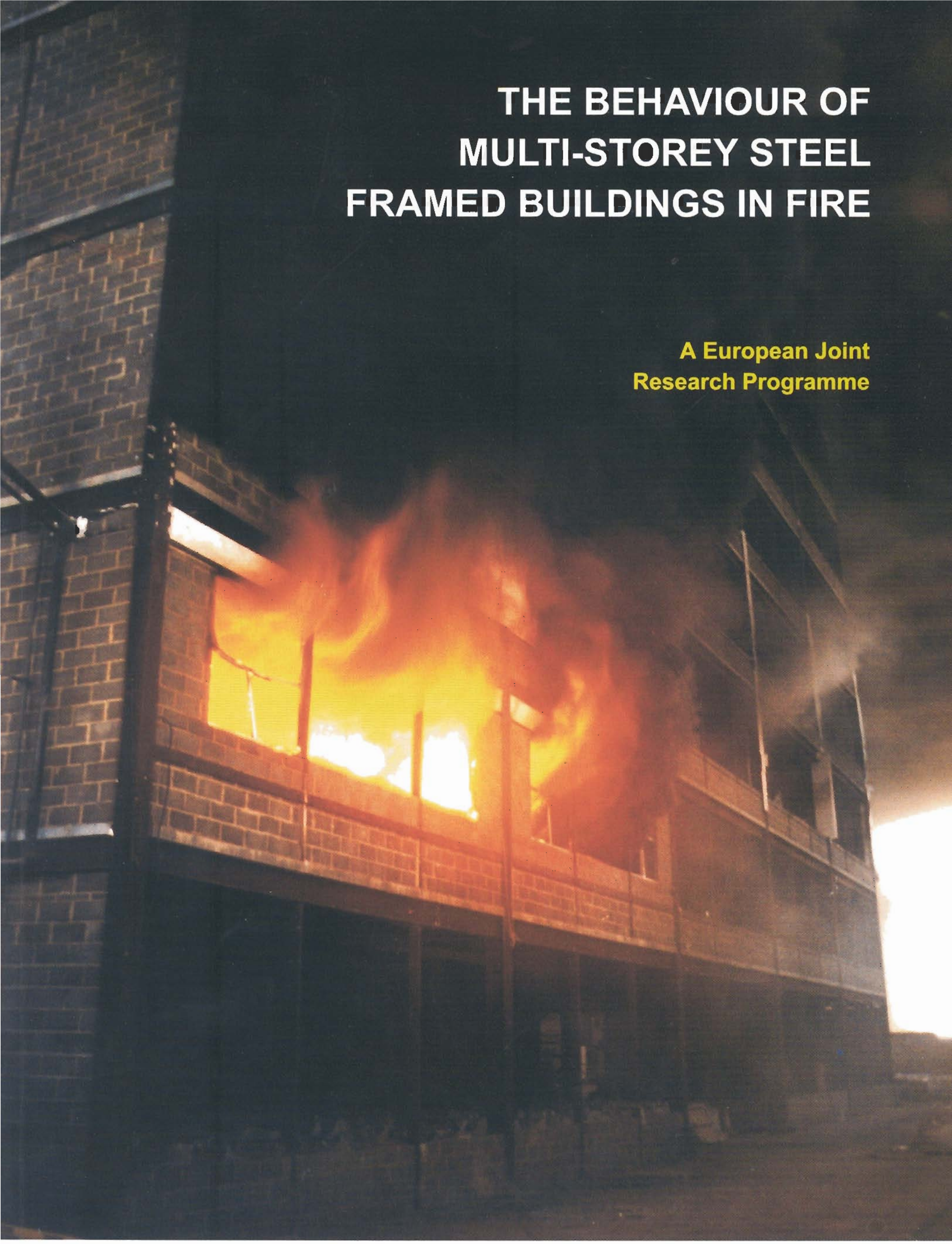
THE BEHAVIOUR OF MULTI-STOREY STEEL FRAMED BUILDINGS IN FIRE

A European Joint Research Programme



BRE

DETR
Environment
Transport
Regions

CTICM

TNO

British Steel

The Steel Construction Institute

THE BEHAVIOUR OF MULTI-STOREY STEEL FRAMED BUILDINGS IN FIRE

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Project Team Organisations

British Steel, Swinden Technology Centre (STC) which has been co-ordinating the European funded (ECSC) fire research programme, is one of the worlds leading organisations in applied steel research. It has over 350 professional research staff based at its site in Rotherham covering all aspects in the technological development of steel products and their use in practice. During the last 20 years, the Fire Engineering Section has been responsible for conducting much of the fundamental research carried out in the UK concerned with the behaviour of steel framed buildings in fire as well as the development of fire engineering principles, building design codes and standards. Facilities exist for advanced numerical modelling studies and these have been used extensively in understanding the performance of real structures in fire.

The **TNO Centre for Fire Research,** which is part of TNO Building and Construction Research, is primarily concerned with promoting rational fire prevention policies in buildings, installations and on board ships. It provides a comprehensive range of consultancy services for assessing the behaviour of materials, structures and building installations in fire. The Centre has a broad range of state of the art test facilities, including custom built furnaces for determining the fire resistance of building products some of which are unique in the Netherlands and large enough to accommodate complete wall and floor systems. Over many years it has developed a wide range of speciality software for running on its powerful computers which are used for predicting structural behaviour in fire, smoke movement in buildings, CFD analysis and human egress models.

Centre Technique Industriel de la Construction Metallique (CTICM) is the leading research organisation for steel construction in France. The Centre is one of the two main fire resistance test laboratories in France and has a wide range of experimental facilities for evaluating the performance of building products and materials to meet National and International requirements on fire safety performance. The Research and Fire Engineering Section has for more than 30 years, been involved in both testing and the development of analytical numerical techniques for predicting the behaviour of steel and composite structures under different fire scenarios and throughout this time has been engaged in numerous national and international research programmes.

The **Building Research Establishment** (BRE) has been responsible for leading and implementing the UK funded (Dept of the Environment, Transport and the Regions - DETR) fire tests on the 8-storey frame. It is one of the worlds leading organisations having 350 staff based at three sites for research and advice into; construction quality and production, environmental impact of construction, building performance - structures, materials and systems, the prevention and control of fire. Its Cardington site in Shortstown near Bedford, offers a unique facility for large scale testing and houses the multi-storey frame on which both the European and UK funded research programmes have been conducted.

The Steel Construction Institute (SCI) is an independent, member-based organisation with the aim of developing and promoting the effective use of steel in construction. Member companies include architects, engineers, steelwork contractors, suppliers academics, local authorities and government departments within the UK, elsewhere in Europe and overseas.The SCI carries out research into most aspects of structural steel design, including the effects of fire on steelwork, offshore structures and the environmental aspects of using steel. Dissemination through design guides and educational courses are a major activity.

The Academy of Steel Construction at **The University of Sheffield** is an established centre for undergraduate, postgraduate teaching and research in a wide range of areas related to the use of steel in buildings. One of its principal activities has been the development of computer modelling techniques to simulate the response of steel structures in fire. This has developed over a period of 15 years from simple, isolated elements to complete, composite steel frames.

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FOREWORD

During the last decade a considerable number of fire resistance tests have been carried out in many European countries. Most of these tests have, however, been limited to evaluating the performances of individual structural members in compartments with carefully controlled gas fire atmospheres. Tests of this type are useful for establishing the behaviour of steel members in a standard test but they cannot represent natural fire scenarios since they do not allow for the restraining effect of other structural elements present in a building nor do they adequately represent the thermal environment of a natural fire.

In view of the above British Steel, Swinden Technology Centre, The Building Research Establishment, TNO Building & Construction Research, Centre Technique de la Construction Metallique (CTICM), The Steel Construction Institute and The University of Sheffield formed a research team to investigate the true behaviour of steel framed buildings when subjected to fire attack. The BRE Cardington facility provided a unique opportunity to carry out the necessary large scale fire tests on a real multi-storey steel framed building which had been designed and constructed under normal commercial conditions.

The project work included six large scale fire tests and extensive numerical modelling. The work was funded primarily by British Steel plc, The European Coal and Steel Community, The Building Research Establishment, The Department of Environment Transport and Regions, TNO Building & Construction Research and Centre Technique Industriel de la Construction Metallique (CTICM).

This document places the project in context (by reference to previous relevant work) and summarises the main results achieved to date, particularly those relating to the major fire tests, in a way that will be of immediate benefit to the fire engineering community and to the construction industry throughout the European Community. It is anticipated that the detailed results and subsequent analyses will form the basis of a new more rational design methodology for steel framed buildings under fire attack.

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SUMMARY

British Steel, Swinden Technology Centre, in collaboration with the Building Research Establishment recently led a large European research initiative to study the behaviour of a steel framed multi-storey building subjected to fire attack. The overall objective was to gain a greater understanding of the natural fire resistance of such structures, to correlate existing predictive numerical models and to establish the basis for a new more rational design methodology for steel framed buildings subject to fire attack.

This publication introduces the research project as a whole and, in particular, summarises the results of six major fire tests carried out within the eight storey steel framed structure located within the BRE Large Building Test Facility at Cardington, Bedfordshire.

It was found that this composite steel framed building possessed a very significant degree of inherent fire resistance even although the steel floor beams remained entirely unprotected against fire attack.

The detailed results of the fire test programme represent a very significant contribution to the development of structural fire engineering and will lead, together with the associated numerical analyses, to a more logical approach to the design of steel framed buildings in fire.

1 INTRODUCTION

In this publication the recent research carried out at the BRE Cardington Test Facility at Bedford, UK, is described. The research work was sponsored by British Steel, TNO and CTICM, jointly with the European Coal and Steel Community (ECSC) and the UK government through the Department of the Environment, Transport and the Regions (DETR). The purpose of the research was to investigate the behaviour of a modern composite steel framed building in fire. Although the aim of the project was to focus on the structural behaviour in fire, valuable data has been obtained on fire development.

This publication is in three parts. Firstly, a summary of the observed behaviour of steel framed buildings in fire is presented. This includes the behaviour of real structures in accidental fires and test structures with experimental fires. Secondly, the major fire tests at Cardington are described and finally there is a discussion of the observed structural behaviour and numerical modelling.

Using the major fire tests as a basis, both numerical studies and further testing have already commenced with the aim of developing design guidance for steel framed buildings in fire. In addition, a large risk and hazard assessment study is currently underway, which includes participants from every country of the European Community. Figure 1.1 illustrates the various individual projects and their relationship within the overall fire research investigation. The ultimate aim is to be able to design steel framed buildings with a known inherent degree of fire resistance. In cases where the inherent fire resistance is insufficient to meet the fire risk, additional fire precautions will need to be applied effectively and economically. The use of active measures such as sprinklers is strongly advocated in reducing the thermal loading upon the structure, the threat to life safety, and spread of the fire to other parts of the building.

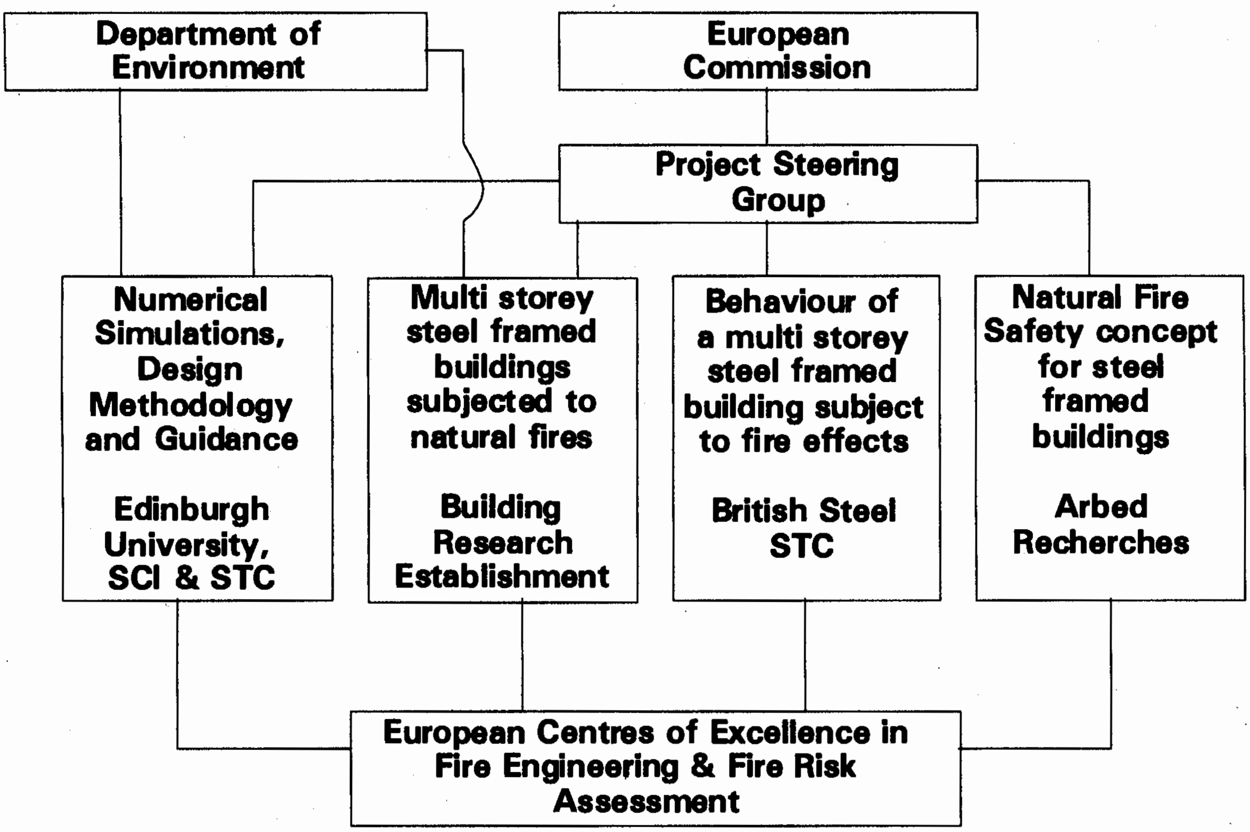


Figure 1.1 *Related research projects investigating the fire resistance of steel framed buildings*

1.1 British Steel plc ECSC Programme

The British Steel ECSC programme was designed to be complimentary to that being undertaken by the BRE and indeed both projects were closely coordinated within the overall scheme outlined in Figure 1.1.

Significant changes have taken place over the last decade regarding the methods of construction for steel and composite structures. The development of new composite floor systems and advances in fabrication technology have resulted in extremely competitive steel framed buildings. Over the same period the development and application of fire engineering techniques has brought about a significant reduction in the cost of fire protecting steel frames.

There is, however, a growing opinion that the structural contribution of modern composite floor systems is under-utilised when designing for the fire limit state. In this research project we sought to demonstrate the true inherent fire resistance of a modern composite steel framed building under severe fire scenarios and to gain sufficient information to correlate existing predictive numerical models.

The British Steel fire tests increased in complexity from a gas fired furnace type test on a single floor beam to a large natural fire test within a realistic office environment. A number of test locations were utilised within the structure in order to vary the degree of structural restraint and load levels involved.

Fundamental work is being carried out to agree, on a European basis, the correct procedures to follow when modelling steel framed buildings under fire conditions. Numerical models are to be developed on a number of different platforms in order to assess existing capability to replicate the structural behaviour observed and recorded during the fire tests.

The ultimate goal is to be able to assess with confidence the inherent natural fire resistance of steel framed buildings and to formulate a more rational design methodology for such structures.

Dr David Martin
British Steel plc

1.2 BRE Programme

The objectives of the BRE fire test programme were to examine the behaviour of multi-storey steel framed buildings subject to real fires and to use the data from the tests to validate computer models for structural analysis at elevated temperatures. It is anticipated that the work will provide substantial benefits and produce high quality data. This will inform decisions on the degree of fire protection required for steel framed buildings with composite floors, which could significantly reduce costs whilst maintaining existing levels of safety.

Evidence from analytical models and from investigations following real fires have suggested that the fire resistance of complete structures is better than that of the single elements from which fire resistance is universally assessed. In order to verify these observations it was necessary to carry out full scale tests to improve the design procedures for modern steel framed buildings and to quantify safety margins.

The philosophy of the BRE compartment fire tests was to assess the behaviour of a real building designed and constructed to normal commercial requirements subject to real fires. Traditional laboratory tests have attempted to isolate a single, or, at best, a small number of parameters for consideration by introducing idealised conditions. With these fire tests the intention has been, as far as possible, to avoid such idealised conditions and to consider realistic scenarios both in terms of loading and compartment design and layout.

