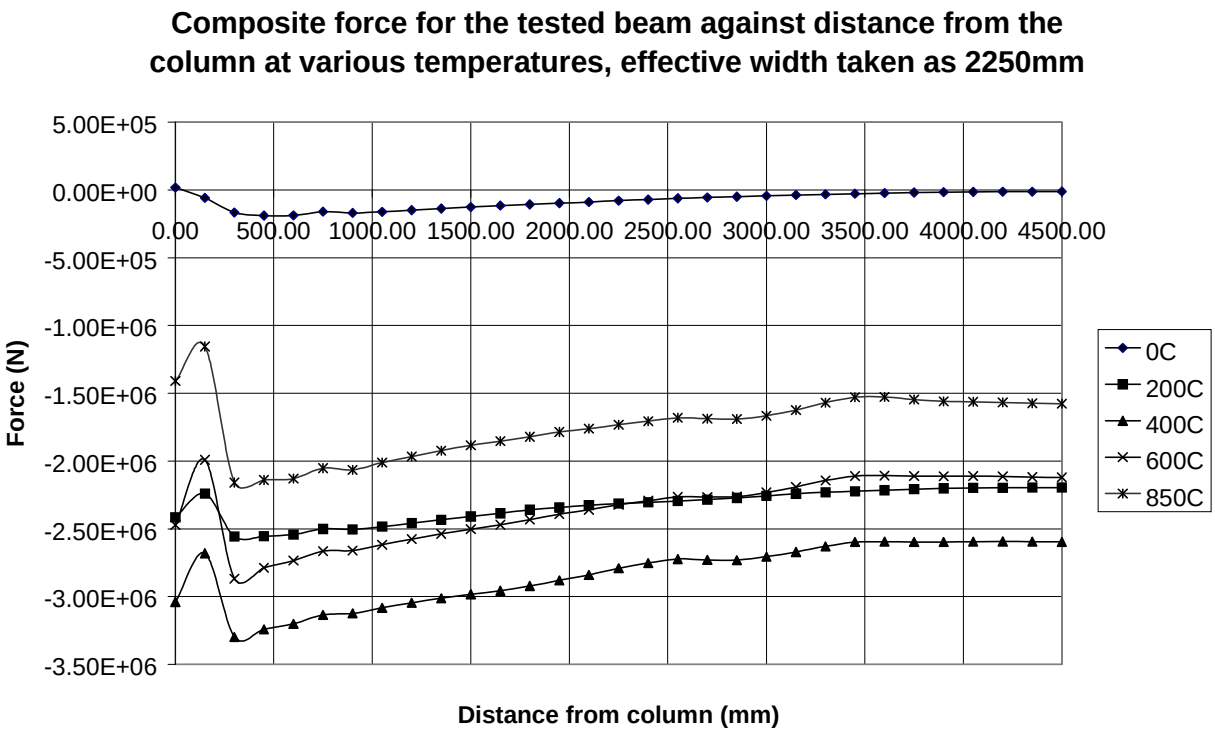
The description above is from the reference grillage model (see report MD1). The same broad picture has been produced by all the other models with obvious differences arising from the type of modelling undertaken.

Figure 3.46 shows the total axial force derived from a stress resultant shell model (described in report MD3) in the steel joist and a strip of slab of width equal to that assumed in the grillage model to act compositely with the beam. This when compared to the axial forces in **Figure 3.38** from the grillage model matches very well, both in terms of distribution and magnitude. The notable differences being; reduction in the shell axial forces at the compartment boundary, which is a result of redistribution to the sides of the compartment caused by local buckling of the beam at 150 °C; reduction in axial force near the midspan (although still in considerable compression) which can be explained by the larger deflections in the slab at midspan and the compressions being concentrated nearer the stiffer compartment boundaries with lower deflections.

Figure 3.47 shows axial forces in the steel joist from the same shell model (compare with **Figure 3.39**); again the differences are in the detail rather than the pattern of behaviour. The magnitudes match well other than for much lower axial forces in the midspan section of the beam in the grillage model. These may be explained by lower restraints to expansion provided by the grillage model ribs compared to the much more uniform distribution of this restraining stiffness in the shell model.

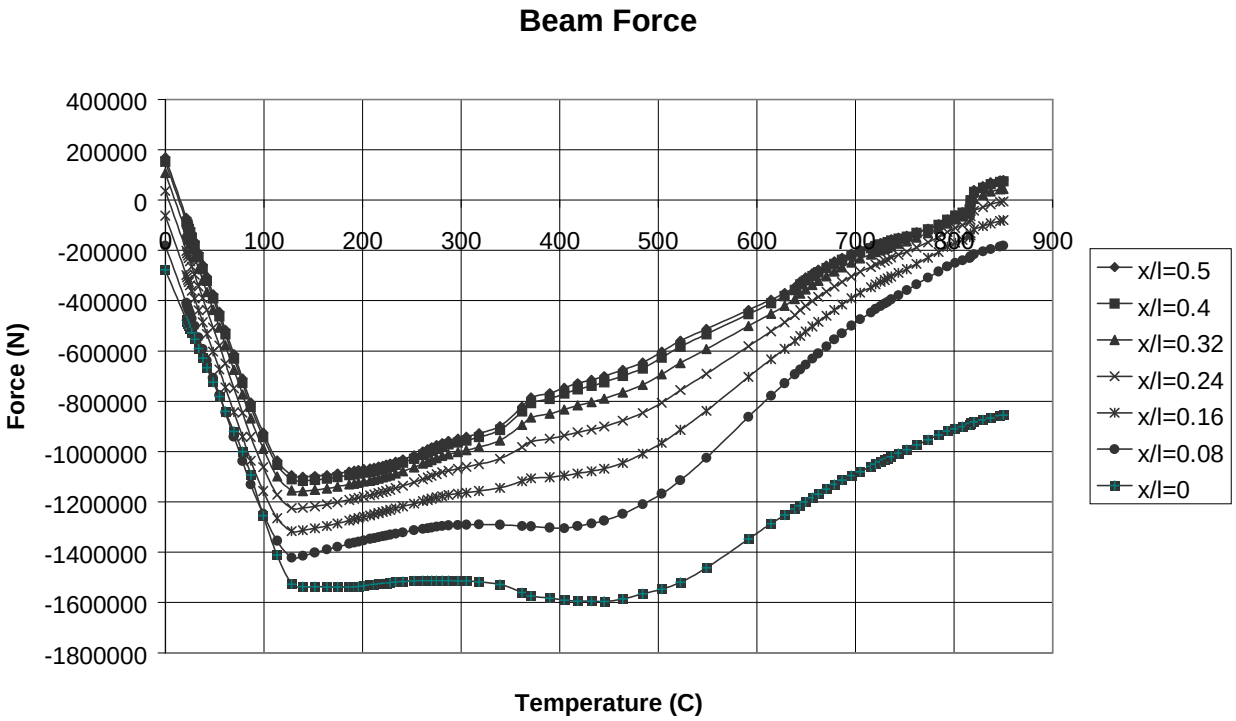
Figure 3.48 shows 'rib' forces extracted from the shell model. These are considerably different from **Figure 3.45** (grillage model) in that they show the central ribs to increase in tension from the start. This difference is explained by the evolution of heating in the grillage model, which was assumed to be linear (from zero to the maximum value for both slab and joist) while for the shell model the heat evolution followed that measured in the test. This made the slab a lot hotter earlier on in the grillage model and hence the compressions developed. In the shell model the composite beam deflected a significantly before the ribs started heating which made them go from the initial hogging tension to higher tensions imposed by deflection compatibility. This is a much more consistent explanation from the compatibility point of view (shorter side expanding much less then the longer one and hence developing tensions from the very start).

Figure 3.46 also shows that the end rib is in compression as seen in the grillage models, however the difference is that this compression increases to the end of the fire, while the one in the grillage model reaches a peak at approx. 200 °C, stabilises, and then decreases. This is also very likely to be the effect of the difference in thermal regimes applied to the two models.