Dr David Moore
Building Research Establishment

Figure 1.2 *General view of test structure illustrating the steel frame and steel decking prior to placing concrete and constructing walls.*

2 OBSERVED BEHAVIOUR IN REAL BUILDING FIRES

In recent years two building fires in England have provided the opportunity to observe how modern steel framed buildings can perform in fire. The experience from these fires was influential in stimulating thought about how actual buildings can be designed to resist fire and in bringing about the Cardington experiments.

2.1 Broadgate Phase 8 Fire, London

In 1990 a fire developed in a partly completed 14 storey office block on the Broadgate development in London[1]. The fire began inside a large site hut on the first level of the building. Fire temperatures were estimated to be over 1000°C. Following the fire structural elements covering an area of approximately 40 x 20m were replaced, but importantly no structural failure occurred and the integrity of the floor slab was maintained during the fire. The direct fire loss was in excess of £25M, of which less than £2M was attributed to the structural frame and floor damage, the other costs resulted from smoke damage.

The floor was constructed using composite long span lattice trusses and composite beams supporting a composite floor slab. The floor slab was designed to have 90 minutes fire resistance. At the time of the fire the building was under construction and the passive fire protection to the steelwork was incomplete. The sprinkler system and other active measures were not yet operational.

After the fire, a metallurgical investigation concluded that the temperature of the steelwork did not exceed 600°C. A similar investigation on the bolts used in the steel-to-steel connections was also conducted. This concluded that the maximum temperature reached, either during manufacture or as a consequence of the fire, was 540°C.

Following the fire the steel beams had a maximum measured permanent deflection of 270mm and a minimum of 82mm. Beams which had higher permanent displacements showed evidence of local buckling of the bottom flange and web near to their supports. From this, it was concluded that the behaviour of the beams was strongly influenced by restraint to thermal expansion. This restraint was provided by the surrounding structure which was at a substantially lower temperature than that affected by the fire.

The fabricated steel trusses supporting the floor slab spanned 13.5m and had a maximum permanent vertical displacement of 552mm, with some components showing buckling deformation. It was concluded that the restraint to thermal expansion provided by other members of the truss, together with differential heating of members, caused additional compressive axial forces which resulted in buckling.

The steel columns consisted of standard rolled sections and in cases where these were unprotected the column had deformed and shortened by approximately 100mm. These columns were adjacent to much heavier columns which showed no significant signs of permanent deformation. It was thought that this shortening was caused by restrained thermal expansion. This restraint was provided by a rigid transfer beam at an upper level of the building, together with the columns outside the fire affected area.

Although some of the columns deformed, the structure showed no signs of possible collapse. It was thought that the less effected parts of the structure were able to carry additional loads which were redistributed away from the weakened areas.

The floor slab consisted of a standard profiled metal deck, concrete and mesh composite construction. Following the fire the floor suffered gross deformations with a maximum permanent vertical displacement of 600mm. Some failure of the reinforcement was observed. In some areas the steel profiled deck had debonded from the concrete. This was considered to be caused mainly by steam release from the concrete together with the effects of thermal restraint and differential expansion.

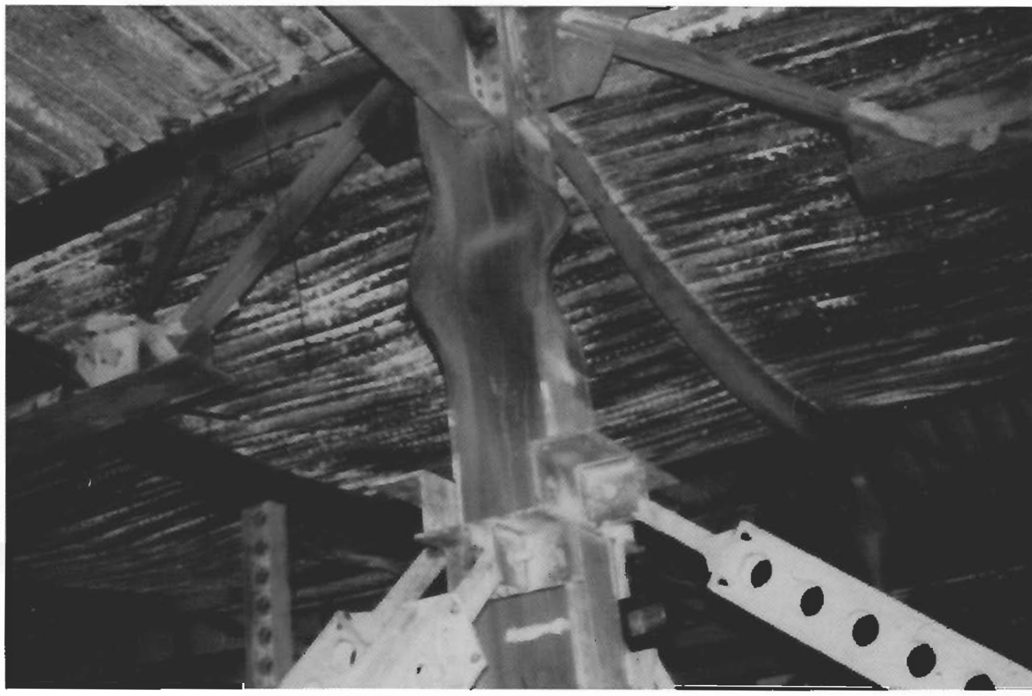


Figure 2.1 *A buckled column at Broadgate following an accidental fire*

Figure 2.2 *A buckled beam and floor slab at Broadgate following the fire*

A mixture of cleat and end-plate type of connections were used. Following the fire no connections were observed to have failed although deformation was evident. In the cleated connections there was some deformation of bolt holes. In one end-plate connection two of the bolts had fractured and in another the plate had fractured down one side of the beam, but the connection was still able to transfer shear. The main cause of deformation was thought to be due to the tensile forces induced during cooling.

2.2 Churchill Plaza Building, Basingstoke

In 1991 a fire took place in the Mercantile Credit Insurance Building, Churchill Plaza, Basingstoke. The building, constructed in 1988, was twelve storeys high. The columns had passive fire protection in the form of boards and the composite floor beams had spray applied protection. The underside of the composite floor was not fire protected and the structure was designed to have 90 minutes fire resistance.

The fire started on the eighth floor and spread rapidly to the tenth floor as the glazing failed. The fire protection materials performed well and there was no permanent deformation of the steel frame. The fire was believed to be comparatively "cool" due to the failure of the glazing permitting a cross wind to increase the ventilation. The protected connections showed no deformation.

In places the dovetail steel deck had indicated signs of debonding from the concrete floor slab. Similar to the Broadgate fire, it was thought that this was due to steam from the heated concrete forcing the steel deck to break its bond from the concrete. A load test was conducted on the most badly affected area which demonstrated that the slab had adequate load carrying capacity and could be reused without repair. The estimated cost of repair to the building was over £5M, with most of the damage being due to smoke. The fire protection was replaced although visually it appeared undamaged. No structural repair was required to the protected steelwork.

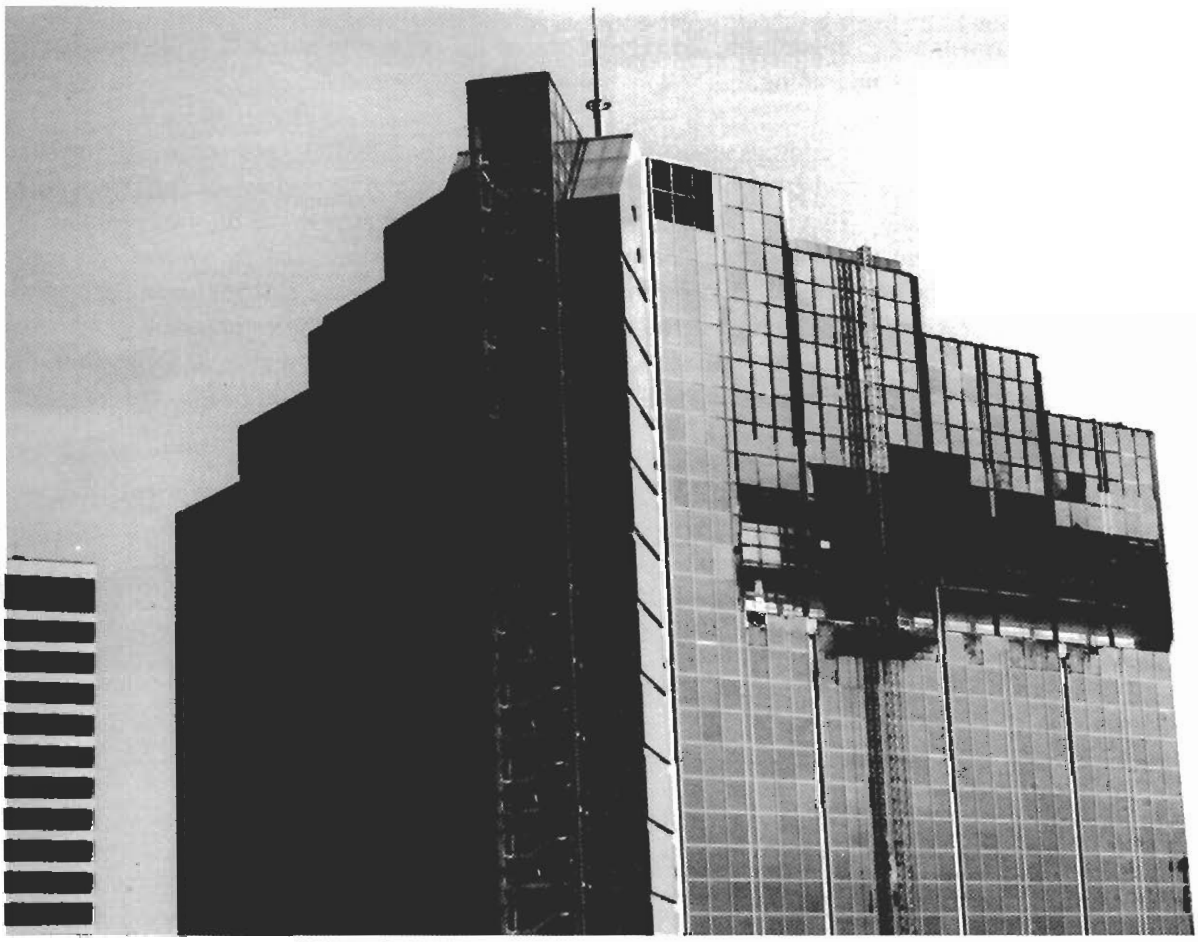


Figure 2.3 *Churchill Plaza, Basingstoke following the fire*

3 SUMMARY OF SIMILAR RESEARCH

Large scale tests of the type conducted at Cardington are rare. However, some notable tests have been carried out and these are now reviewed.

3.1 BHP William Street Fire Tests

BHP are Australia's biggest steel maker and have been actively researching fire engineered solutions for steel framed buildings for many years. They have carried out a number of tests. The two main test programmes were designed to demonstrate the performance of specific buildings and these are now discussed.

The 41 storey building at 140 William Street in the centre of Melbourne was the tallest in Australia when it was built in 1971. The beams and the soffit of the composite steel deck floors had been protected with asbestos based material and a decision was made to remove these hazardous coatings during a refurbishment programme in 1990. The building was square on plan, with a central square inner core. The steelwork around the inner core and external steel columns were protected by concrete, which remained unchanged after refurbishment.

The floor structure had been designed largely to serviceability rather than strength requirements. This meant that there was a reserve of strength which would be very beneficial to the survival of the frame in fire, as higher temperatures could be sustained before the frame reached its limiting strength.

At the time of the refurbishment the required fire resistance was 120 minutes. Normally this would entail the application of fire protection to the steel beams and also to the soffit of the very lightly reinforced slab (present Australian Regulations have been revised which allow the soffit of the slab to be left unprotected for 120 minutes fire resistance). In addition, the regulations, at the time, also stated that the light hazard sprinkler system would require upgrading.

During 1990, the fire resistance of buildings was subject to national debate and the opportunity was taken to conduct a risk assessment to assess whether fire protecting the steelwork and upgrading the sprinkler system was necessary for this building. Two assessments were made. The first was made on the basis that the building conformed to current regulations with no additional safety measures and the second was made assuming no protection to the beams and soffit of the slab, together with using the existing sprinkler system. Also, in the second assessment the effect of detection systems and building management systems were included. If the results from the second risk assessment were at least as favourable as those from the previous assessment then the use of the existing sprinkler system and unprotected steel beams and composite slabs were considered acceptable.

A series of four fire tests were then carried out to obtain data for the second risk assessment. This included matters such as the likely nature of the fire, the performance of the existing sprinkler system, the behaviour of the unprotected composite slab and castellated beams subjected to real fires, and the likely generation of smoke and toxic products.

The tests were conducted on a purpose built test building at BHP Research Melbourne Laboratories. This simulated a typical 12m x 12m corner bay of the actual building. The test building was furnished to resemble an office environment with a 4m x 4m small office constructed adjacent to the perimeter of the building. This office was enclosed by plasterboard, windows, a door, and the facade of the test building. Imposed loading was applied by water tanks.

A total of four fire tests were conducted. The first two were concerned with testing the performance of the existing light hazard sprinkler system. In Test 1 a fire was started in the small office and the sprinklers were automatically activated. This office had a fire load of 52kg/m^2. The atmosphere temperatures reached 60°C before the sprinklers controlled and extinguished the fire. In Test 2, a fire was started in the open-plan area midway between four sprinklers. This area had a fire load of 53.5kg/m^2. The atmosphere temperature reached 118°C before the sprinklers controlled and extinguished the fire. These two tests showed that the existing light hazard sprinkler system was adequate.

The composite slab was assessed in Test 3. The supporting beams were partially protected. The fire was started in the open plan area and allowed to develop with the sprinklers switched off. The maximum atmosphere temperature reached 1254°C. The fire was extinguished once it was considered that the atmosphere temperatures were past their peak. The slab supported the imposed load. The maximum temperature recorded on the top surface of the floor slab was 72°C. The underside of the slab had been partially protected by the ceiling system which remained substantially in place during the fire.

In Test 4 the steel beams were left unprotected and the fire was started in the small office. The fire did not spread to the open-plan area despite manual breaking of windows to increase the ventilation. Fires were therefore then ignited from an external source in the open-plan area. The maximum recorded atmosphere temperature was 1228°C with a maximum steel temperature of 632°C. The fire was extinguished when it was considered the atmosphere temperatures were past their peak. However, since steel temperatures typically lag behind the atmosphere temperatures it is not conclusive that the maximum steel temperatures were reached during this test. Again the steel beams and floor were partially shielded by the ceiling. The central displacement of the castellated beam was 120mm and most of this deflection was recovered after the test.

It was concluded from the four fire tests that the existing light hazard sprinkler system was adequate, and no fire protection was required to the steel beams or soffit of the composite slab. Therefore, any fire in the William Street building should not excessively deform the slab or steel beams provided the steel temperatures do not exceed those recorded in the tests. This relies on protection provided by the suspended ceiling system, which remained largely intact during the tests.

3.2 BHP Collins Street Fire Tests

The purpose of the test was to collect data on fire resulting from combustion of furniture in a typical office compartment. The compartment was 8.4m x 3.6m and filled with typical office furniture, which gave a fire load between 44 and 49 kg/m^2. A non-fire-rated suspended ceiling system was installed, with tiles consisting of plaster with a fibreglass backing blanket. An unloaded concrete slab formed the top of the compartment. During the test, temperatures were recorded in the steel beams between the concrete slab and the suspended ceiling. The temperatures of three internal free-standing columns were also recorded.

Two of these columns were protected with aluminum foil and steel sheeting and the other remained unprotected. Three unloaded external columns were also constructed and placed 300mm from the windows around the perimeter of the compartment.

Following the test it was shown that the non fire rated ceiling system provided an effective fire barrier causing the temperature of the steel beams to remain low. During the test the majority of the suspended ceiling remained in place. Gas temperatures below the ceiling ranged from 831 to 1163°C, with the lower value occurring near to the broken windows. Above the ceiling the air temperatures ranged from 344 to 724°C, with higher temperatures occurring where the ceiling was breached. The maximum steel beam temperature was 430°C.

The internal columns reached a peak temperature of 730°C for the unprotected case and below 400°C for the protected cases. The external columns recorded a peak temperature of 480°C, with the flange nearest to the fire compartment being hotter.

This fire test showed that the temperatures of the beams and external columns were sufficiently low to justify the use of unprotected steel and, as in the William Street tests, the protection afforded by a non-fire-rating suspended ceiling was beneficial.

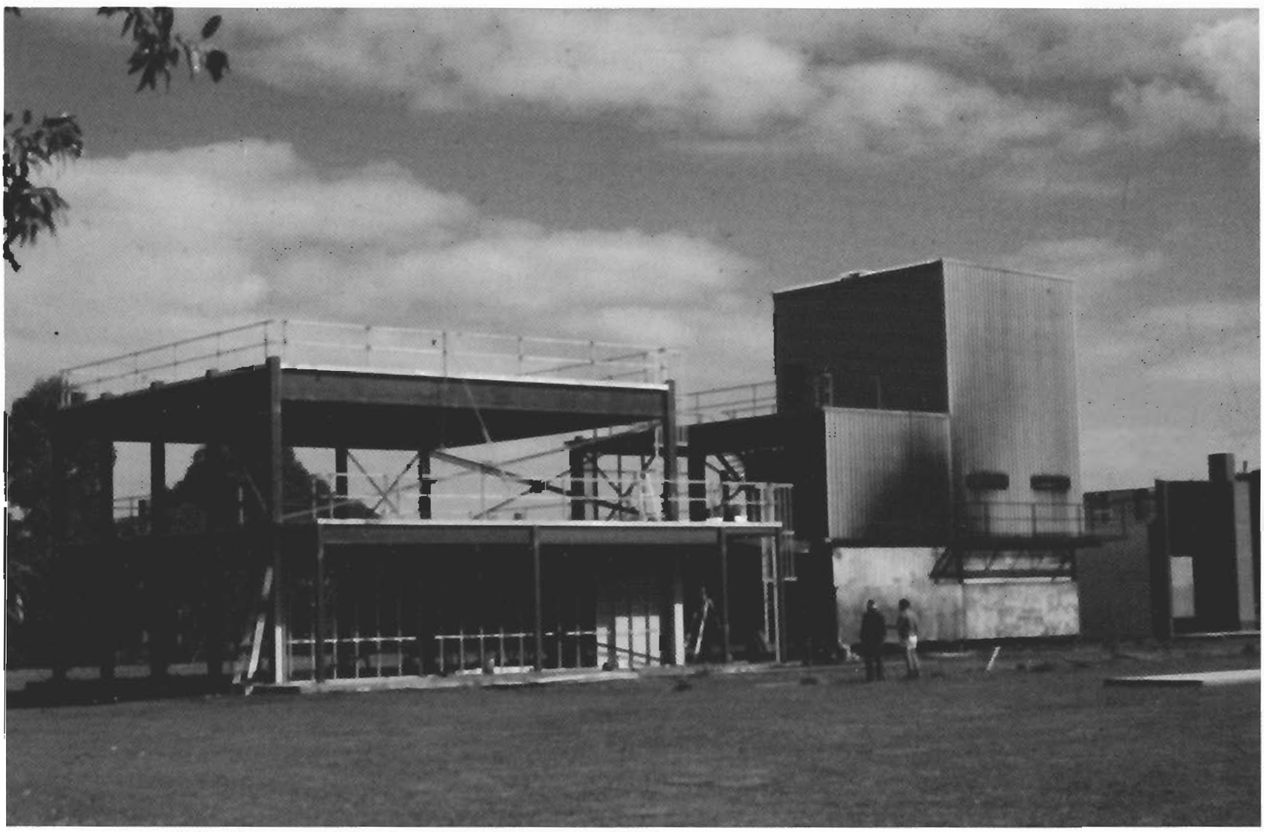


Figure 3.1 *The BHP test building*

Figure 3.2 *BHP fire test*

3.3 Fire Test at the Stuttgart-Vaihingen University Germany

In 1985 a fire test was conducted on a four storey steel-framed demonstration building constructed at the Stuttgart-Vaihingen University in Germany[2]. Following the fire test the building was used as an office and laboratory.

The building was constructed using many different forms of steel and concrete composite elements. These included water filled columns, partially encased columns, concrete filled columns, composite beams and various types of composite floor.

The main fire test was conducted on the third floor, in a compartment covering approximately one third of the building. Wooden cribs provided the fire load and oil drums filled with water provided the gravity load. During the test, the atmosphere temperature exceeded 1000°C, with the floor beams reaching temperatures up to 650°C. Following the test, investigation of the beams showed that the concrete infilled webs had spalled in some areas exposing the reinforcement. However, the beams behaved extremely well during the test with no significant permanent deformations following the fire. The external columns and those around the central core showed no signs of permanent deformation. The composite floor reached a maximum displacement of 60mm during the fire and retained its overall integrity.

Following the fire, refurbishment work involved the complete replacement of the fire damaged external wall panels. The damaged portions of steel decking to the concrete floor slab and the concrete infill to the beams were also replaced. The floor was strengthened by adding reinforcement to the soffit of the slab and spraying with concrete. Overall it was shown that refurbishment to the structure was possible at a low cost and using minimal labour.

4 CONCLUSIONS AND COMMENTS FROM PREVIOUS WORK

The design of the Cardington tests was most influenced by the Broadgate fire. Following this fire it could be seen that some of the elements had lost their load carrying capacity although there were no signs of collapse. It was clear that the composite floor and supporting steelwork had a major influence on the overall stability of the structure, acting as a diaphragm or membrane distributing loads from weakening members.

The Australian tests were interesting illustrations of what was likely to happen in a real building, but their reliance on the non-fire-rating suspended ceilings could be problematic in practice.

The Basingstoke fire showed how well a fully protected building might perform and could be seen as a vindication of our present regulations. However, it gave no indication of how safe the traditional approach with fully protected steelwork actually is (the level of safety was unknown), and therefore no information on possible over specification.

The test in Germany was very successful and showed that many of the composite forms of construction which are now incorporated into EC4 Part 1.2[3] will perform well in real fires. Experience in UK, where a large proportion of multi-storey buildings are constructed in steel, indicates ,however, that many of the systems tested would be uneconomic. This is mainly due to the low cost of fire protection in the UK compared with many other countries within Europe. Therefore, the German test, although an excellent demonstration of the performance of composite construction in fire, was not influential in the design of the Cardington major fire tests.

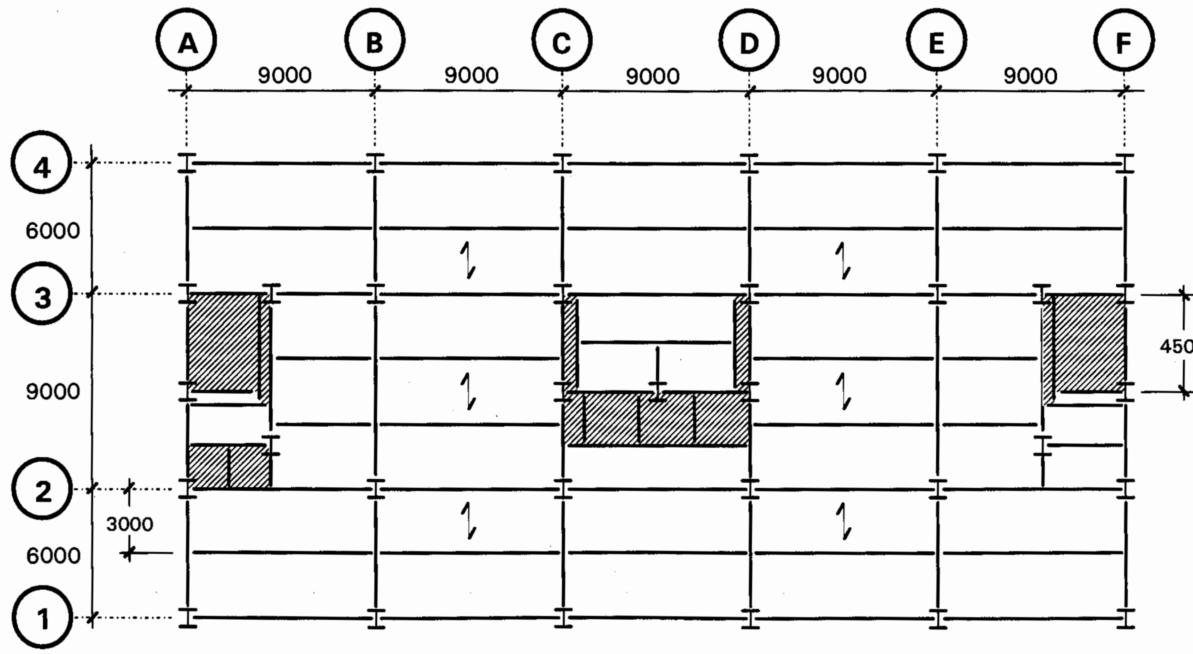
5 THE MULTI-STOREY TEST BUILDING AT CARDINGTON

The eight-storey steel-framed test building was constructed by the Building Research Establishment at its Cardington Laboratory near Bedford. The building was designed and constructed to resemble a typical modern city centre office development (Figure 5.1). On plan, the building covered an area of 21m x 45m with an overall height of 33m. There were 5 equally spaced bays along the length of the building. Across the width there were 3 bays spaced 6m, 9m and 6m. Placed centrally was a 9m x 2.5m lift core with two 4m x 4.5m stairwells placed at either end. A central reception area required the removal of two columns in the first two storeys. The columns above this level were supported by transfer beams. A plan of the building is shown in Figure 5.2.

Figure 5.1 *General view of the structural frame*

The structural design was carried out to the British Standard, BS5950[4], by Consulting Engineers, Peter Brett Associates. The design also complied with the Eurocodes, EC3 and EC4. The fabrication and erection was carried out by Caunton Engineering Limited and was completed in March 1993. The steel members were shot-blasted, but left unpainted. Rolling of the members took place at three different British Steel mills, Lackenby, Shelton and Scunthorpe. Mill release tests showed that the yield strength of the Grade S275 material was in the range 291 to 318 N/mm^2 and the S355 material 371 to 413 N/mm^2.

The structure was designed as a braced frame with lateral restraint provided by cross bracing of S355 flat steel, around the three vertical access shafts. The overall frame was designed to localize accidental damage, with transfer beams at second floor level together with the supporting columns and their respective restraining members designed as 'key elements' to BS5950. This involved ensuring that these elements could withstand a blast loading of 34kN/m^2, which resulted in the need for horizontal tying members to the columns at first and second floor levels. S275 rolled steel angles were used as windposts positioned at 3.0m centres along the edge beams from ground to fourth floor and at 2.25m centres above this level. These had an estimated yield stress of 280 N/mm^2.



Main beam sizes:

9.0m secondary beam :	*305x165x40 UB (Grade S275)*
9.0m primary beam :	*610x229x101 UB (Grade S275)*
6.0m primary beam :	*356x171x51 UB (Grade S355)*
9.0m perimeter beam :	*356x171x51 UB (Grade S355)*

Figure 5.2 *General plan of test building*

The beams were designed as simply-supported acting compositely with the floor slab with 95mm x 19mm diameter shear studs. The composite flooring system consisted of 0.9mm thick steel deck (PMF CF70), which was continuous over a minimum of two spans. Grade 35 lightweight concrete (nominal density 1900 kg/m^2) was specified with A142 anti-crack mesh consisting of 6mm diameter wires at 200mm centres (Figure 5.3). The overall minimum depth of the slab was 130mm.

To comply with the present UK Building Regulations [5] this type of building would require 90 minutes fire resistance. The design engineers/architects would usually specify the fire protection to the steel frame. Typical section sizes used in the frame together with their associated section factors (defined as the exposed surface area of the member divided by the volume of the member, per unit length) are included in Appendix A. These section factors are used to determine the required thickness of fire protection which would typically be in the form of a board, spray or intumescent coating.

The steel-to-steel connections consisted of fin plates (Figure 5.4) for the beam-to-beam and flexible end-plates (Figure 5.5) for the beam-to-column connections. Column splices consisted of cap and base plates.

Perimeter columns and 75% of the columns around the bracing cores were spliced once over the building height, with the heavier loaded internal columns being spliced twice. Throughout the structural design the underlying philosophy was to obtain a frame that used the minimum amount of material, was simple to manufacture, and at all stages of construction and erection reflected normal building practice rather than specialist research procedures.

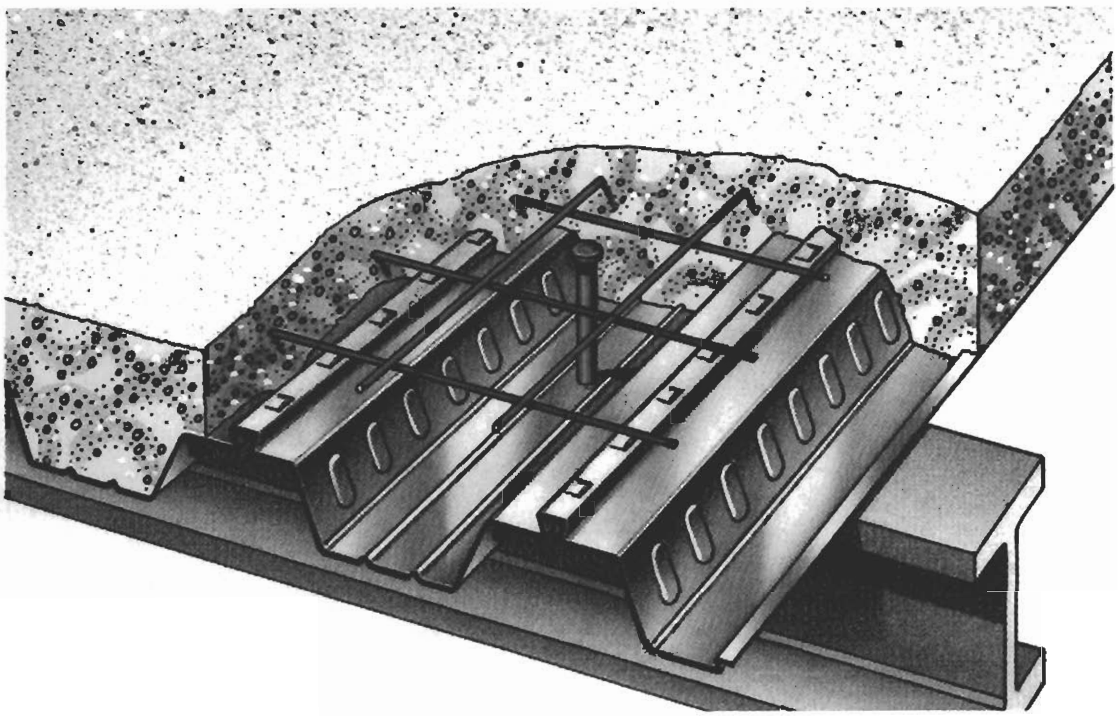


Figure 5.3 *PMF CF70 Composite slab (note, due to the raised re-entrant the mesh can be placed directly onto the deck, with the cross-wires down)*

Figure 5.4 *Fin-plate (beam-to-beam) connection*

Sand bags, each weighing 11kN, were used to simulate the applied load during the fire tests. The characteristic design loads and the load applied during the tests are given in Appendix B. Recent fire design codes use the concept of load ratio, defined as the load applied during the fire divided by the normal (cold) resistance.The load ratios for the main beams used in the frame are included in Appendix B.

Figure 5.5 *Flexible end-plate (beam-to-column) connection*

6 DETAILS OF THE TESTS

The objectives of the test programme were:-

- To provide data to verify computer models of steel frame behaviour in fire.
- To demonstrate the behaviour of large scale structures in fire.
- To provide the basis for the preparation of a more rational design methodology for steel framed buildings under fire conditions.

The two programmes of tests (BS/ECSC and BRE) took place between January 1995 and July 1996. Where possible, lessons from one test influenced the details of the following tests. In all, 6 major fire tests were carried out. These are summarised in the following table and their locations are shown in Figure 6.1.