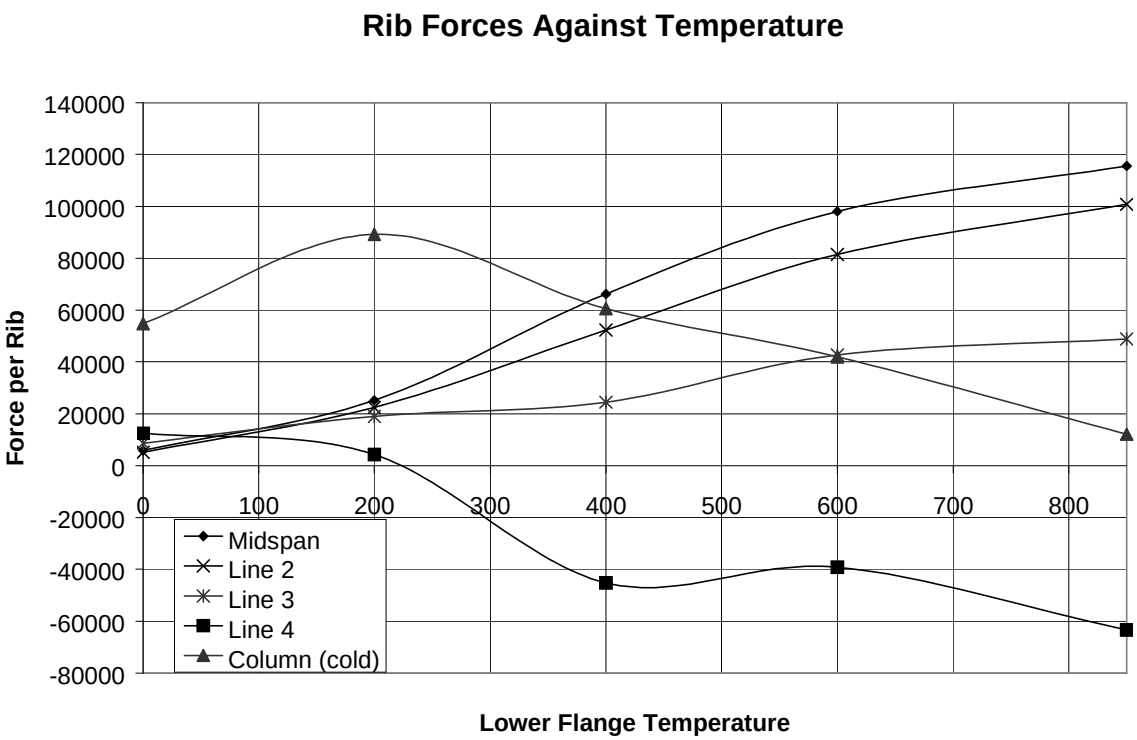
Composite axial force (FEAST model)

Figure 3.46



Joist axial force (FEAST model)

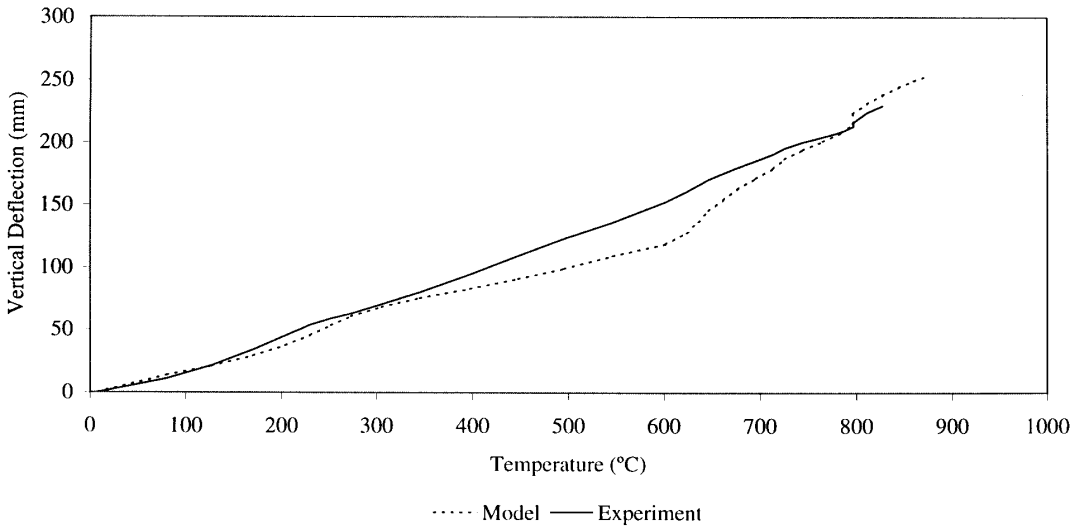
Figure 3.47



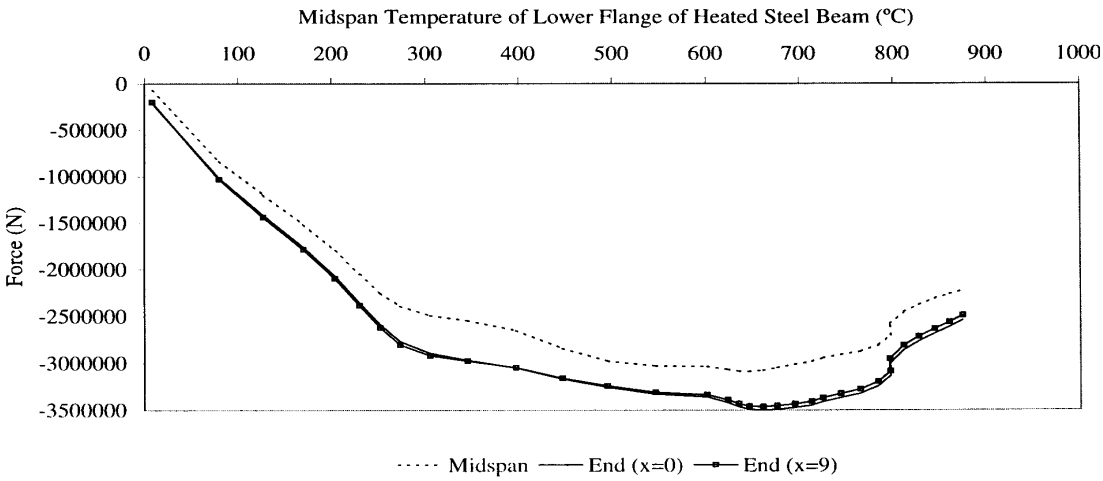
Rib forces (FEAST model)

Figure 3.48

The results obtained from the grillage models undertaken by ADAPTIC (MD15, AM7) produced very similar observations and findings to those discussed above. **Figure 3.49**, for example, depicts the midspan vertical deflection of the restrained beam obtained from one of the grillage models of ADAPTIC in comparison with the experimental result. Another illustration is given in **Figure 3.50** which shows the axial force in the composite beam at midspan as well as at the ends of the restrained member. The same trends obtained previously using other models, were again confirmed by the results of ADAPTIC. The discrepancy in the actual quantitiative comparisons between the various models was within the expected range caused by differences in the modelling assumptions adopted.



Comparison of experimental and numerical midspan deflection
Figure 3.49



Composite axial force at midspan and endpoints of heated beam
Figure 3.50

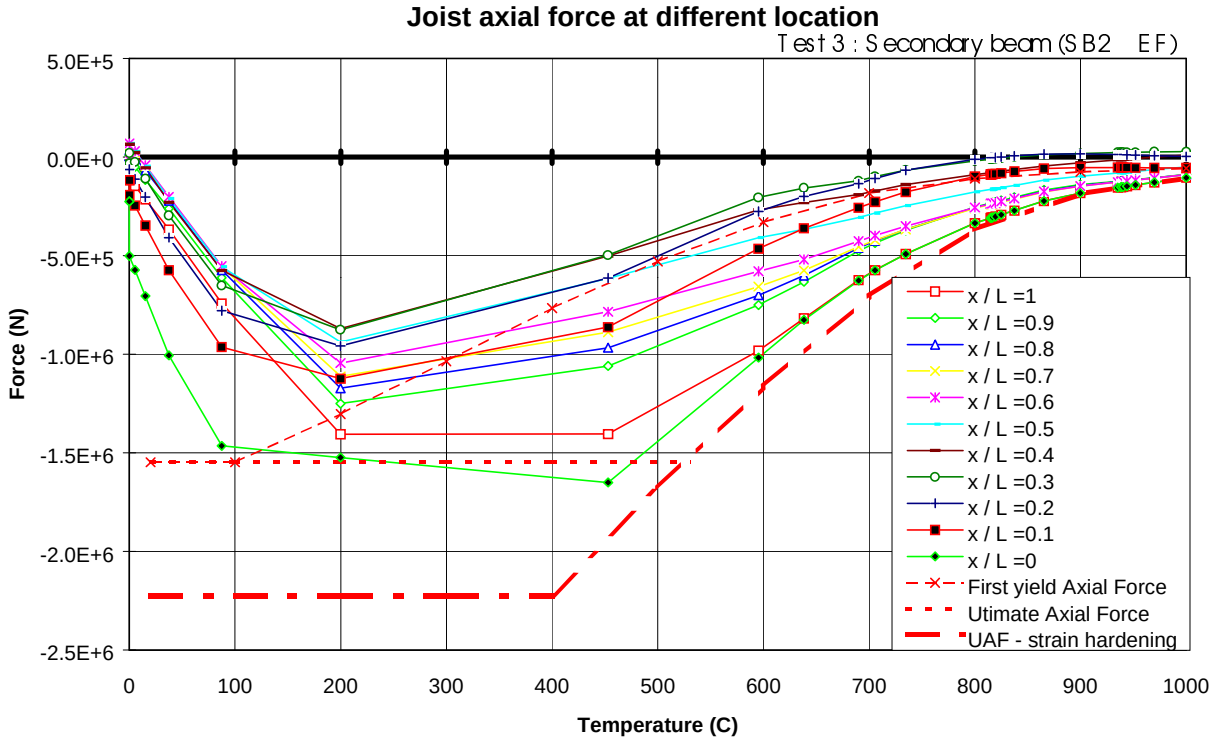
3.3.1.2 Discussion of test 3

The amount of information produced from analysing the modelling outputs from Test 3 is enormous and a significant proportion of it appears in reports MD10-13, MD15, AM4-6 and AM7, which must be read if a detailed understanding of the behaviour in Test 3 is sought. Only a flavour of these results is given here. It may also be noted that in modelling tests such as Test 3 it is much more difficult to elicit clear explanations from the models. There are many competing phenomena interacting with each other and several possible explanations may exist for any particular piece of structural response. The only way to get to the complete explanation is to compare the results from different types of models and quite often reanalyse with changed parameters until a satisfactory explanation is achieved. This is a very time and labour intensive activity. To a large extent this has been the way the research collaborators have functioned in this project however given the size of the task and limited resources some of this work remains to be completed. That said, the broad conclusions of structural behaviour are robust and consistent with the fundamental principles of mechanics briefly outlined earlier.