Table 6.1 *Summary of the test programme*

Test	Sponsor	Description	Floor area (m^2)	Location
1	BS /ECSC	Restrained beam	24	level 7
2	BS /ECSC	Plane frame	53	level 4
3	BS /ECSC	1^{st} Corner	76	level 2
4	BRE	2^{nd} Corner	54	level 3
5	BRE	Large compartment	340	level 3
6	BS /ECSC	Large compartment (office)	136	level 2

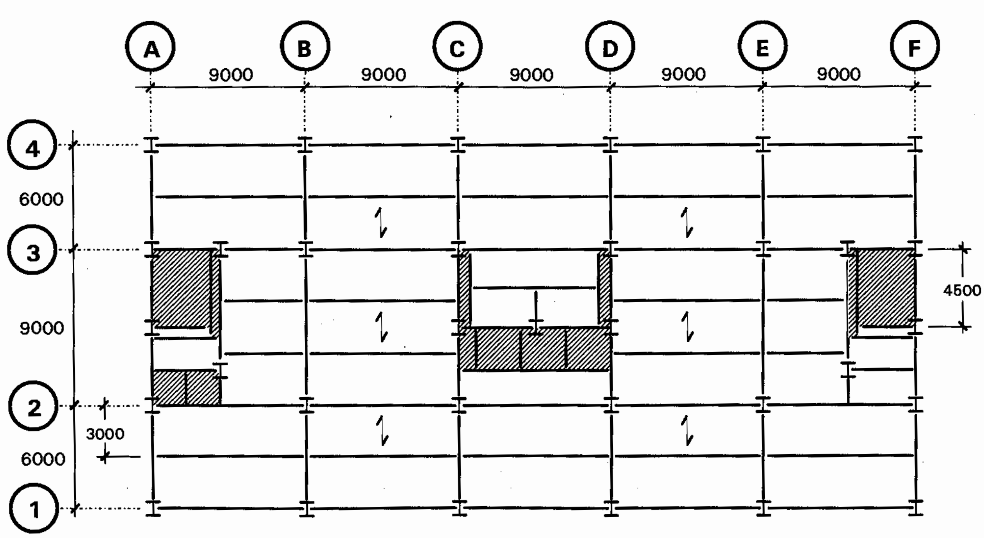


Figure 6.1 *Floor layout and location of the fire tests*

6.1 Test 1, Restrained Beam

The restrained beam test was carried out on the steelwork supporting the seventh floor of the building. A purpose built gas fired furnace, 8.0m long and 3.0m wide (Figure 6.2) was designed and constructed (on the sixth floor) to heat a 305x165x40UB, and part of the surrounding structure, spanning between two columns (254x254x89UC). The beam was heated over the middle 8.0m of its 9.0m length, thus keeping the connections as near as possible at ambient temperature.

Figure 6.2 *Purpose built gas fired furnace*

A flexible ceramic fibre curtain was placed between the underside of the slab and top of the fabricated furnace wall, allowing the floor to deflect unimpeded. Collars comprising stainless steel sheets with ceramic fibre infill were fitted around the ends of the beams as they passed through the furnace walls. This was done to allow the beams to freely deflect and also minimise leakage of hot gases along the beam. The voids between the top flange of the heated beam and underside of the steel decked floor were filled with mineral wool. Previous research has shown that this is not necessary, resulting in the common practice of leaving these voids unfilled. However, in this case the voids were filled to reduce the thermal gradient through the heated beam, thus simplifying the computer simulations. On all the other tests the voids were left unfilled.

The heated steel beam and surrounding steel structure was extensively instrumented using strain gauges, position sensors, inclinometers and thermocouples. The composite floor was also instrumented to obtain the temperature distribution through the slab, and the strain in the mesh. In total, 300 pieces of instrumentation were installed.

The beam was heated at between 3-10°C per minute until temperatures within the range of 800-900°C were recorded through the profile of the section. At this point the mid-span deflection had reached 232mm. Although at these temperatures the strength of the structural steel is less than 10% of its yield strength at ambient temperature, Figure 6.3 illustrates that, even when the test was terminated, 'runaway' displacement (instability) was not reached. The heating rate adopted was much slower in comparison to that experienced by unprotected steel in a standard fire test, and was more appropriate to a protected steel member tested over a period of 1 to 2 hours. However, an essential feature of the test was to evaluate the effect of composite action between the floor and beam and a slower heating rate would allow more heat to be conducted through the concrete to the reinforcement. At the beam's maximum temperature (887°C in the lower flange) the midspan deflection was 232mm. Once the beam had cooled back to ambient temperature the mid-span deflection recovered to 113mm.

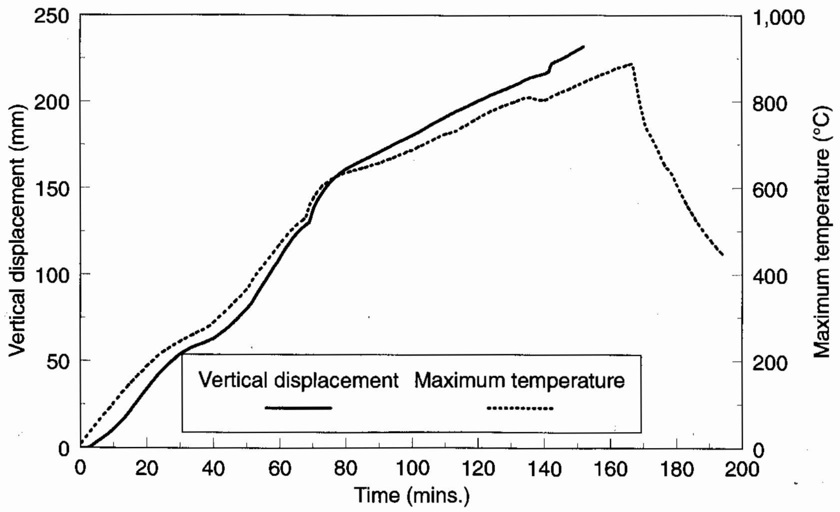


Figure 6.3 *Maximum vertical displacement and temperature recorded in the beam during the test.*

During the test local buckling had occurred at both ends of the test beam, just inside the furnace wall (Figure 6.4). In addition, close examination of the ends of the beam also revealed the lower flange had distorted as it expanded against the web of the column section. The time at which these effects occurred can be identified from the instrumentation readings. For example, from the strain gauge readings, local buckling occurred at approximately 70 minutes into the test. At which point the lower flange, web and upper flange temperatures were 554, 507 and 390°C respectively.

Visual inspection of the beam after the test showed that the end-plate connection at both ends of the beam had fractured in the region adjacent to the edge of the heat affect zone (Figure 6.5), on one side of the beam. This was caused by thermal contraction of the beam during cooling, generating very high tensile forces. Although the plate has sheared down one side, this mechanism relieves the induced tensile strains, with the plate on the other side of the beam retaining its integrity and thus providing shear capacity to the beam. The fracturing of the plate can be identified from the strain gauge readings, which show that during cooling the fracture process occurred over a period of time rather than instantaneous rupture.

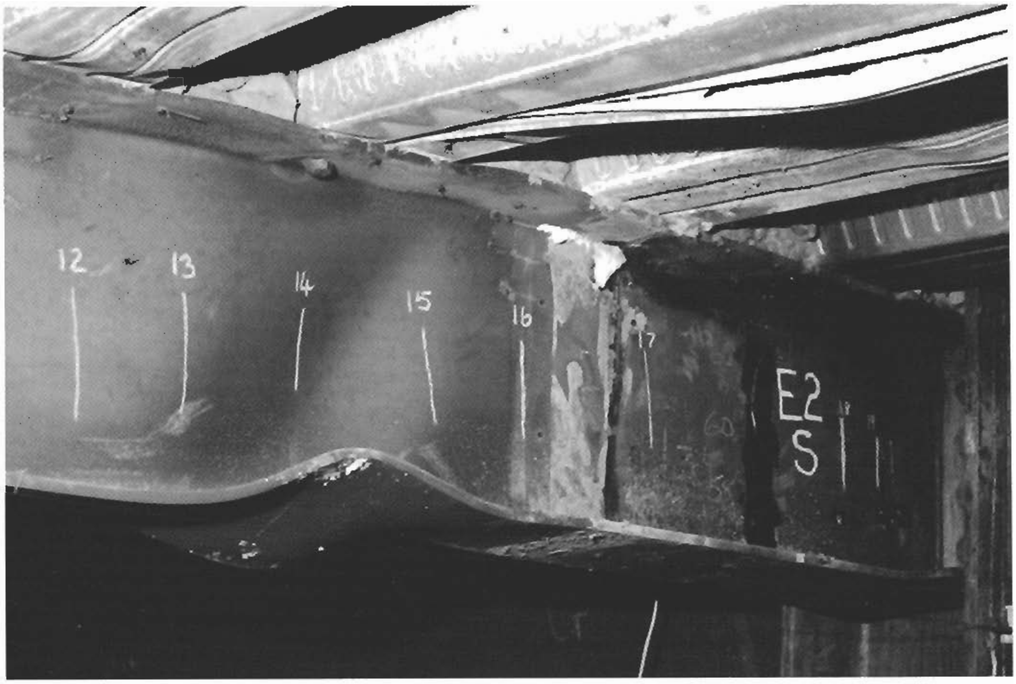


Figure 6.4 *Test 1: Local buckling of test beam just inside furnace*

Figure 6.5 *Fractured connection*

6.2 Test 2, Plane Frame

This test was designed to investigate the behaviour of the primary beams and columns on gridline B, supporting the fourth floor across the full width of the building, as shown in Figure 6.6.

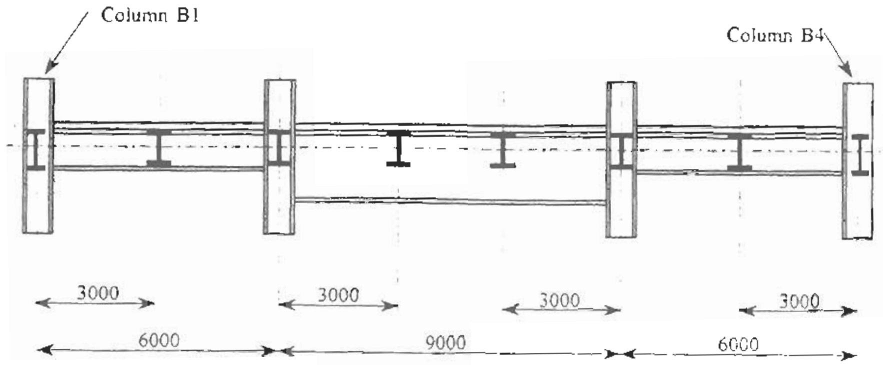


Figure 6.6 *Schematic representation of the plane frame test*

One of the objectives of the test was to investigate the behaviour of the structure in the proximity of the connections and also the behaviour of the connections themselves. In addition it was important to define the need and extent of fire protection around the connection area, when the columns are insulated.

Using concrete blockwork, a gas fired furnace 21 m long by 2.5 m wide by 4.0 m high, and insulated on the internal surface, was constructed across the full width of the building (Figure 6.7). A 400mm high ceramic fibre blanket was installed between the top of the blockwork and underside of the steel decking. This allowed the composite floor slab and primary beams to deflect freely . Slots were also built into the furnace walls to allow free movement of the secondary beams and for instrumentation bars to transmit displacement of the internal structure to externally placed transducers. Heating was provided by eight independent industrial burners mounted on one side of the furnace near to the third floor level.

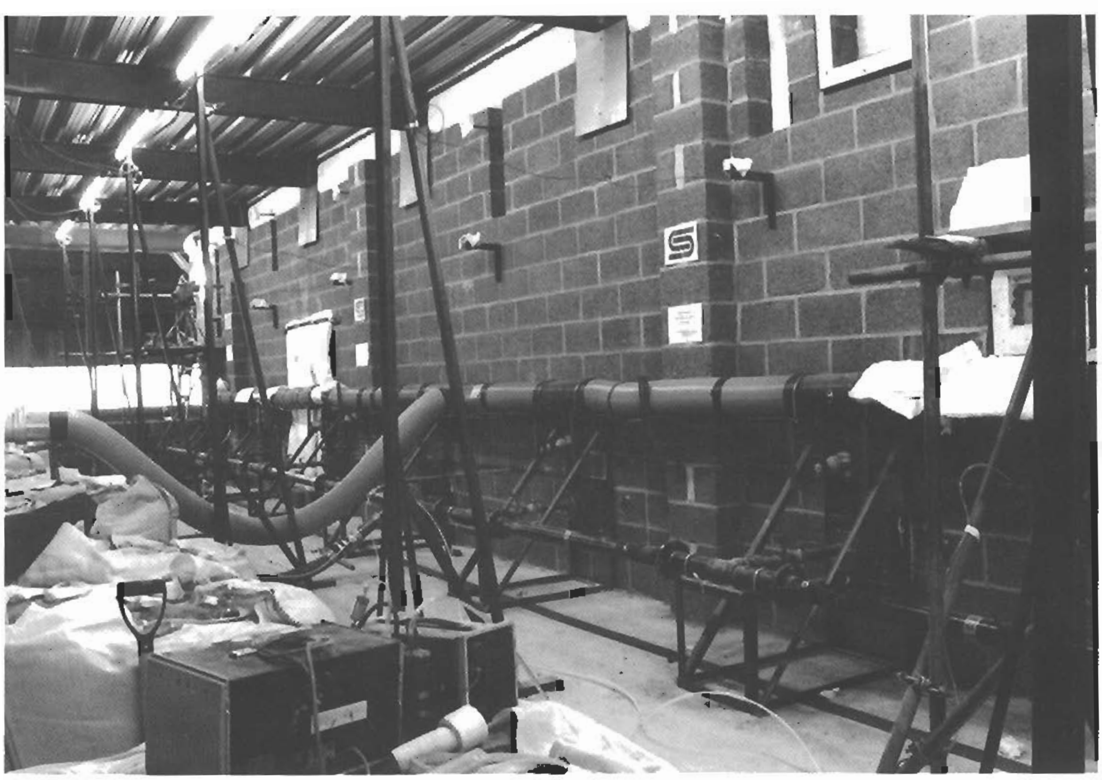


Figure 6.7 *Part of the 21m furnace*

The primary and secondary beams together with the underside of the composite floor were left unprotected. The columns were protected with mineral wool insulation boards to a height at which a suspended ceiling might be installed (although no such ceiling was present). This resulted in the top 800mm of the columns, which incorporated the connections, being unprotected. The protection was specified such that the columns would not exceed a temperature of 500°C.

Six hundred separate pieces of instrumentation were installed to measure the following:

- Temperatures at specific locations along the heated steel beams.
- Temperature profiles through the composite floor, directly above the primary beams and also at locations between the heated steel and furnace wall.
- Temperatures around the connections.
- Temperatures at specific locations within the protected and unprotected parts of the columns.
- Vertical and lateral displacements of the primary steel beams.
- Lateral displacements of the columns.
- Rotations at the main beam-to-column connections.
- Strains in the heated columns and surrounding structure.

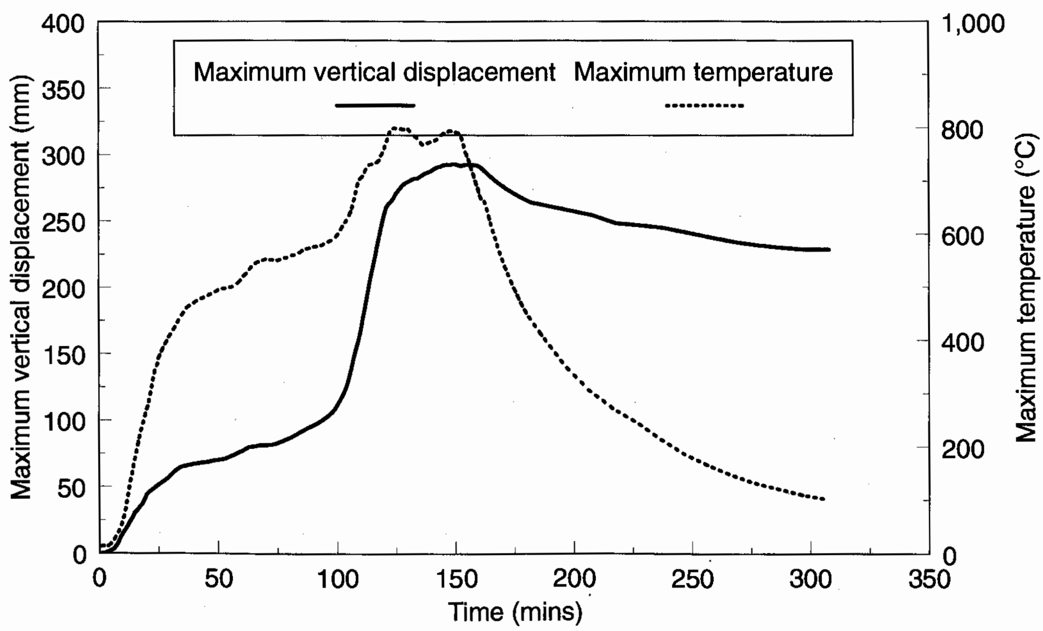


Figure 6.8 *Maximum vertical displacement and temperature of the central 9.0m steel beam*

The rate of vertical displacement of the central 9.0m steel beam increased rapidly between approximately 110 and 125 minutes. This was caused by the exposed areas of the internal columns squashing by approximately 180mm. The measured maximum displacement and temperature is shown in Figure 6.8 and the squashing of the column in Figure 6.9.