Figure 3.51 shows the Axial forces in the steel joist of the secondary composite beam on gridline 2(EF) between an interior and an exterior column (obtained from the grillage model reported in MD10). It may be mentioned here that this model which was a quite a good idealisation of Test 1, is not so appropriate for Test 3 due to much greater interaction between the structural members in both directions. This beam in Test3 is the closest in terms of its configuration to the beam in Test1, although it has a highly asymmetrical lateral restraint condition. Despite this, the development of compression at the two ends is not very different, suggesting that the restraint provided by the exterior column is still considerable.

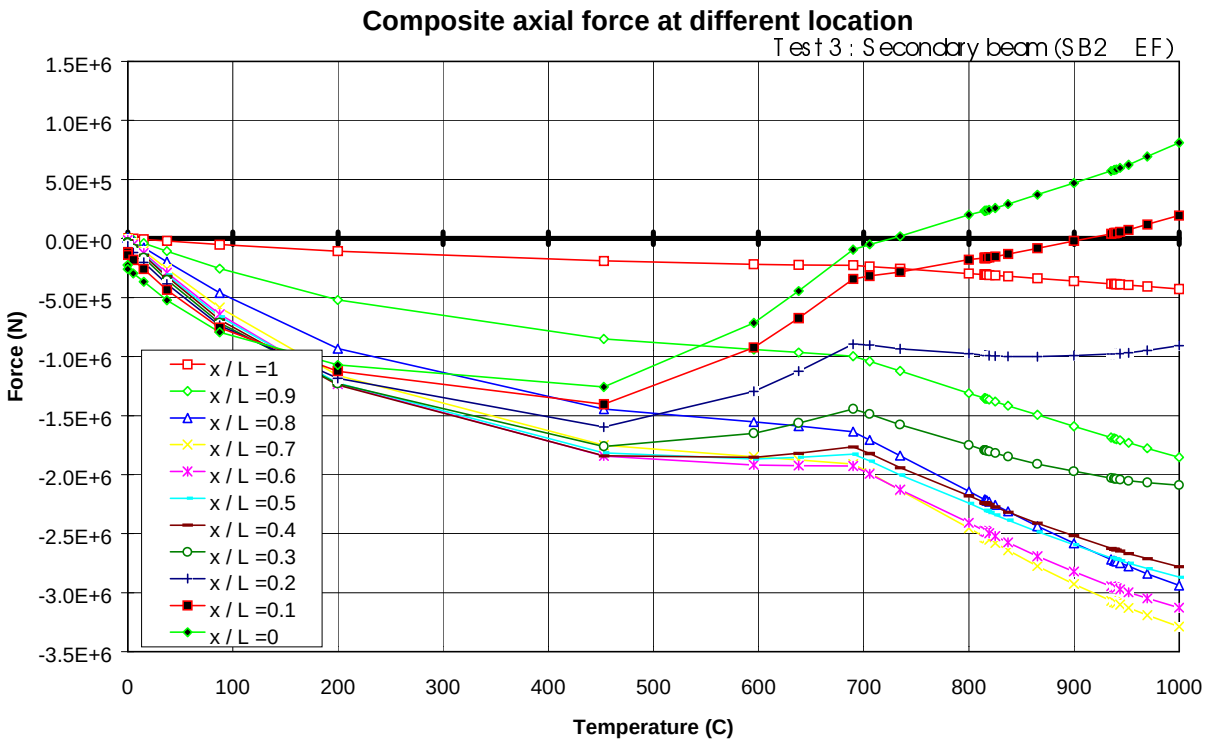
Figure 3.52 shows the total composite axial force for the same beam. This force is seen to be in low compression at the exterior column and going from compression to tension at the interior column. Comparing this with the previous Figure suggests that the slab attracted considerable tensions. A considerable amount of tensile hardening was allowed for in this model to obtain stable solutions, and this may have produced misleading results here. These high tensions may develop if the effect of the thermal gradients are dominant. The central parts of the beam remain in considerable compression. The kinks in the curves at 700 °C is to do with the thermal evolution in the slab, which was assumed to be piecewise linear, rising more steeply beyond 700 °C.

Figure 3.53 shows the axial force development in the central rib. It shows an increasing tensile force with increased heating and with distance from the edge beam. This tensile force is expected because of the compatibility condition discussed earlier. The increase in tensile forces towards the interior of the compartment is to do with the restraint being a minimum at the edge and maximum in the interior.



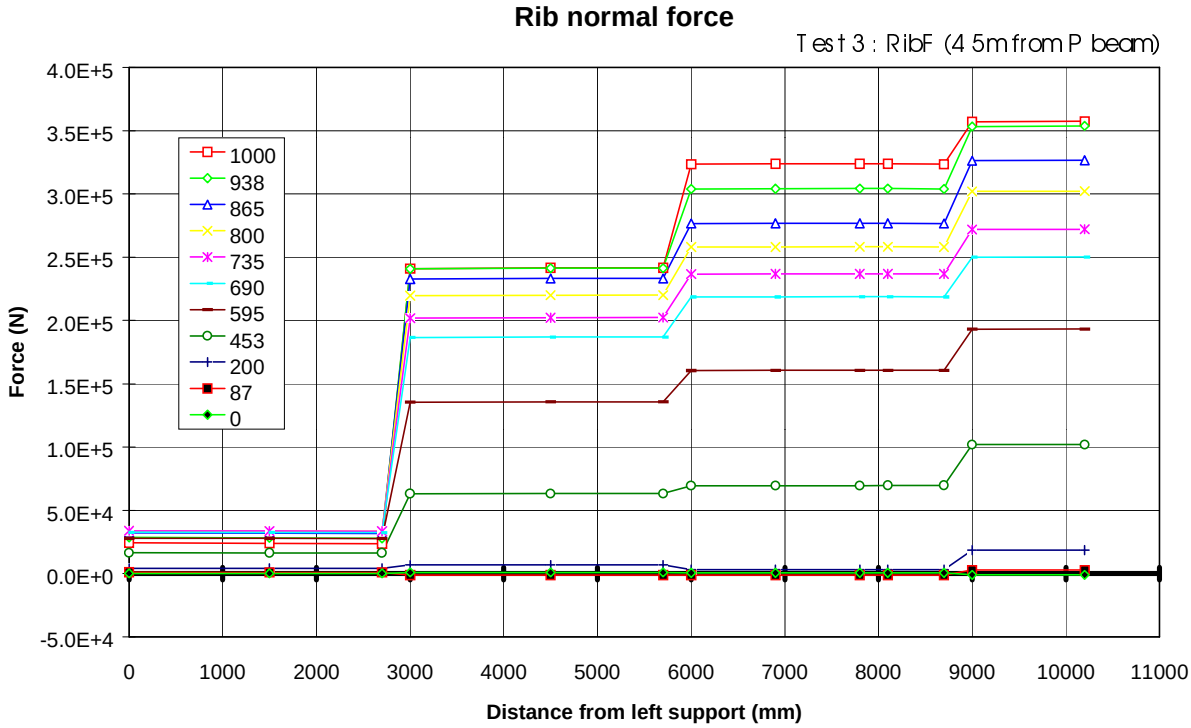
Axial forces along the secondary joist during fire in Test3

Figure 3.51



Composite force at different location of the secondary joist during fire in Test3

Figure 3.52



Axial force in rib at mid-span in Test3

Figure 3.53

Other grillage models for Test 3 were also constructed using ADAPTIC (MD15, AM7). The results obtained were again in general agreement with other models in terms of behavioural patterns and overall response. The results obtained from the numerical simulations of the corner compartment test further illustrated the salient influence of restraint to thermal expansion on the response, even when the floor area subjected to fire is not fully surrounded by other parts of the structure.

3.4 Studies with models (SM1-4)

This item covers a number of activities that took place in parallel with the main modelling work including some suggested by reviewers.