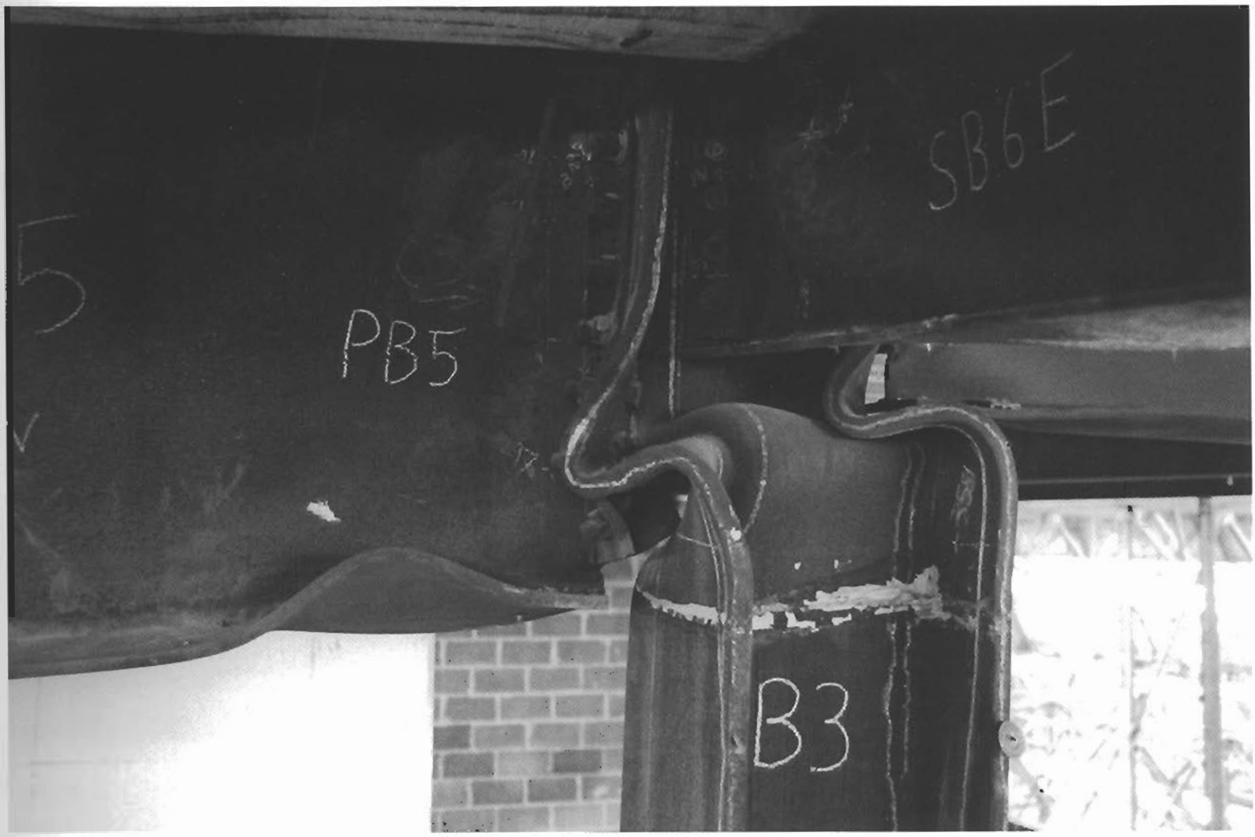


Figure 6.9 *Squashing of the exposed part of the column. (Note: the original extent of the protection can be defined from the white residue line around the column)*

Figure 6.10 indicates that the temperature of the exposed part of the column was approximately 670°C when squashing commenced.

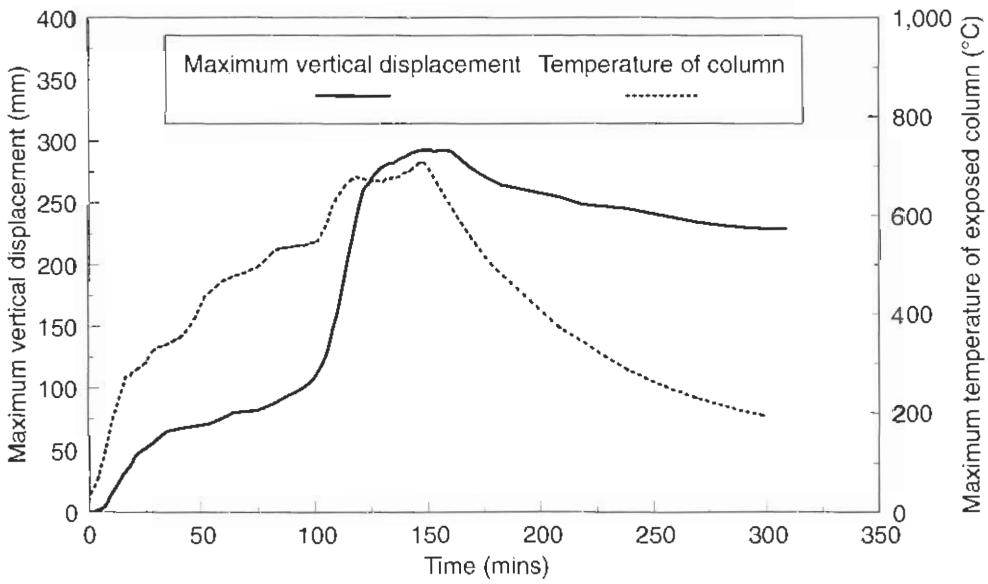


Figure 6.10 *Maximum vertical displacement of central 9.0m beam and temperature of exposed top section of internal columns*

A general view of the frame following the fire is shown in Figure 6.11, which clearly shows the localised squashing of the column. This behaviour resulted in all the floors above the fire compartment falling by approximately 180mm. In a real building this will cause the floors above the fire compartment, together with the compartment area itself, to be unusable until the damaged structure is reinstated. This would, of course, result in considerable disruption and consequential financial losses for the occupants of the building. Therefore, since the aim is to localise damage to the compartment area only, this test indicates that the columns need to be protected over their full length. This was done in all the following tests.

Figure 6.11 *General view of the frame following the test*

The secondary beams were heated over a length of approximately 1.0m on both sides of the primary beams. After the test, investigation showed that many of the bolts in the fin-plate connections had sheared (Figure 6.12). However, since secondary beams were connected both sides of the primary beams, the bolts had only sheared in the connection on one side of the primary beam. In a similar manner to the fracturing of the plate in Test 1, the bolts sheared due to thermal contraction of the beam section during cooling. The thermal contraction generated very high tensile forces which were relieved once the bolts sheared in the fin-plate on one side of the primary beam.

6.3 Test 3, 1st Corner

The objective of this test was to investigate the behaviour of a complete floor system. In particular, the role of bridging/membrane action of the floor in providing alternative load paths as the supporting steel frame reaches high temperatures.

Figure 6.12 *Bolt shear in beam-to-beam connections (fin-plates)*

Using concrete blockwork, a compartment 10m wide by 7.6m deep was constructed on the first floor of the building in one corner, as shown in Figure 6.13. A 400mm high ceramic fibre curtain was fitted between the top of the furnace wall and underside of the steel decking to allow unimpeded vertical displacement of the composite floor. Slots were also made in the furnace wall to allow all beams to deflect freely. Any remaining gaps around the furnace were loosely filled with ceramic fibre.

To ensure that the compartment walls did not contribute to supporting the applied loads all the restraints and ties in the gable wall and the top layer of blockwork were removed, together with the mineral fibre board in the expansion joints. Where the compartment wall butted up against the flange of a column section, or was built into the web of a column section, a 10mm gap was left between the steel and masonry. Therefore, both the existing and new wall construction of the furnace only provided a means of containing the fire and did not contribute to the structural performance of the building.

To avoid additional complication with the structural analysis, the wind posts on gridline 1, in the fire compartment, between the first and second floors were removed and those between the second and third floors were detached from the edge beam. The circular hollow section which provided lateral restraint to the column E2 was also removed.

Column E2 and the internal surfaces of the perimeter columns 1E, 1F and 2F, together with column-to-beam connections were protected using 25mm ceramic fibre blanket. The external perimeter beams were protected in a similar manner. All the internal primary and secondary beams including the beam-to-beam connections, were left unprotected. The voids between the underside of the steel decking and top of the steel beams were left unfilled. Figure 6.14 shows an internal view of the compartment nearing completion.

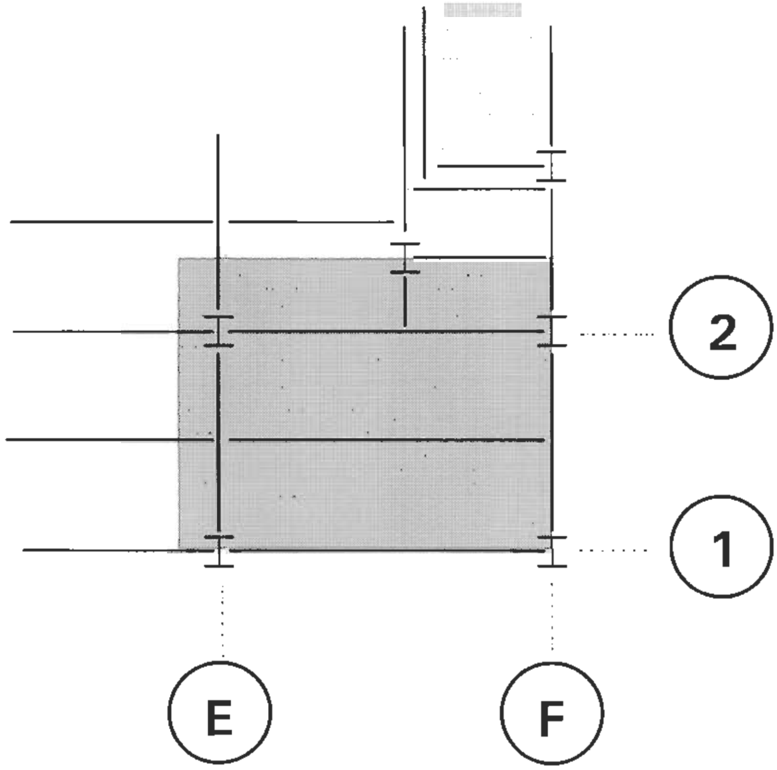


Figure 6.13 *Schematic representation of Test 3*

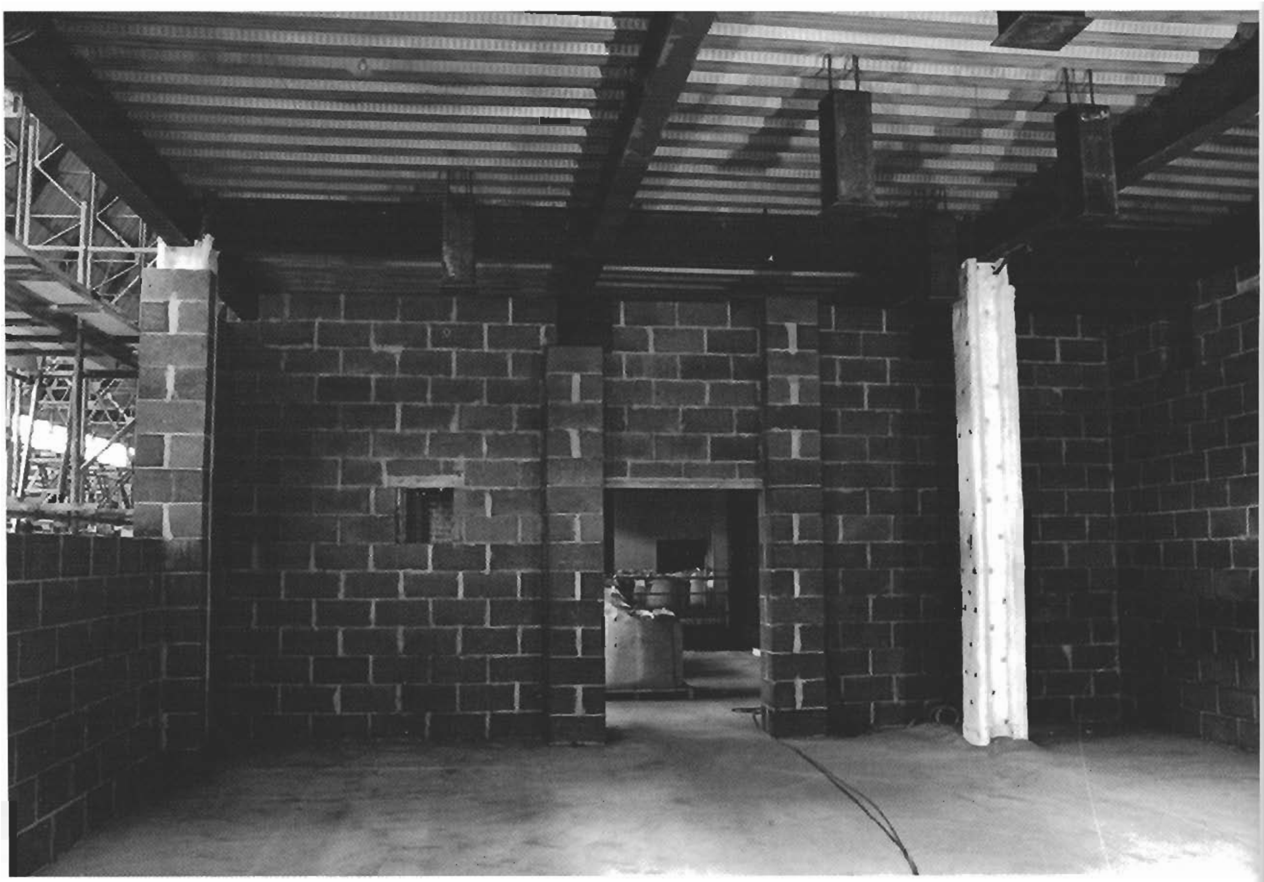


Figure 6.14 *Internal view of test compartment nearing completion*

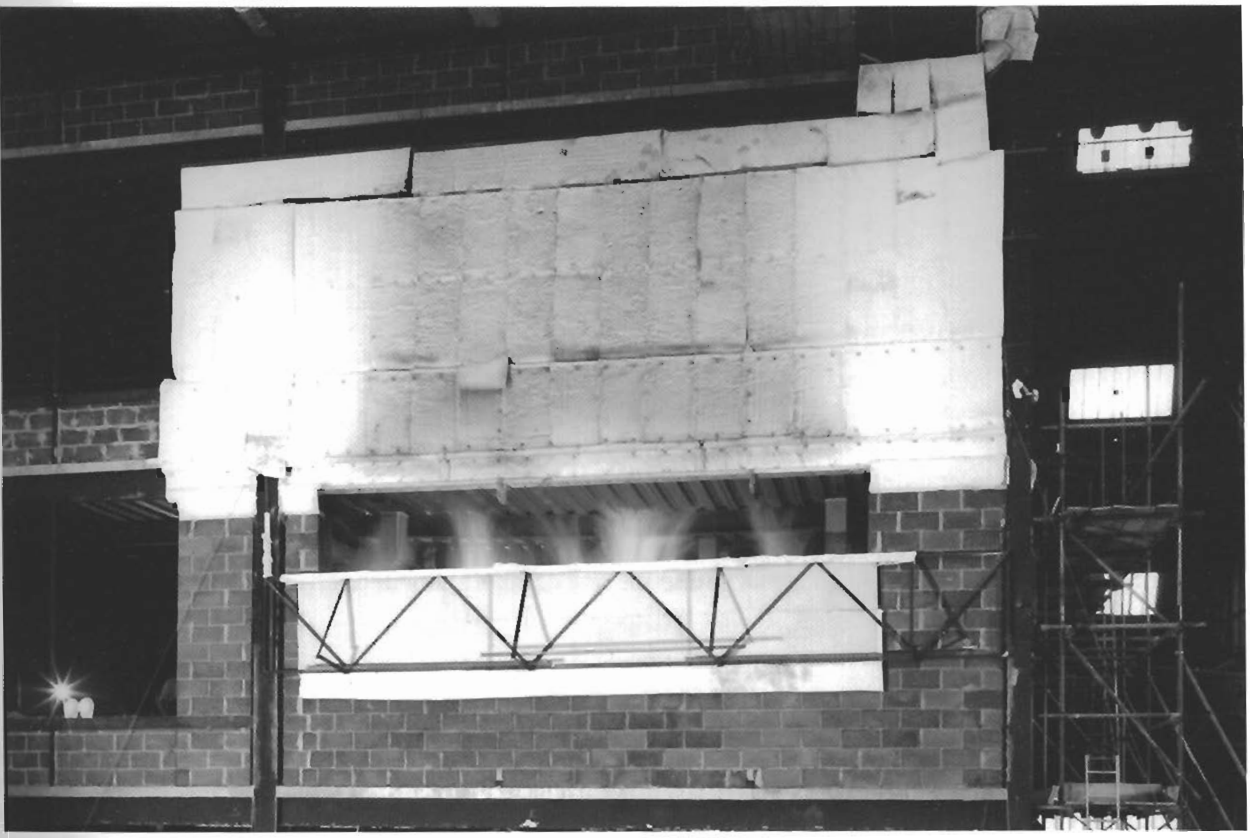


Figure 6.15 *Front view of compartment during early fire development stage*

From observations of the previous tests, it was decided that this test should be designed so that the steel temperatures exceed 1000°C. Calculations based on the parametric equations in EC1 Part 2.2[6], showed that a fire loading of 45kg of wood/m^2 of floor area with only a small adjustment to the initial ventilation conditions, would achieve this maximum temperature. The fire loading was distributed throughout the compartment in the form of twenty 1m square cribs, using 50x50mm softwood. At the height of the fire the measured heat release was 19MW. Figure 6.16 shows a comparison between the measured and calculated atmosphere temperatures.