- Using elementary models to check certain structural behaviour phenomena
- Using elementary models to check ABAQUS capabilities
- Using "validated models" for a variety of parametric studies

Of the above item activities the first two are self-explanatory, the third item however covers a number of separate sub-activities, which can be listed as follows:

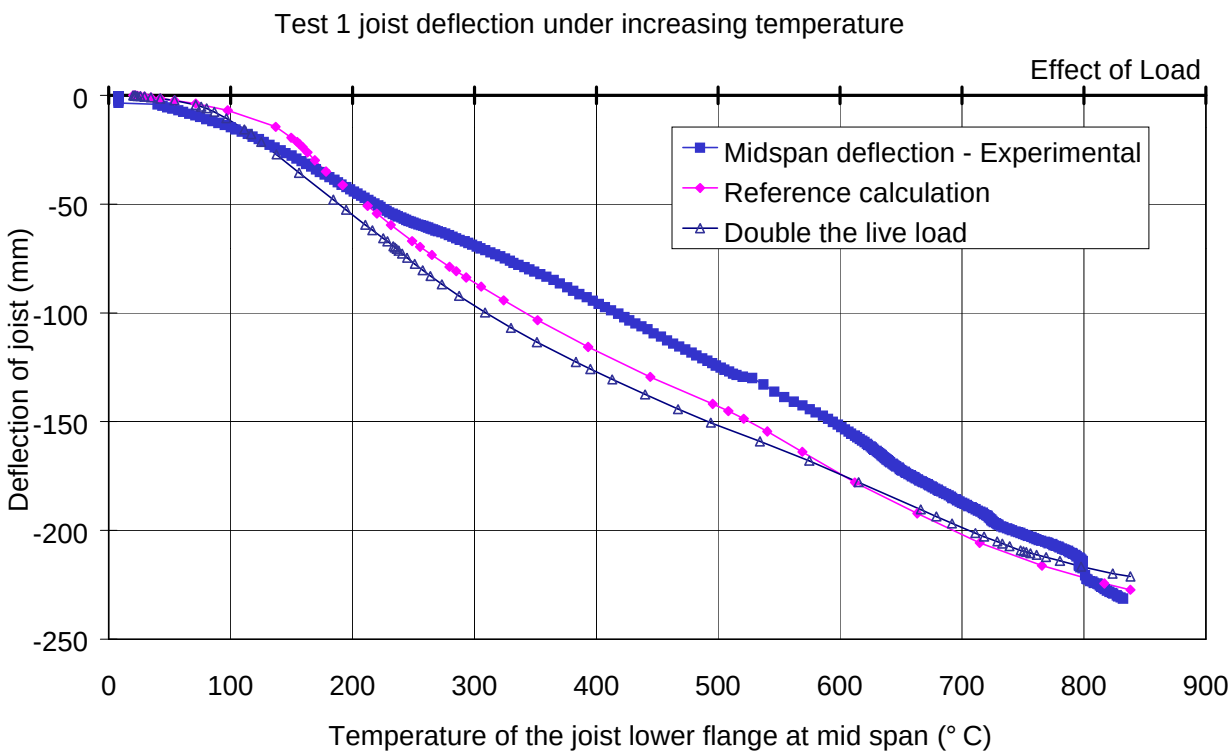
Identifying fundamental "weaknesses" in the model, in terms of the assumptions made and in terms of its limitations in modelling particular structural phenomena. Then identifying means of testing the models sensitivity to these weaknesses.

Sensitivity of the model to input parameters which are based on engineering judgement and represent a best estimate of an unknown value. Limiting values of such parameters may be used to test model sensitivity.
Identifying parameters relevant to fire design considerations and study model response to appropriate what if scenarios.

Some examples of calculations performed as a result of the considerations listed above appear in the following sub-sections.

3.4.1 Effect of loads

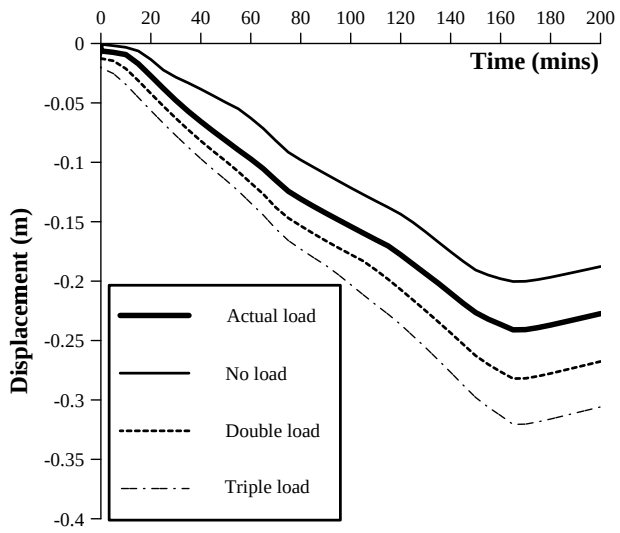
Figure 3.54 shows the result of an analysis performed using the reference grillage model (MD1) with double the live load. No significant change in behaviour is observed, confirming the contention that structural displacements in fire are dominated by thermally induced strains and restraints to them. A considerably more complete analysis was performed using ADAPTIC models on this issue, once again confirming the same conclusions, *i.e.* **static load doesn't matter** (clearly this implies remaining within the realms of the possible, limited by realistic maximum loads and realistic maximum fire temperatures and durations).



Effect of load on the deflection
Figure 3.54

In order to illustrate the influence of load level as well as thermal expansion on the response observed in Test 1, **Figure 3.55** depicts the vertical deflection at the midspan of the restrained beam as obtained from one of the numerical models undertaken using

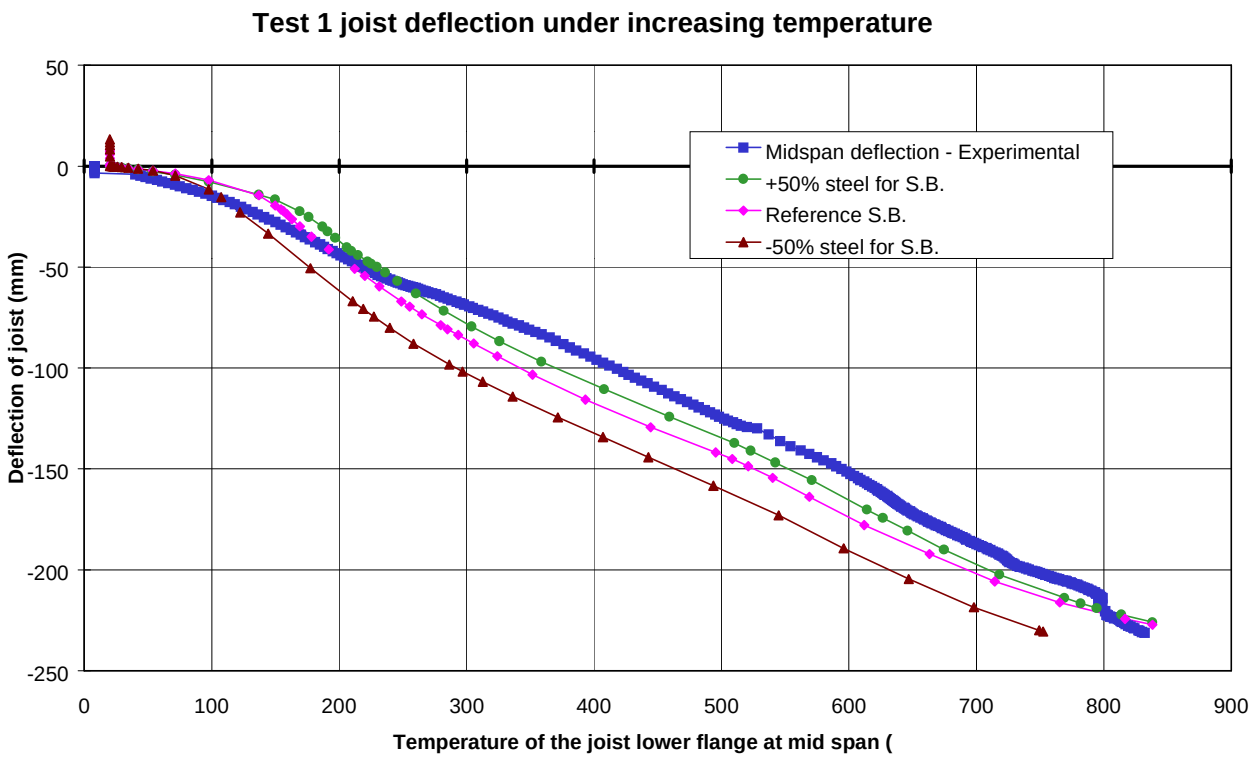
ADAPTIC (MD15, AM7). It should be noted that the maximum temperature in the steel beam corresponds to a time of about 160 minutes. In addition to the response under the actual load applied in the test, the analysis is also undertaken for a case where no load is applied and others in which double or triple the load is used. As indicated in the figure, doubling the load level or eliminating it all together causes a variation in the maximum displacement of ±15%. This clearly indicates that the large vertical deflections attained are mainly a consequence of restraint to thermal expansion.



Effect of load level on the response
Figure 3.55

3.4.2 Effect of section size variation

Figure 3.56 shows the results of varying the steel section used for the secondary joist by varying its area (and its second moment) by –50% and +50%. Again the results show only small changes in deflection. This is yet another confirmation of the contention that thermal actions dominate behaviour and that **strength does not matter** (with limitations indicated above). This study provides information on the effect of the variation of section sizes directly, however, it also suggests indirectly that the strength properties of steel do not have to be known with high a degree of accuracy and an estimate based on available data is sufficient in this context.



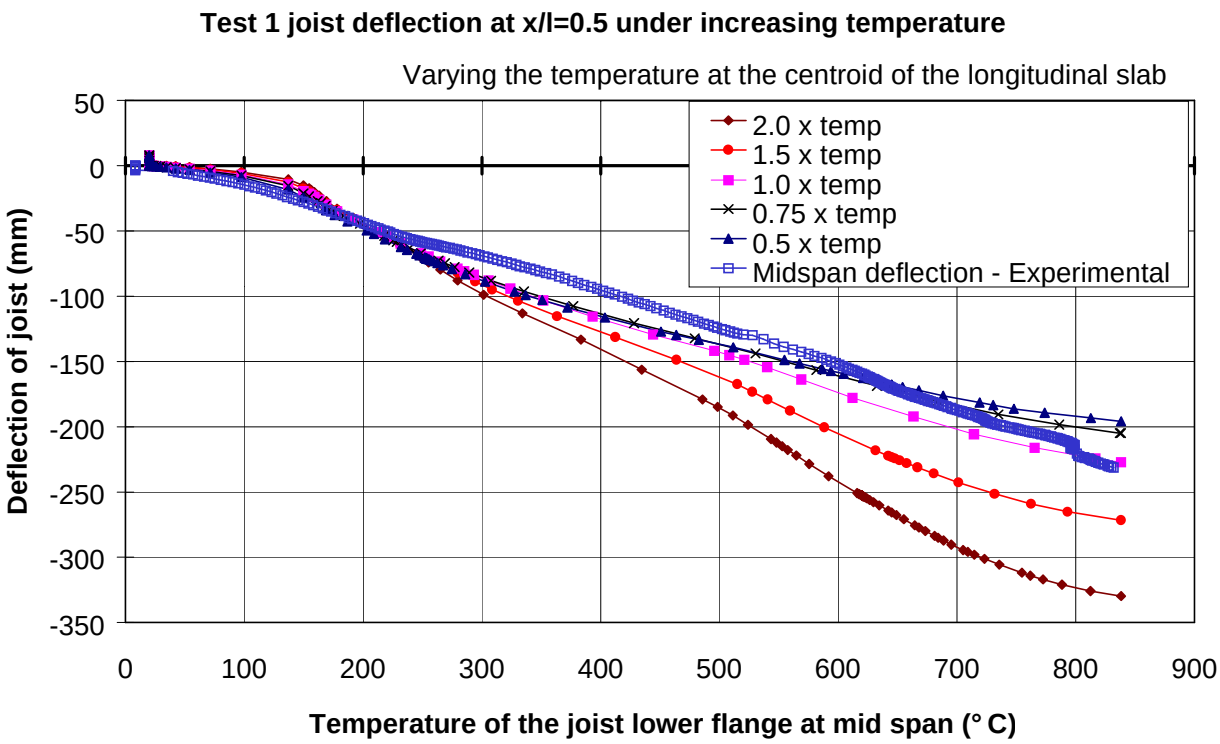
Effect of steel sections on the deflection
Figure 3.56

3.4.3 Effect of restraint conditions

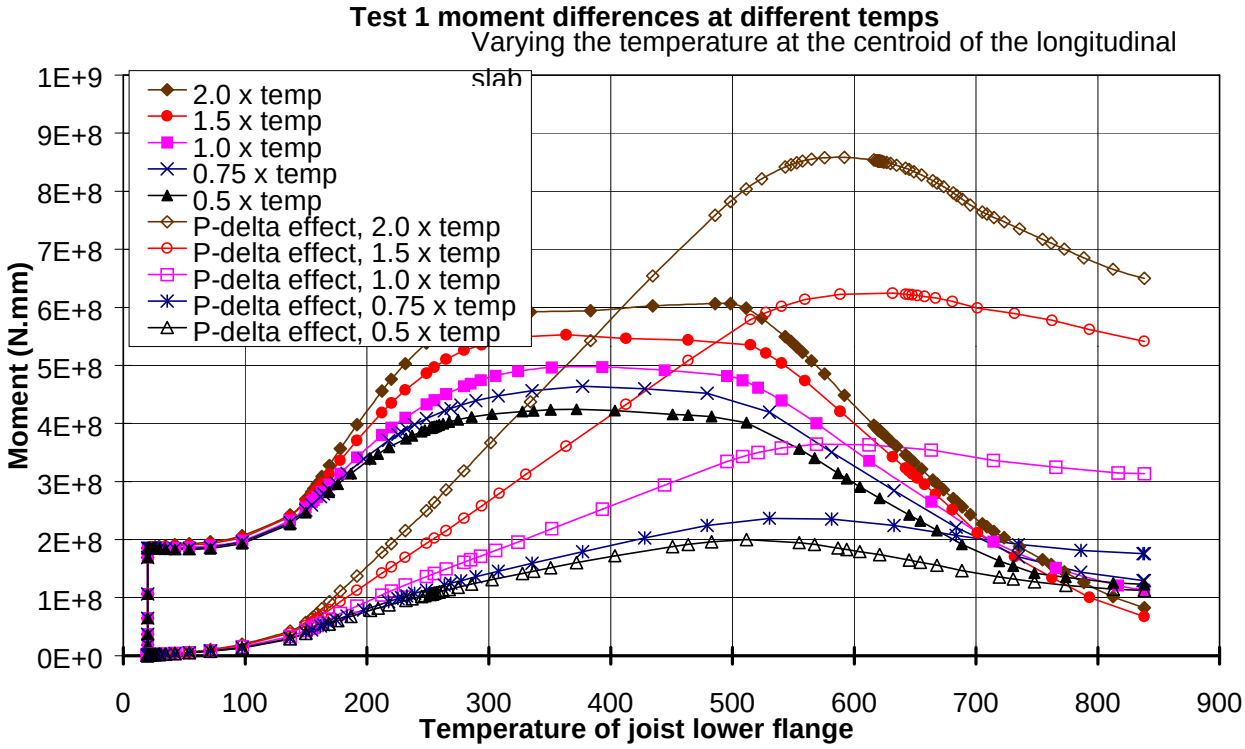
Restraint conditions can certainly have a major effect on the distribution of the internal forces and the displacements that occur as has been illustrated by the simple theoretical and computational analyses undertaken in reports TM1 and TM2. The degree of restraint available also changes during a fire as illustrated in describing the results from Test 1, where the rotational restraint available to the composite beam at the beginning is lost at around 200 °C due to the local buckling of the steel joist and tensile capacity of the slab being reached. It was clear from the results presented above that rotational restraints resulted in increasing hogging moments (until a "plastic hinge" was achieved) and lateral translation restraints produced compression forces if thermal expansion was dominant and tension forces if thermal bowing was dominant. At large deflections lateral restraints provided an anchor to the tensile membrane mechanisms. The source of this restraint is obvious for interior compartments - the colder and stiffer surrounding structure. For exterior compartments it is not so clear if sufficient restraints are still available. For rotational restraint, it may be argued that other than at an exterior column, little restraint is available. For lateral translational restraint, again it is clear that there can be considerable restraint at an exterior column.
As the amount of restraint required is not very high (for slender flexural members, such as beams and slabs) it is likely that sufficient lateral restraint is available at exterior boundaries through the actions of tension and compression *rings*. These phenomena are however difficult to obtain in grillage models and interrogation of shell models will be necessary to investigate them fully, a task which is still ongoing.

3.4.4 Effect of varying the mean temperature in Test 1

As has been mentioned many times before, the thermal actions determine the behaviour of a structure under normal loadings. **Figure 3.57, Figure 3.58** and **Figure 3.59** show the results of varying the mean temperature of the slab in the longitudinal direction (direction of the tested joist using the reference grillage model MD1). The deflections in **Figure 3.57** show a considerable increase with increasing mean temperatures as predicted from the understanding developed so far. **Figure 3.58** shows the effect of this variation on the moments (P-Δ and end-to-centre difference), both of which rise significantly with increasing mean temperature. **Figure 3.59** shows the axial force in the central rib in Test 1 (at a point just above the steel joist), it can be seen clearly that for larger mean temperatures the tensile forces in the rib develop much earlier and to a larger magnitude. The full result of these investigations (including the effect of varying thermal gradients) can be read in detail in report SM4 (see list of publications [5-6] in the appendix).

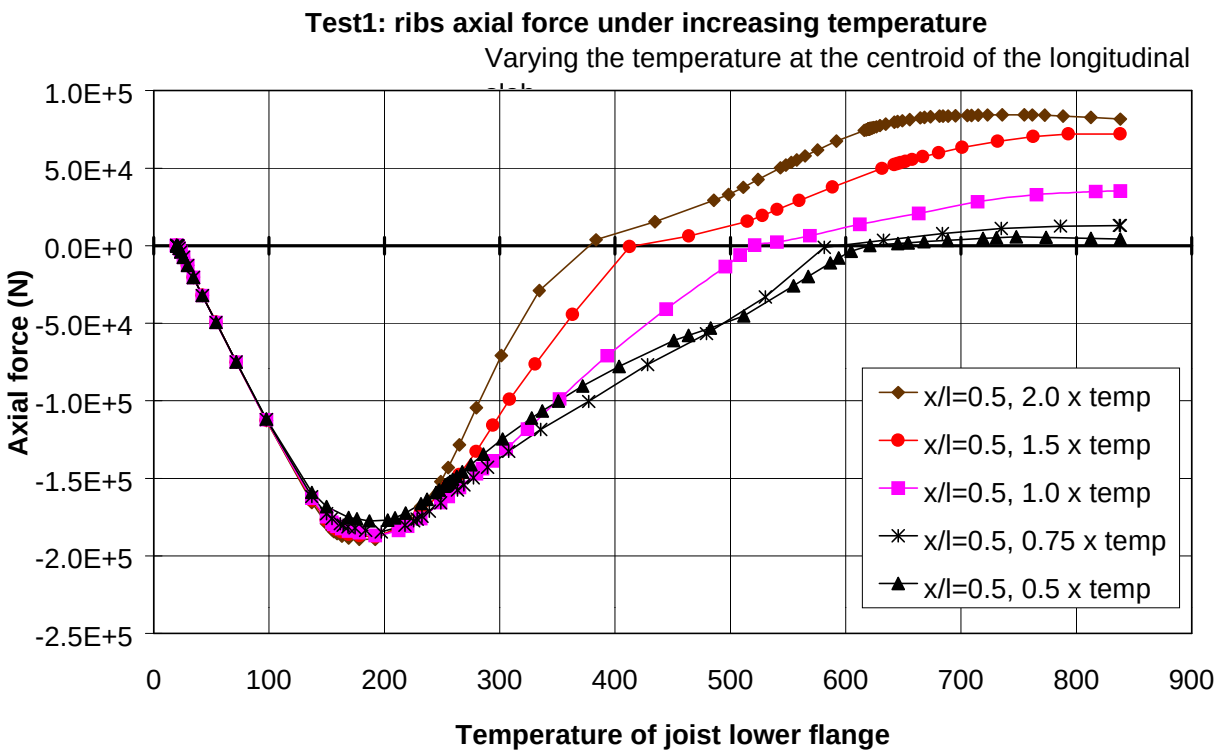


Effect of Temperature increase in the longitudinal direction on the Deflections
Figure 3.57