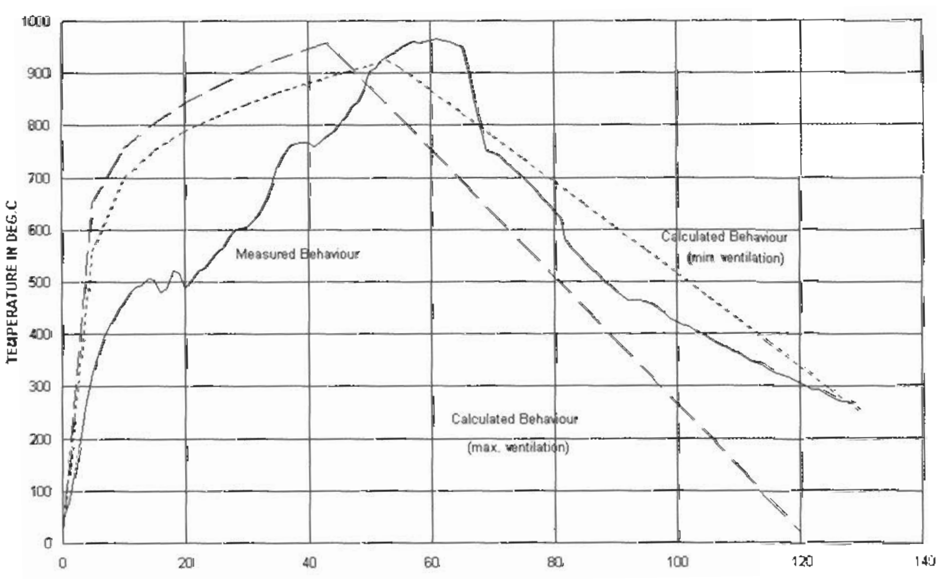


Figure 6.16 *Comparison between measured and predicted atmosphere time/temperature response*

Indicative protected steel sections were included in the test compartment to measure the fire severity in relation to a heating period in the ISO 834[7] fire resistance test. This was found to be equivalent to 86 minutes which compares favourably with 94.5 minutes calculated using the EC1 design method and 89 minutes calculated using the Pettersson formula[8]. In both calculations the thermal properties of the compartment boundaries were considered.

Six hundred separate pieces of instrumentation were installed to measure:

- Steel temperature profiles along the primary, secondary and edge beams.
- Temperature profiles through the composite floor.
- Steel temperatures at specific locations along the columns.
- Steel temperatures around the connections.
- Vertical and lateral deflections and displacements of the beams, columns and floor slab.
- Beam and column rotation at the connections.
- Strain profiles across the columns within the test compartment and the surrounding structure.
- Strain across the composite floor.
- Compartment atmosphere temperatures.

The maximum vertical displacement of 428mm occurred at the centre of the secondary beam spanning between gridlines E and F, located between gridlines 1 and 2. On cooling this recovered to a permanent displacement of 296mm. The maximum recorded temperature of the secondary beam was 935°C (Figure 6.17)

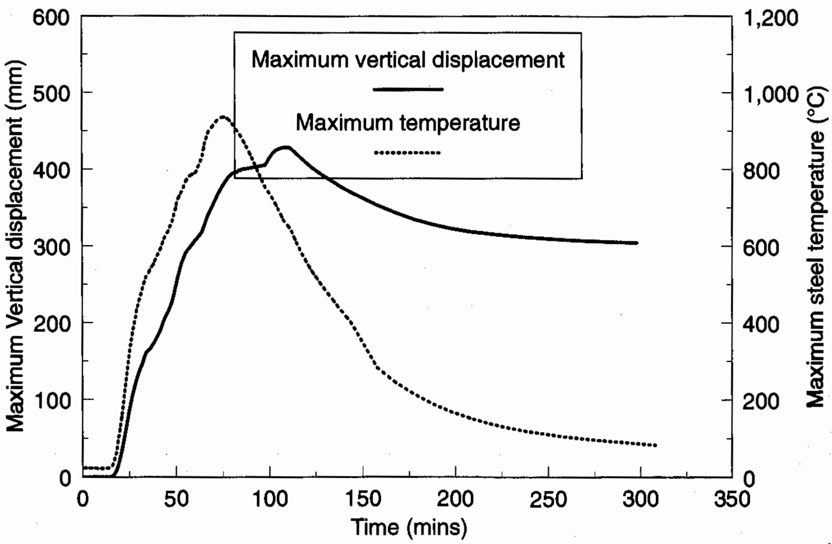


Figure 6.17 *Maximum vertical displacement and temperature of 9.0m secondary beam*

All of the combustible material within the compartment was consumed by the fire, as shown in Figure 6.18. Overall the structure behaved extremely well with no signs of collapse. Figure 6.19 shows the deformation of the structure once the furnace was removed.

Figure 6.18 *Internal view of compartment (looking towards gable wall) following the test*

[redacted]

Figure 6.18 *Internal view of compartment (looking towards gable wall) following the test*

Figure 6.19 *Structural deformation following the fire*

Extensive buckling occurred in the proximity of the beam to column connections (Figure 6.20). It was of interest to note the behaviour of the fin-plate connection of the internal 9.0m secondary beam. Local buckling occurred at the beams's end which was connected to the primary beam on grid-line E (Figure 6.21). This was caused by axial restraint from the connecting steel members and composite slab. At the other end of the beam which was connected to an external beam, no local buckling occurred (Figure 6.22). This was due to the thermal expansion, of the secondary beam, causing the external beam to twist, thus providing insufficient restraint to cause local buckling. Both connections, of the beam, experienced no shearing of the bolts indicating that tensile forces induced during cooling were not very high.

Figure 6.20 *Extensive local buckling occurred in the proximity of the beam to column connections*

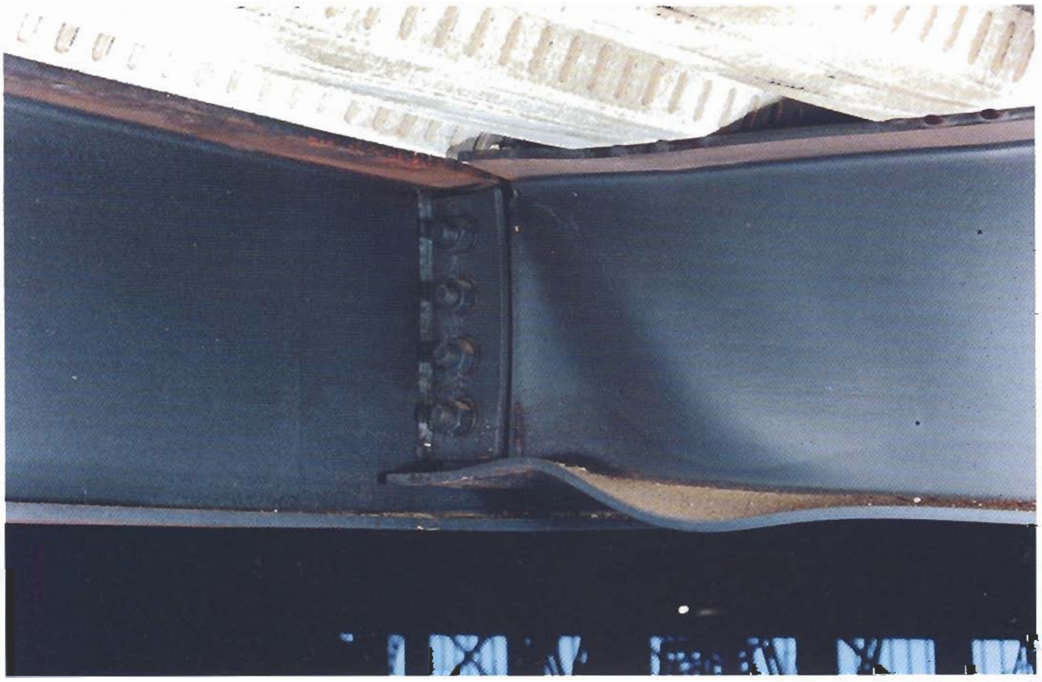


Figure 6.21 *Fin-plate connection of the secondary 9.0m beam showing local buckling (connection is secondary to primary beam on grid-line E)*

Figure 6.22 *Twisting of external beam to external beam*

6.4 Test 4, 2nd Corner

This test was carried out on the corner bay between gridlines E to F and 3 to 4 (Figure 6.23). The compartment was built on the second floor with the third floor steel and slab being heated. The internal boundaries of the compartment were constructed using steel stud partitions with fire resistant board and placed centrally on gridlines E and slightly offset from gridline 3, as shown in Figure 6.24. The stud partition was specified to have 2 hours fire resistance, with a deflection head of 25mm (Figure 6.25). The existing full-height blockwork wall formed the boundary on gridline F. On gridline 4 the existing 1m high blockwork dado wall remained in place and a glazing system was constructed. The compartment was totally enclosed with all windows and doors closed. The internal column on gridline E3 was fully protected up to the underside of the floor slab, including the connections. Protection was also provided to the two external columns on gridlines E4 and F4. The two remaining columns were outside the compartment, behind the shaft walling used to protect the stairwell.

Twelve timber cribs were used to give a fire load of 40kg/m^2. This resulted in a total fire load of 2160kg over the compartment area of 54m^2.

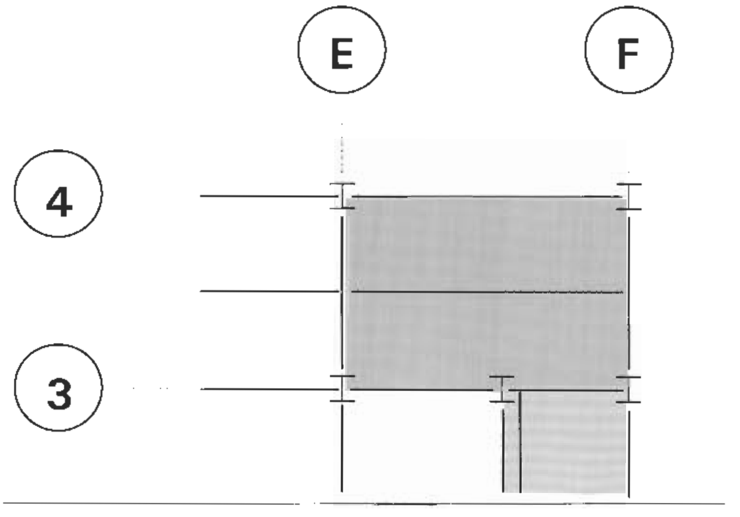


Figure 6.23 *Schematic of test 4 (second corner test)*

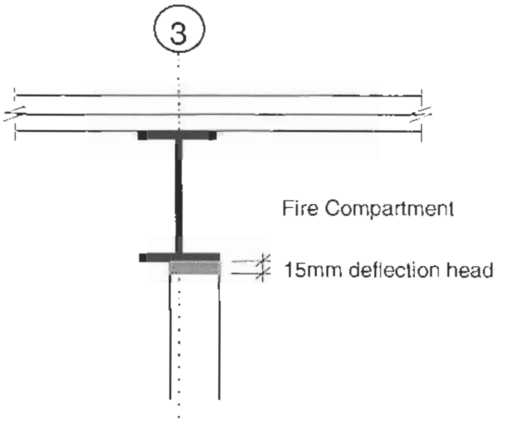


Figure 6.24 *Stud partition arrangement on gridline 3*

Figure 6.25 *Start of fire test 4*

278 thermocouples were used to monitor the temperature of the steel columns and beams, the concrete slab, and the atmosphere temperature within the compartment. Also, temperatures were measured at various locations immediately outside the compartment area including indicative steel sections which were suspended from the compartment ceiling. The internal and external temperatures of the gable wall were also monitored.

300 strain gauges were used to measure the response of the structure to the fire. Seven columns on the fire floor, together with the floor above and below, and the seventh floor, were instrumented in addition to four beams on the fire floor outside the fire compartment. Additional strain gauges were used to measure the response of the reinforcing mesh and the concrete surface on the third floor.

47 travel displacement transducers were used to measure the deformation of the concrete slab on the third floor and the axial and lateral movement of the columns on gridlines E3 and E4. 12 clinometers were used to measure the major axis rotations of the connections within the compartment. An innovative laser system monitored any movement of the gable wall during the fire test.

Figure 6.26 (a) *Fire test in progress*

Figure 6.26 (b) *Fire test in progress*

The development of the fire was largely influenced by the lack of oxygen within the compartment. After an initial rise in temperature the fire died down and continued to smoulder until the fire brigade intervened to vent the compartment by removal of a single pane of glazing. This resulted in a small increase in temperature followed by a decrease. Flashover did not occur until a second pane, immediately below the first, was removed. This initiated a sharp increase in temperature which continued as the fire developed. The progress of the fire can be seen in Figure 6.26. The maximum recorded atmosphere temperature in the centre of the compartment was 1051°C after 102 minutes (Figure 6.27). The maximum steel temperature of 903°C was recorded after 114 minutes in the bottom flange of the central beam.

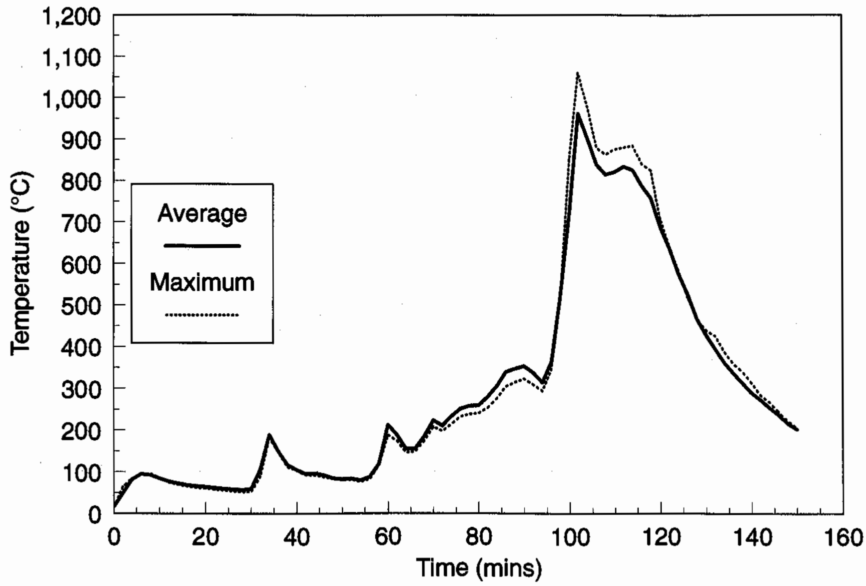


Figure 6.27 ***Maximum and average recorded atmosphere temperature throughout the fire compartment***

The maximum recorded value of 269mm for the slab displacement occurred in the centre of the compartment after 130 minutes (Figure 6.28). Measurements taken the following day indicated that the slab had recovered to a permanent displacement of 160mm.