Effect of Temperature increase in the longitudinal direction on the P-Δ and Moment carried

Figure 3.58



Effect of Temperature increase in the longitudinal direction on the membrane forces

Figure 3.59

3.5 Discussion of modelling limitations and the likely impact of these limitations

Most of the results presented above were based on the observations made from grillage type models of the Cardington fire tests. Many of these observations have been confirmed by the results from other more accurate models using shell representations of the slab and also through simple analytical expressions developed from theory. Nevertheless, considerable simplifications do indeed occur when a slab is modelled using beam elements. Even in using more accurate shell models certain simplifying assumptions have been made to allow solutions to be developed in a stable and tractable manner. The following sections discuss some of these issues and their likely impact.

3.5.1 General assumptions

The temperature dependent stress-strain properties of concrete and steel were taken from EC2 [3] and EC3 [2] respectively. As discussed earlier the effect of steel stress-strain properties is very limited and the results are not sensitive to minor variations in these. The concrete properties were used to develop generalised stress-strain relationships (force-strain and moment-curvature) for both grillage and shell models developed by the Edinburgh group using ABAQUS. The resulting relationships in general were allowed to harden beyond a yield point. This clearly is an assumption and it was necessary to obtain a stable solution for a very difficult computational problem with both geometric and material non-linearities present to a high degree. The impact of this assumption is to allow very large displacements to occur without rupture. This however, was not the case in the real structure in the Cardington tests. If there were material failures they were limited to local areas with little observable impact on the global behaviour. The models using this assumption remain capable of modelling the development of large plastic rotations which are clearly necessary in determining the global displacements. Furthermore, it is clear from the analyses undertaken that the maximum compressive stresses in the slab are much lower than the capacity. In certain cases, such as the Test 3 grillage model, instances of large tensile stresses next to the columns did occur. The impact of this was the development of larger hogging moments and lower deflections, which can be seen as matters of detail and not a serious mis-representation of the global behaviour.

The property that would clearly make a considerable difference to the conclusions is the coefficient of thermal expansion. The values of thermal expansion of steel may quite confidently be assumed to be reliable. Those of concrete however are perhaps less so. These values change with the type of concrete, the type of aggregate used, the level of stress in the concrete etc [5]. An investigation of the effect of large variations in concrete thermal expansion is therefore desirable. This was indeed done by varying the mean temperatures in the concrete as discussed above (see report SM4) however there is scope for a much more detailed study.

3.5.2 Grillage models

In general the limitations of a grillage model include:

- Loss of in-plane shear transfer (unless a highly refined grillage is used, which defeats the object)
- Compatibility of displacements only enforced over joints (the members between joints can go into different displaced shapes)
- In-plane bending and torsion stiffnesses cannot be accurately modelled (usually large values are assigned)
- In the context of restrained thermal expansion, large forces and moments are generated at discrete locations instead of being dissipated over a larger area. The consequence of this is to reduce the effect of the restraint mechanisms compared to a continuum model

Two types of grillage models were used in this project. The particular limitations in the ABAQUS grillage models were:

- Axial-flexural interaction is not available with the generalised stress-strain beam elements used for concrete
- Only one line of beam element was used to model the slab over each secondary joist (all ribs were modelled)
- Elastic unloading was not available for the element used (available in ADAPTIC)
- Hardening was included in the temperature dependent generalised stress-strain relationship (impact discussed earlier)

Otherwise these models were fully materially and geometrically non-linear. The effect of axial flexural interaction is important in structures subjected to large moments and axial forces as clearly the full capacity is not possible. In the test results discussed above, large moments and axial forces are both present. The largest values however occur where the slab and steel beams are acting compositely. Therefore to develop large composite moments only requires the concrete to develop large axial forces. The moments in the concrete are found to be negligible. It can be concluded that this was not a serious limitation and this has indeed been proven to be the case when the results were compared with the shell models.

The flexural stiffness of the slab in the longitudinal directions (parallel to secondary steel joists) is an order of magnitude lower than the flexural stiffness of the ribs and the main action of the longitudinal slab is its composite behaviour with the secondary beam, therefore it can be argued that a single slab element in the longitudinal direction is adequate. The other grillage models (ADAPTIC) use more than one line of beam elements in the longitudinal direction but arrive at similar results.

Elastic unloading may indeed be occurring in local regions of the structure during the redistribution of stresses taking place when the slab moves into membrane mechanisms. This however should not be a serious deficiency as the redistribution occurs from the highest stressed locations and is essentially localised in character.

ADAPTIC grillage models include the correct unloading algorithm and arrive at the same pattern of behaviour. Unloading will need to be modelled much more accurately when cooling is modelled (which was not in the scope of this project).

3.5.3 Shell models

Two types of ABAQUS shell models were also used. Stress resultant shell models based on the user-defined subroutine FEAST were developed at Edinburgh. Elastic shell models using standard ABAQUS shell elements were used at British Steel, with the justification that the stresses in the slab were low other than in very local regions which were modelled specially using stiffness discontinuities. The only limitation of the FEAST based shell model is that it does not have elastic unloading.

4 IMPLICATIONS FOR STRUCTURAL DESIGN OF STEEL FRAMEWORKS IN FIRE

The results from both, the Cardington experiments and the computer modelling of those experiments described in the previous chapters, demonstrate that the composite steel framed building tested exhibited inherently stable behaviour under the applied fire scenarios due to the highly redundant nature of the structural system. The behaviour is characterised by several thermo-mechanical phenomena, which interact. This complex interaction is highly dependent upon the structural layout and the thermal regime in the fire compartment considered.

Several thermo-mechanical phenomena have been identified. These include:-

- restrained thermal expansion leading to thermal buckling in both beam and slab structural elements
- thermal bowing due to differential temperatures in the main structural elements and through depth thermal gradients leading to large hogging moments if the ends are continuous and tensions if the ends are pinned
- induced P-δ moments due to high axial forces (in highly restrained compartments) and large deflections due to large thermal straining
- material degradation leading to reducing forces in the steel members
- alternative load carrying mechanisms arise from the membrane stiffness of the concrete slab and contribute to load redistribution based upon the displacement constraints enforced by the requirements of compatibility

Of the identified phenomena, the dominant effect is that of restrained thermal expansion in the main structural elements. In the early stages of the fire, lateral restraint to translation (causing compression), coupled with rotational restraint to thermal bowing (causing hogging moments), leads to local buckling in the steel beams which limits the forces applied to the rest of the structure and to increasing deflections in a post-buckled state. In the later stages of the fire, the membrane stiffness of the concrete slab plays a key role by carrying both the floor load and the loads induced by P-δ moments. The slab is much better able to mobilise the tensile membrane mechanism because of the highly deflected shape without large mechanical strains and very often, 'beneficial' mechanical strains (compressive) induced by restrained thermal expansion of the slab. This can be likened to the temperature load inducing a prestressing force in the slab, which must be exhausted before excessive tensile mechanical strains can take place in the slab. This delays considerably the moment of rupturing of the slab tensile reinforcement, in practice, beyond the duration of normal fires in most cases.

The implications for designers are obvious. This structural behaviour is significantly different than that posed by present design techniques based upon the behaviour of single unrestrained structural elements in the standard fire test. This test is based upon statically determinate elements and the time to failure is based on a displacement criterion (often termed 'runaway' failure). In contrast, increasing deflections are beneficial in real structures in that thermal strains give rise to deflections rather than increasing forces and moments in the members, and at the points of restraint, which may lead to catastrophic events. The large deflections give rise to increased P-δ moments however, especially in highly restrained structures, which may be

potentially detrimental to structural performance. There are two factors which make such an outcome unlikely; one, that this is a gradual phenomena which increases and peaks over the duration of the fire (coinciding with loss of steel properties); second, is the competing effect of thermal gradients (leading to tensions if end rotations are permitted), which reduces the compressions developed quite significantly. In fact this can be seen as a beneficial mechanism, leading to a 'controlled' destruction of steel strength and therefore avoiding a potentially more destructive event, through inertia forces caused by a sudden release of strain energy.

In conclusion the identified behaviour indicates a number of competing effects in real structures which counter conventional understanding of structural behaviour in fire based upon traditional design methods. These effects will need to be taken into account in any design method, which allows designers to take advantage of the demonstrated inherent fire resistance of traditional composite steel construction. This really represents a paradigm shift in the understanding of the behaviour of composite structures in fire.

As the behaviour of composite structures is radically different from the present design philosophy, a new design philosophy is required, based upon a new definition of the fire limit state for this class of highly redundant structures. It is likely that the complex nature of the structural behaviour will move the onus for the fire design from the architect to the structural engineer. This is because structural performance will be dependent on the whole structural system and not the isolated behaviour of individual elements.

This additional complexity will be justified in that unprotected steel beams for composite steel structures are attainable provided that the robustness of the beneficial mechanisms and the quantification of the detrimental mechanisms can be ensured. This robustness can be ensured by a combination of good detailing practice to enable beneficial mechanisms to materialise and further work to identify worst case scenarios for structural layout and thermal regime.

5 RECOMMENDATIONS

The main thrust of this project has been on developing computational models of the structural response in the Cardington full-scale fire tests. The results have produced a *paradigm shift* in the understanding of the structural behaviour. As a result of this understanding it is possible to develop practical design guidance for steel frame composite structures in fire. The development of such a guide (which is acceptable to a wide range of professionals) requires a considerable amount of inter-disciplinary and inter-organisation collaborative effort to take place, preferably in the near future. It is recommended that worst case scenarios be identified by carrying out comprehensive parametric studies on composite structures under fire using some of the models developed in this project. This study should consider the effect of varying compartment sizes, structure geometry and fire scenarios including the effect of whole compartment fires (post flashover) and spreading fires.

In the absence of a more general practical guide, it is still possible to use the knowledge and understanding generated in this project to perform the fire design of large steel frame structures in a rational and scientific manner for a limited range of fire scenarios. To obtain a design based on the results of this project, the following route may be followed:

- Complete design for all other limit states
- Determine compartmentation and fire scenarios in the structure
    - As most studies in this report are limited to the British Steel Cardington tests, it is suggested that no more than a quarter of the whole floor be considered as a compartment (further work is needed to remove this limit).
    - Design fire scenarios may be determined using any accepted method of calculation of atmospheric temperature evolution in the compartment, with a good level of confidence that the results of the calculation lead to conservative estimates. Two different fire scenarios must be chosen:
        a) A hot and fast fire (high rate of heat release) leading to highest steel temperature
        b) A moderate and slow fire (low heat release rate) leading to highest mean slab temperature
- Use a recommended method of analysing the structure subjected to the two fires (see below) using the characteristic factors for strength and loading for fire limit state. Use hand calculations to check consistency of model results with expected response, in terms of maximum deflections, column displacements and internal forces and moments (see reports TM1, TM2 and TM4).
- If the analysis does not produce any of the following conditions, consider it safe (this only means structurally safe, and is distinct from any other limit states that may be part of the design brief, such as a limit on maximum deflections, which may already have been reached)
    - The maximum deflections in the fire exposed composite slab-beam system are within the range of deflections that would arise from the induced thermal strains and compatibility. An example of such a calculation appears in report TM4. One could also perform a number of analyses with the same fire scenario and different loads, to identify the point of transition when the structural response changes from a regime that is dominated by the thermal effects of expansion and bowing to another regime when the

imposed loads begin to dominate the response. This transition occurs just before structural collapse of the composite slab (runaway) is imminent (as illustrated in Figure 2 of report TM2).

- The tensile strains (mechanical strains, after excluding the thermal strains) in the concrete slab reinforcement, in the midspan regions, do not exceed the rupture strain. This needs to also be checked in the regions over supports, where in local areas the tensile reinforcement strains could be very high (close to rupture), however this may not precipitate a progressive 'unzipping' type failure because of the presence of compression ring type mechanisms to anchor the tensile membrane forces.
- Exterior column displacements do not exceed a reasonable value
- The deflections of beams at the compartment boundaries do not exceed a value that may cause partition failure and consequent breach of compartmentation.

Some important considerations in using these recommendations are discussed in the following sections.

5.1 Recommendations for modellers and analysts

Structural engineers undertaking the computational analysis of composite structures in fire should consider the recommendations presented below.

5.1.1 Which software?

The issue of what software to use in the analysis of a structure for fire is of paramount importance. The reasons for using ABAQUS in this research have been stated in Chapter 1 of this report in considerable detail. It is not necessary to use ABAQUS for analysing structures in fire, however it is necessary to ensure that all the phenomena and their interactions can be modelled by the software one chooses. Some important criteria are as follows:

- The software is capable of dealing with non-linear behaviour arising from changes in geometry and material properties during the analysis
- It is thoroughly verified using a very large number of different problems where the solution is known (benchmarking using analytical solutions, for instance using formulas presented in report TM2) to ensure that the algorithms are indeed producing the results that are consistent with the theory
- It is already in widespread use by the industry, researchers and academia with a well established user and support network (so all the major bugs have already been corrected)
- New versions are introduced regularly with additional capabilities (so that there is a formal mechanism in place to transfer the most recent developments in computational mechanics to the user)
- For analysing structural response in fire many of the simple examples presented in this report can be used to verify the capabilities of the program (see reports TM1, TM2, TM3 and TM4) and all the interactions between them checked in a relatively short span of time.
- Validation work comparable to the level of this report has been carried out

It is obvious that only the most prominent and best commercial software packages will fulfil the above criteria in the near future. There are however many special purpose non-linear structural mechanics codes developed at various institutions, mainly as tools of research and some of them have undergone considerable benchmarking and continuous development for them to used reliably for the purpose of analysing structures in fire. For instance, the ADAPTIC code (developed at Imperial College) which has also been used in this project, has consistently produced the same broad picture of the structural behaviour in Cardington tests as produced from ABAQUS models. There can even be certain advantages in using research codes such as ADAPTIC for modelling numerically difficult problems (such as brittle material behaviour at large strains, such as concrete – which it must be said is a rather open-ended problem with solutions tending to be limited to one kind of compromise or another), where the code may be improved and modified much more readily than commercial codes. However, in such situations one has to possess the source code and considerable familiarity with it, and also a high degree of skill in computational mechanics programming. This is clearly a very unlikely scenario, however a more practical alternative may be a close relationship with the authors who have all the skills if given enough time (and resources) to make the required changes. If this is not the case then this advantage vanishes rapidly. Finally, irrespective of the software used, the single most important consideration is the knowledge and experience of the analyst who should have spent considerable time understanding the behaviour of frame structures in response to thermal actions from first principles before undertaking the analysis of real structures for fire design. These skills are indispensable to ensure the reliability of the results from complex computational analyses.

5.1.2 What kind of model? What assumptions can be safely made?

A number of different types of models with different levels of complexity have been used in this project. For analysing such structures in the design office, relatively simple models can be used, to allow quicker and cheaper analyses to be carried out. All analyses need simplifying assumptions, which enables the computer model to be much simpler than the real structure. In choosing these assumptions the over-riding consideration must be that the model should not be simplified to the extent that one risks missing key phenomena. Some such simplifying assumptions suitable for a Cardington type structure are listed as follows:

- Beams and columns may be modelled using geometrically non-linear (GNL) beam elements
- Slabs may be modelled using GNL shell (not plate-bending) elements with coupled membrane and bending stiffness
- Slabs may also be modelled using a grillage of GNL beam elements
- Any reasonable constitutive model for high temperature material behaviour may be chosen for both steel an concrete (as the levels of yield and ultimate strength are not of primary importance)
- Elastic plastic behaviour (with small hardening) may be assumed without softening to enable stable non-linear convergence (a smooth relationship with fewer kinks will help convergence)

- Depth integrated or stress resultant beam and shell elements may be used as long as the formulation allows for material degradation effects
- The effect of localised stress concentrations causing concrete cracking or crushing can be assumed to be unimportant for overall global behaviour. Large plastic strains with a very low concrete strength in tension (of the order of 1%) is sufficient to model these effects to produce satisfactory and conservative global behaviour
- Connections between beam and column (partial depth end-pate) may be modelled using a pinned connection.
- The composite action between beams and slab may be modelled using *rigid links* or *multiple point constraints* at the locations of the shear connectors.
- It is sufficient to model a substructure extending one or half a bay into the unheated structure with appropriate boundary conditions (unless clear conditions exist that suggest to the contrary)
- Stiff points in the cold structure (braced bays, shear walls and cores etc.) may be modelled using a laterally restrained boundary condition

5.1.3 How to interpret the results?

Once results have been obtained from an analysis, the numbers must be checked against simple hand calculations (see for example report TM4) to ensure that the model results are of the right order and sign etc. and also produce a coherent description of behaviour consistent with that expected from the simple calculations. If this does not happen then both the hand calculations and analysis must be revisited until both are fully reconciled. Some important items that must be checked are:

- The maximum deflections should be quite closely approximated by the hand calculations for rectangular compartments (depending on the stiffness of the composite slab and beams, the average temperature and average through depth temperature gradient in the members, the restraint conditions and finally compatibility of the displacements)
- Column displacements (through a reasonable estimate of restraint stiffness)
- Forces and moments in the members (using estimates of restraint stiffness)

Once a satisfactory comparison has been achieved the computational results may be analysed in more detail to determine the forces and moments in all members to make sure that the design is satisfactory. If at this stage member dimensions need to be changed or members need to be fire protected another analysis with those changes should be done, until a satisfactory design is achieved.