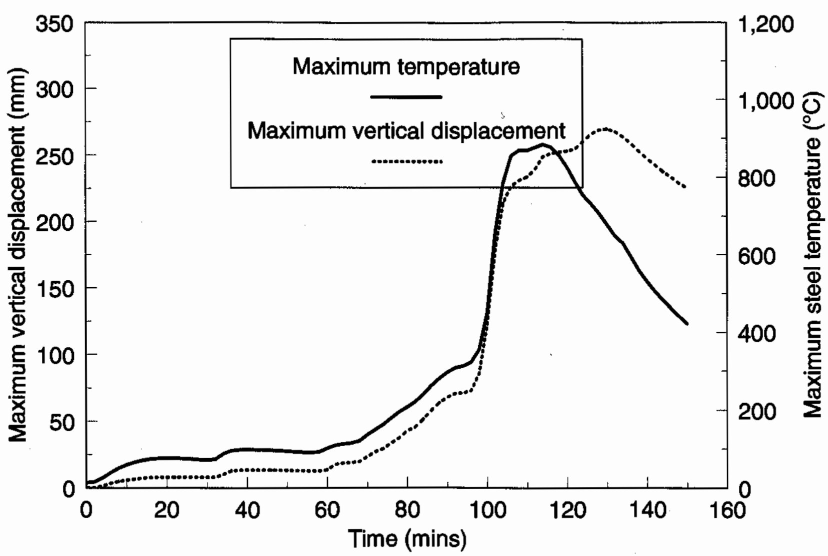


Figure 6.28 ***Recorded vertical displacement and temperature of the secondary beam at the centre of the fire compartment***

The unprotected edge beam was observed during the test to be completely engulfed in fire (Figure 6.26). However, the maximum temperature of this beam only reached 680°C, with a corresponding maximum displacement of 52mm, recorded after 114 minutes (Figure 6.29). Even with lower temperatures, compared to the internal beams, the displacement of the edge beam was very small. This was attributed to the wind posts above the compartment, which acted in tension during the test, providing support to the heated beam.The tops of these wind posts were fixed by bolts through slotted holes which were 80mm in length. Therefore a large proportion of the 52 mm displacement could be attributed to the bolt slippage. Unfortunately the position of the bolts was not recorded before the test. Figure 6.30 illustrate the permanent deformation of the edge beam following the test. It should be noted that since the displacements of the edge beam were very small no damage occurred to the facade of the building above the compartment.

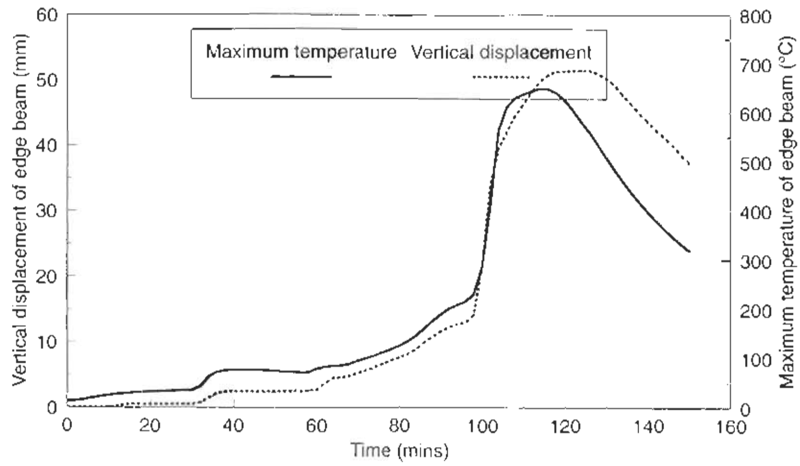


Figure 6.29 *Maximum vertical displacement and temperature of edge beam*

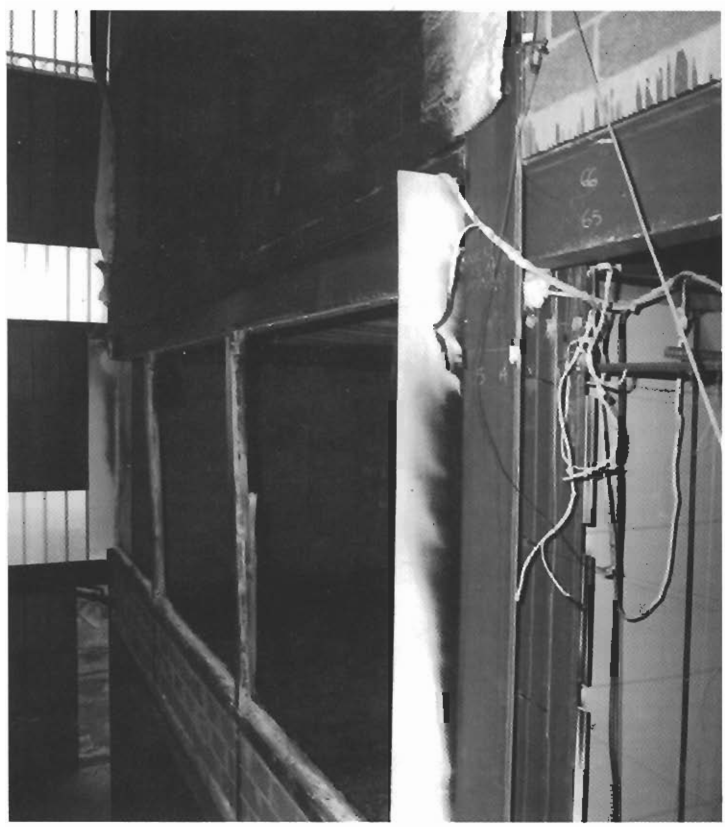


Figure 6.30 *Deformation of the unprotected edge beam following the test*

The internal compartment walls were constructed on the gridlines under unprotected beams. In practice these beams would require protection to comply with the insulation criterion, for compartmentation, of the

ISO 834 test. However, it is of interest to notice how well the compartment wall performed (Figure 6.31), with its integrity being maintained for the duration of the test. On removal of the wall it could be seen that the beam had distortionally buckled over most of its length (Figure 6.32). This was caused by the high thermal gradient through the cross-section of the beam (caused by the positioning of the compartment wall), together with high restraint to thermal expansion.

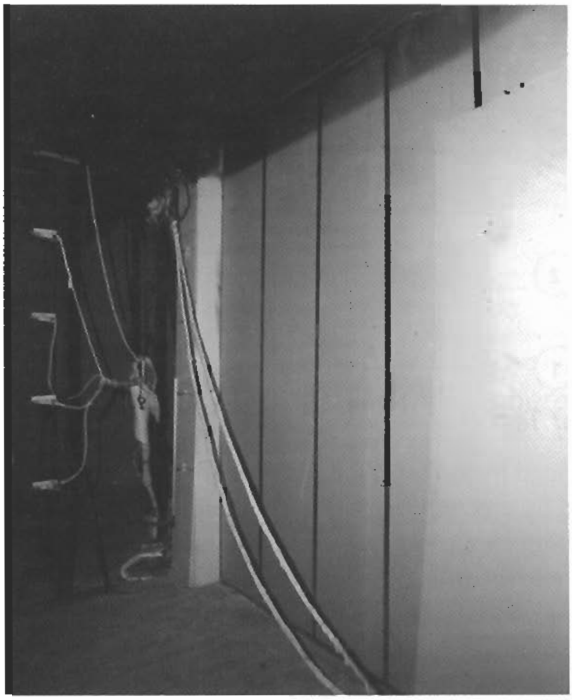


Figure 6.31 *Integrity of the compartment wall was maintained during the test*

Figure 6.32 *Distortional buckling of the beam above the compartment wall (after removal of the wall)*

No local buckling occurred in any of the beams, and the connections showed none of the characteristic signs of high tensile forces which were encountered on cooling in the previous tests.

6.5 Test 5, Large Compartment

This test was carried out between the second and third floor, with the fire compartment extending over the full width of the building and between gridline A and 0.5 m from gridline C, covering an area of 340m^2.

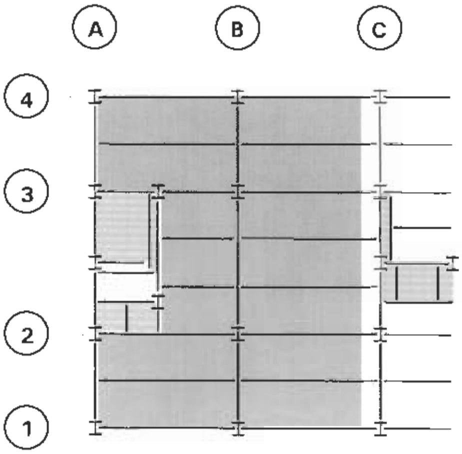


Figure 6.33 *Schematic of Test 5*

The fire load of 40kg/m^2 was provided by timber cribs arranged uniformly over the floor area (Figure 6.34). The compartment was constructed by erecting a fire resistant wall across the full width of the building and by constructing additional protection to the lift shaft. Double glazing was installed on two sides of the building on gridlines 1 and 4. To allow sufficient ventilation for the fire to develop the middle third of the glazing on both sides of the building was left open. All the steel beams, including the perimeter beams, were left unprotected. The internal and external columns were protected up to and including the connections.

Figure 6.34 *Timber cribs within the compartment*

In a manner similar to test 4, the ventilation condition governed the severity of the fire. There was an initial rapid rise in temperature as the glazing was destroyed creating large openings on both sides of the building. The large ventilation area in two opposite sides of the compartment gave rise to a fire of long duration but lower than expected temperatures. The maximum recorded atmosphere temperature was 746°C (Figure 6.35), with a maximum steel temperature of 691°C, recorded at the centre of the compartment.

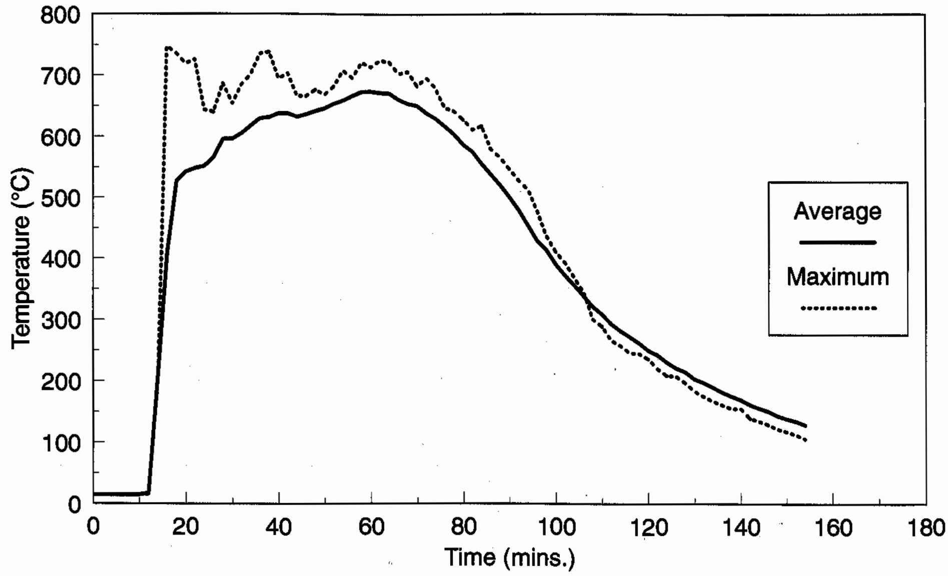


Figure 6.35 *Maximum and average recorded atmosphere temperature throughout the fire compartment*

179 thermocouples were used to monitor the temperature of the steel columns and beams, the concrete slab, and the atmosphere temperature within the compartment. Also, thermocouples were used to measure temperatures immediately outside the compartment wall next to gridline C, and in the hanger steel. Due to limited available resources some of the steel beams within the compartment area were not instrumented, and thus their temperature response was not recorded.

204 strain gauges were used to measure the response of the columns, reinforcement mesh and surface of the concrete slab. 50 displacement transducers were used to measure the vertical displacement of the beams and slab within the compartment. The response of the fire resistant wall next to gridline C was monitored using 6 displacement transducers. A laser system was used to measure the movement of the gable masonry walls.

The maximum slab displacement reached a value of 557mm (Figure 6.36) which was recorded halfway between gridlines 2 and 3 and B and C. This recovered to a value of 481mm when the structure cooled. The temperatures on only one of the 9.0m internal secondary beams (on gridline 3) were measured (Figure 6.36).

Figure 6.37 shows the structural response in the latter stages of the fire. It can be seen that the structure performed very well with stability and integrity of the compartment being maintained during the heating stage of the fire. However, the fire was not very severe, with a maximum recorded steel temperature of 691°C.

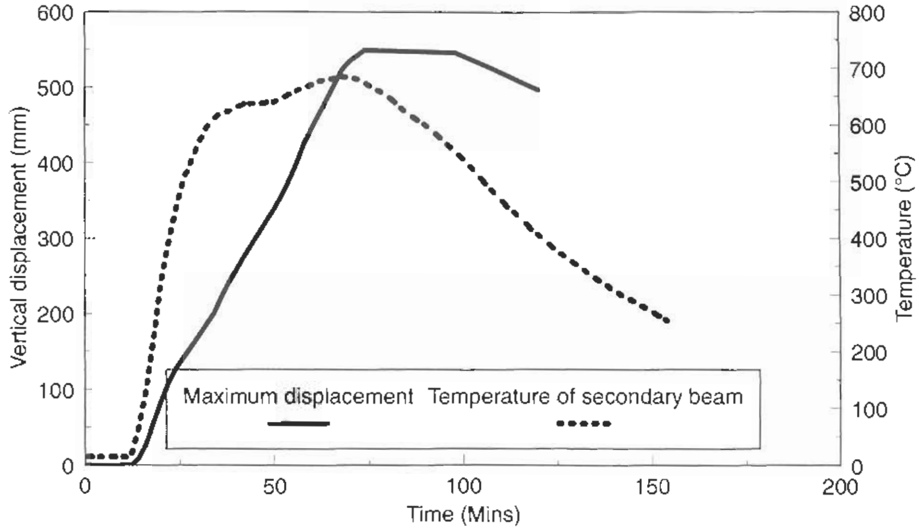


Figure 6.36 *Maximum slab displacement and temperature of 9.0m secondary beam*

Figure 6.37 *Structural behaviour towards the end of the test (notice the local buckling on the second internal secondary beam)*

Extensive local buckling occurred in the proximity of the beam-to-beam connections (Figure 6.38). On cooling a number of the end-plate connections fractured down one side; in one instance the web detached itself from the end-plate (Figure 6.39).