5.2 Recommendations for designers and builders

Structural engineers undertaking fire design of steel structures may use some of the recommendations above. However, it is quite evident that it will be some time before the understanding developed can be converted into practical design guidance. There are however practical steps that designers and builders can take immediately (while still designing using traditional methods) which will ensure a more robust fire performance of a structure:

- Pay attention to detailing
- Ensure that sufficient lapping of steel mesh reinforcement is provided over the hogging regions of the slab
- Mild steel reinforcement meshes may be used (for larger strain capacity, especially in large compartments)
- Fire protect the edge beams
- Fire protect all columns
- Beam column connections that allow a certain amount of end translation will be beneficial both during heating and cooling.

REFERENCES

1. Structural Fire Engineering Investigation of Broadgate Phase 8 Fire. *The Steel Construction Institute*, 1991.
2. ENV 1993-1-2:1995: 'EuroCode 3 - Design of Steel Structures - Part 1-2: General Rules - Structural Fire Design
3. ENV 1992-1-2:1995: 'EuroCode 2 - Design of Concrete Structures - Part 1-2: General Rules - Structural Fire Design
4. D.J. O'Connor, B. McAllister, J. Munro, H.R. Bennett, 'Determination of the fire endurance of model concrete slabs using a plastic analysis methodology', The Structural Engineer, Volume 73, No. 19, October 1995.
5. G.A. Khoury, B.N. Grainger and P.J.E. Sullivan. Strain of Concrete during fire heating to 600 °C under load. *Magazine of Concrete Research* Vol. 37:195-215, 1985.

APPENDIX

1. List of supplementary reports
2. List of Publications

LIST OF SUPPLEMENTARY REPORTS

No.	Title of report	Org	Author	Remarks

ED (*Experimental Data*)

1	British Steel data on Cardington fire tests (Existing)	BS	BK	Public domain
2	BRE report on concrete slab test at ambient temperature	BRE	CB	Sent in March

AE (*Analysis of Experimental data to discover trends of behaviour*)

1	Analyses of BS Cardngton test data	BS	MOC	Pending

MD (*Models Developed with typical results*)

1	BS/TEST1 reference ABAQUS Model using beam general section	EU	AMS	Attached
2	BS/TEST1 ABAQUS Model using UGENS-I shell	EU	MG	Deleted
3	BS/TEST1 ABAQUS Model using UGENS-II (FEAST) shell model	EU	MG	Attached
4	BS/TEST1 ABAQUS Model using Shell elements for beam and beam general section for slab	BS	MOC	Pending
5	BS/TEST1 ABAQUS half-floor Model using elastic shell for slab	BS	MOC	Pending
6	BS/TEST1 ABAQUS half-floor Model using beam general section for slab	BS	MOC	Pending
7	BS/TEST2 ABAQUS Model using Shell elements for beams and beam general section for slab	BS	MOC	Pending
8	BS/TEST2 ABAQUS half-floor Model using elastic shell for slab	BS	MOC	Pending
9	BS/TEST2 ABAQUS Model using beam general section for slab	BS	MOC	Pending
10	BS/TEST3 reference ABAQUS Model using beam general section	EU	AMS	Attached
11	BS/TEST3 ABAQUS Model using UGENS-II shell	EU	MG	Pending
12	BS/TEST3 ABAQUS half-floor Model using elastic shell for slab	BS	MOC	Pending
13	BS/TEST3 ABAQUS half-floor Model using elastic shell with rotational discontinuities	BS	MOC	Pending
14	BS/TEST4 ABAQUS half-floor Model using elastic shell with rotational discontinuities	BS	MOC	Pending
15	Models of all BS Cardington tests using ADAPTIC	IC	AYE	Attached

AM (*Analysis of Model results to understand and discover trends of behaviour*)

1	Analysis of results from BS/TEST1 models, Grillage models	EU	AMS	Attached
2	Analysis of results from BS/TEST1 models, Half-floor models	BS	MOC	Pending
3	Analysis of results from BS/TEST1 models, UGENS shell models	EU	MG	Attached
4	Analysis of results from BS/TEST3 models, Grillage models	EU	AMS	Attached
5	Analysis of results from BS/TEST3 models, Half-floor models	BS	MOC	Pending
6	Analysis of results from BS/TEST3 models, UGENS shell models	EU	MG	Pending
7	Analysis of results from all ADAPTIC models	IC	AYE	Attached

No.	Title of report	Org	Author	Remarks

TM (*Theoretical Models developed*)

1	Behaviour of highly indeterminate composite frame structures in fire	EU	JMR,ASU	Attached
2	Fundamental principles of structural behaviour under thermal effects	EU	ASU,JMR	Attached
3	Study of thermal expansion and bowing in a restrained beam	EU	SL	Attached
4	Application of fundamental principles to Cardington restrained beam test	EU	ASU	Attached

SM (*Studies with Models verified to be satisfactory*)

1	Effect of increasing live loads on BS/TEST1	EU	AMS	Attached
2	Effect of changing steel section in the BS/TEST1 composite beam	EU	AMS	Attached
3	Effect of changing boundary restraints on BS/TEST1	EU	AMS	Attached
4	Effect of changing slab/beam temperature evolution on BS/TEST1	EU	SL,AMS	Attached
5	FEAST modelling of tensile membrane action	EU	MG	Attached

SS (*Generalised Stress and Strain relationships developed for FE models*)

1	Development of generalised stress strain relationships for the concrete slab in grillage models	EU	AMS	Attached
2	Development of generalised stress strain relationships for the concrete slab in shell models	EU	MG	Attached
3	Investigation of membrane-flexure interaction in the Cardington slab at elevated temperatures	EU	MG	Attached

HT (*Heat Transfer calculations for models*)

1	Heat transfer modelling for slab temperatures in Cardington tests	EU	SL,ASU	Attached

DD (*Development of Design guidance*)

1	Current status of design guidance	SCI	GN	Sent in March
2	Draft design guide	EU	JMR,ASU	Pending

LIST OF PUBLICATIONS

Papers published:

1. Sanad A.M., Rotter J.M., Usmani A.S. and O'connor M.A. : "Finite element modelling of fire tests on the Cardington composite building", Proceedings of Interflam'99, Interscience Communications Ltd. London, No.2-1999, 1045-1056.

2. Rotter J.M., Sanad A.M., Usmani A.S. and Gillie M. "Structural performance of redundant structures under local fires", Proceedings of Interflam'99, Interscience Communications Ltd. London, No.2-1999, 1069-1080.

3. Sanad A.M., Usmani A.S., Rotter J.M., Gillie M. and Drysdale D.D.:" Finite Element Model For The First Cardington Fire Test", Presented at the Sixth International IAFSS Symposium on Fire Safety Science, July 1999, Poitiers, France.

Papers accepted for publication:

4. Sanad A.M., Rotter J.M., Usmani A.S. and O'connor M.A.: "Composite beams in buildings under fire – numerical modelling and structural behaviour", Fire Safety Journal, accepted for publication June 2000.

5. Sanad A.M., Lamont S., Usmani A.S. and Rotter J.M.: "Structural behaviour in fire compartment under different heating regimes – part 1: slab thermal gradients", Fire Safety Journal, accepted for publication May 2000.

6. Sanad A.M., Lamont S., Usmani A.S. and Rotter J.M.: "Structural behaviour in fire compartment under different heating regimes – part 2: slab mean temperatures", Fire Safety Journal, accepted for publication May 2000

7. Gillie.M., Usmani A.S. and Rotter J.M.: "Modelling of heated composite floor slabs with reference to Cardington experiments", Fire Safety Journal, accepted for publication May 2000

Papers submitted as part of a planned special issue of the Fire Safety Journal

8. Lamont S., Usmani A.S. and Drysdale D.D.: "Heat transfer analysis of the composite slab in the Cardington Frame fire tests" (submitted to Fire Safety Journal, March 2000)

Papers to be submitted for special issue of the Fire Safety Journal

9. Usmani A.S., Rotter J.M., Lamont S, Sanad A.M. and Gillie M.: "Principles governing composite frame structure behaviour in fire". (*as Report TM2*)

10. Sanad A.M, Gillie M, Usmani A.S. and Rotter J.M.: "Modelling of Cardington Test1 - Details of model and description of behaviour"

11. Sanad A.M, Gillie M, Usmani A.S. and Rotter J.M.: “Modelling of Cardington Test3 - Details of model and description of behaviour”

12. Elgahazouli A.Y, Izzuddin B.A, and Richardson A.J: “Numerical Modelling of the Structural Fire Behaviour of Composite Buildings”

Papers presented at the First International Workshop on Structures in Fire (Copenhagen-June 2000)

13. Rotter J.M. and Usmani A.S.: “Fundamental principles of structural behaviour under thermal effects”. (*as Report TM2*)

14. Usmani A.S.: “Application of fundamental structural mechanics principles in assessing the Cardington restrained beam test”. (*as Report TM4*)

15. Gillie M. and Usmani A.S.: “An analysis of the behaviour of the first Cardington test using stress-resultant shell elements”.

Paper submitted to Journal of Constructional Steel Research (June 2000)

16. Gillie M. and Usmani A.S.: “A structural analysis of the first Cardington test”.