Figure 6.38 *Local buckling in the proximity of the steel-to-steel connections*

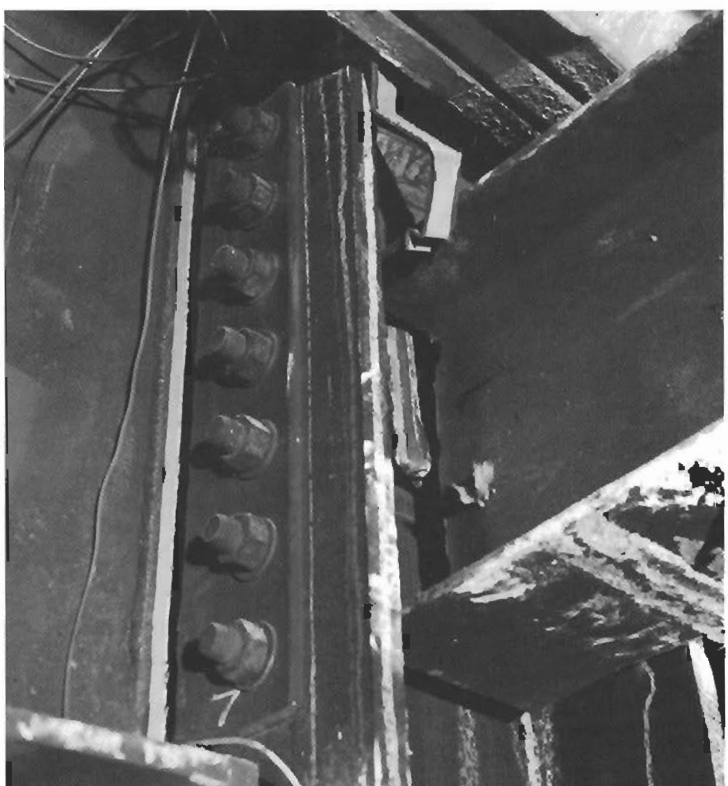


Figure 6.39 *Connection where the beam web has detached itself from the end-plate during the cooling phase of the fire*

6.6 Test 6, Simulated Office

The aim of this test was to demonstrate that the type of structural behaviour observed in the earlier tests would also occur when the building was subjected to a more realistic fire scenario, while at the same time investigating other aspects of structural behaviour not previously addressed.

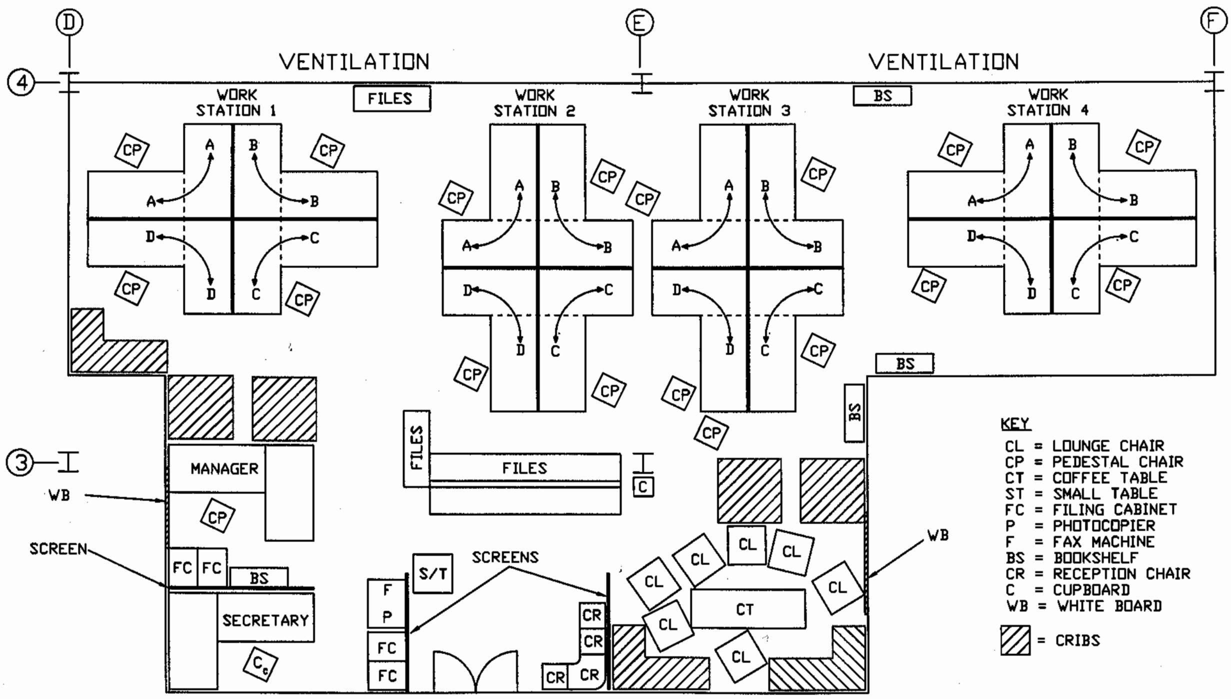


Figure 6.40 *Schematic of Test 6*

A compartment 18m wide and up to 10m deep with a floor area of 135m^2 was constructed between the first and second floors, using concrete blockwork. A gap of approximately 250mm was left between the top of the blockwork and underside of the steel decking, together with slots around the steel beams, to allow the heated structure to deform freely. These gaps were filled with a insulating ceramic fibre blanket. The design of the compartment was such that it represented an open plan office (Figure 6.40) and contained a series of work stations consisting of modern day furnishings, computers and filing systems (Figure 6.41).

Figure 6.41 *Internal view of the compartment*

Although it was possible to identify a wide range of scenarios with respect to fire loading, ventilation, method of ignition, fire growth and spread, the test conditions were designed and calculated to create the most severe fire possible. For example:

- Windows were provided along one wall using single panel aluminium glazing in which the total area of fenestration was equivalent to 20% of the total floor area (which corresponds to the minimum requirements given in the UK Building Regulations). The relative dimensions of the frames with respect to height and width were determined on the basis of providing the most detrimental opening factor to achieve near maximum compartment temperatures when all the glazing was destroyed during the test.
- To ensure that the compartment was not starved of oxygen in the early stages of the test some of the glazing was omitted. The extent of the initial opening was governed by the heat release rate required to generate a hot gas layer of 500-600°C, necessary to cause flashover.
- Just above the windows and below the edge beam, a ceramic fibre curtain (450mm deep) was installed to simulate the end closure of a suspended ceiling. This also had the additional benefit of lowering the height of the hot gas layer to aid flashover.
- The total fire loading was equivalent to 46 kg wood/m^2 of floor area by calorific value. Based on previous surveys of the type of loading found in typical offices, the fire load consisted of 20% plastics, 11% paper and 69% wood. The quantity of fire combustible material was in excess of the 95% fractile for office fire loadings. This is higher than the 80% fractile currently proposed in both European design recommendations and the new UK Fire Engineering Code Draft for Development.[9]
- The location of the fire source was at the rear of the compartment which involved igniting several cribs made up of 81% wood and 19% polypropylene. This simulated a condition in which a small fire was established and was then left to grow on its own accord.

Within the compartment the columns together with the beam-to-column connections were protected using 25mm ceramic fibre blanket. Both the primary and secondary beams remained totally exposed including all the beam-to-beam connections. The gable wall was left as originally constructed and all wind posts, ties and wall restraints remained in place.

Within 10 minutes of ignition, local atmosphere temperatures were in excess of 900°C. The recorded maximum atmosphere and unprotected steel temperatures were 1213°C (Figure 6.42) and 1150°C respectively. At the height of the fire (Figure 6.43), the calculated heat release rate was 58MW. A maximum vertical displacement of 640mm was recorded after 62 minutes (Figure 6.44)

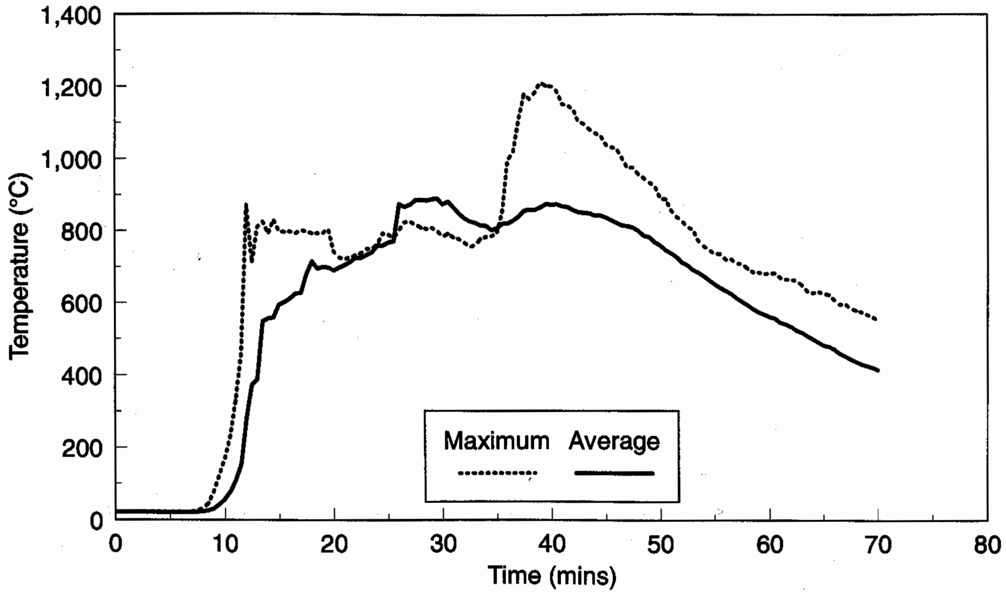


Figure 6.42 *Maximum and average recorded atmosphere temperature 1.2m below steel deck*

Figure 6.43 *Test 6 at the height of the fire*

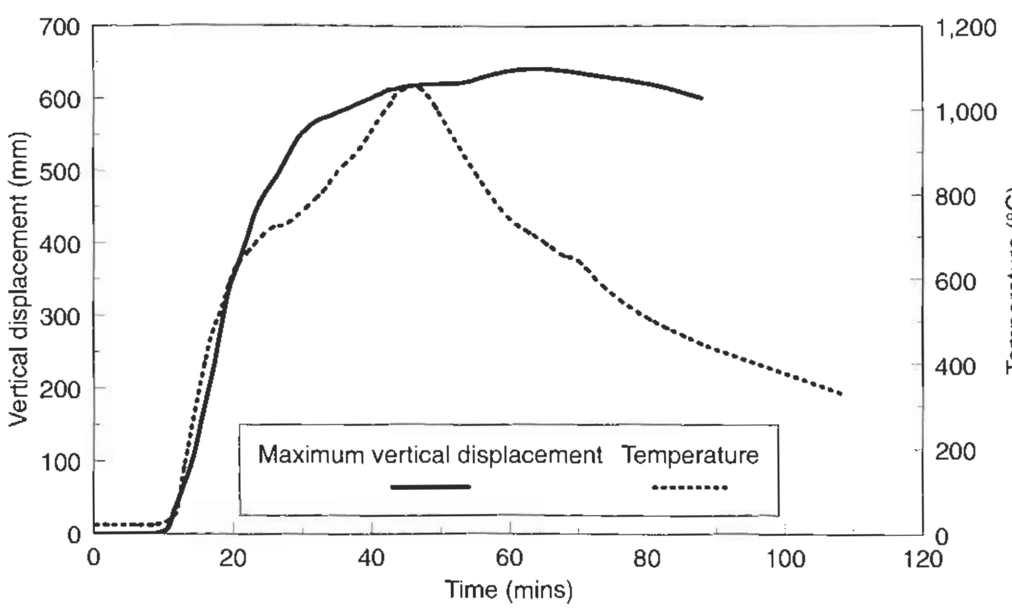


Figure 6.44 *Maximum displacement and temperature of 9.0m secondary beam*

All the combustible material in the compartment was consumed (Figure 6.45) including the contents of the filing cabinets. From the temperatures measured by protected indicative specimens suspended from the floor slab, the fire severity was found to be equivalent to 74 minutes in the standard fire resistance test.

The structure showed no signs of failure, however extensive cracking occurred around the internal column (Figure 6.46). The evidence suggests this occurred during the latter stages of the cooling phase.

Figure 6.45 *Internal view of compartment following the test*

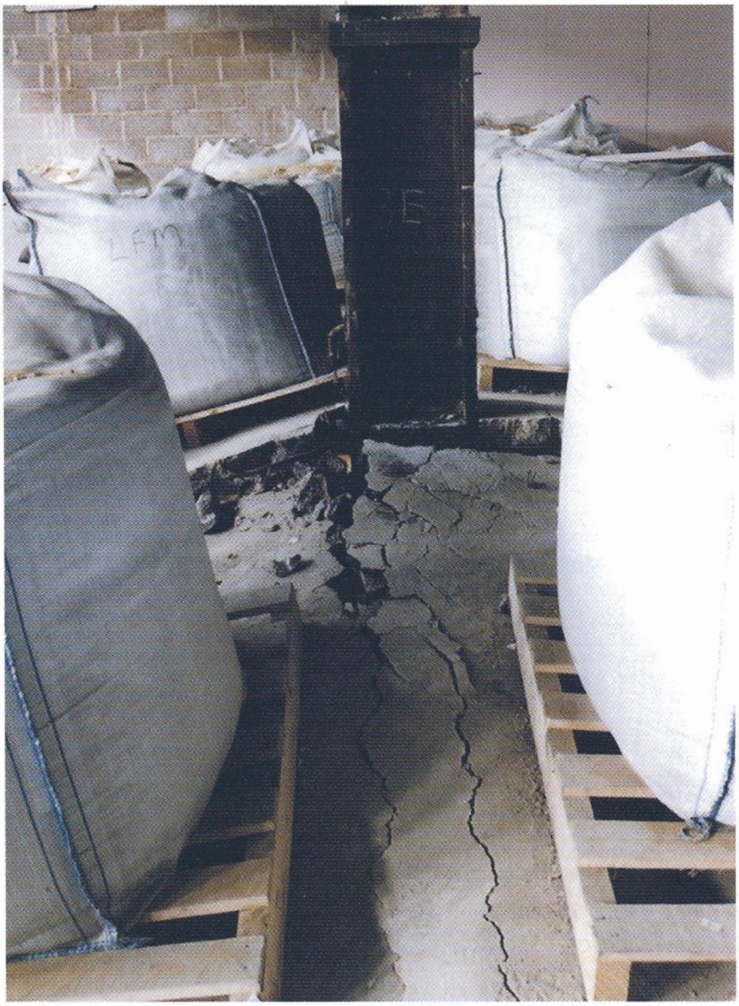


Figure 6.46 *Cracking of the composite slab around internal column*

7 COMPARISON WITH EXISTING STRUCTURAL DESIGN

The structural fire codes, BS5950 Part 8[10], EC3 Part 1.2[11] and EC4 Part 1.2[3] are based predominantly on test results carried out in accordance with ISO 834. These test results have been supplemented by computer models to enable simple design rules to be developed.

The design methods assume that the steel member acts in isolation and ignores any beneficial or detrimental interaction between the member and surrounding structure. To allow a direct comparison between existing design methods, a computer model was used to predict the structural response of the beams shown in Figure 7.1 when they are considered as isolated members. The model used, has previously been shown to model simple beams in fire tests with a high degree of accuracy. The beams were assumed to be simply supported composite beams, with the concrete top flange having an effective width of span/4 (Figure 7.2). To allow a direct comparison with the recorded displacements from the tests, the recorded temperatures were used in the structural modelling.

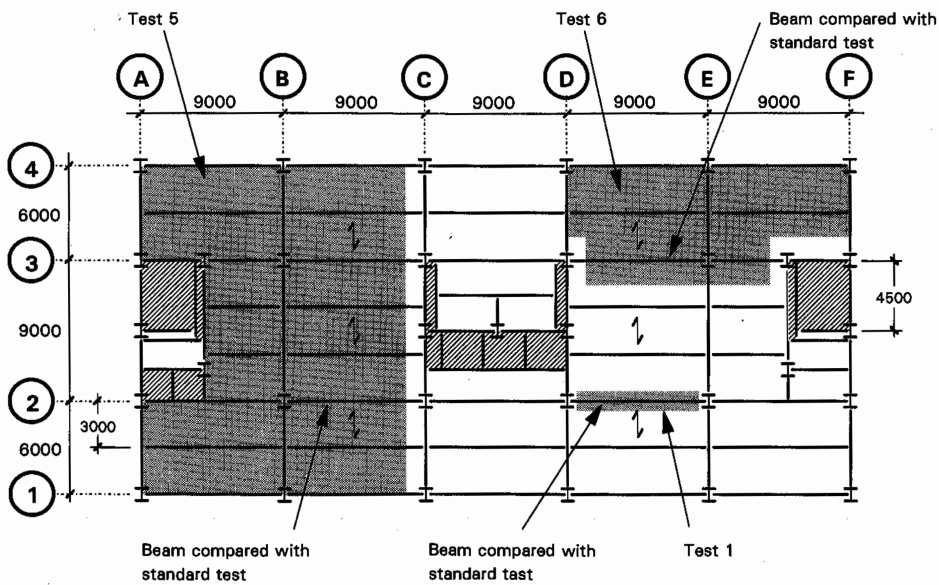


Figure 7.1 *Location of beams in the frame compared with the 305x165x46UB tested in a standard fire test*

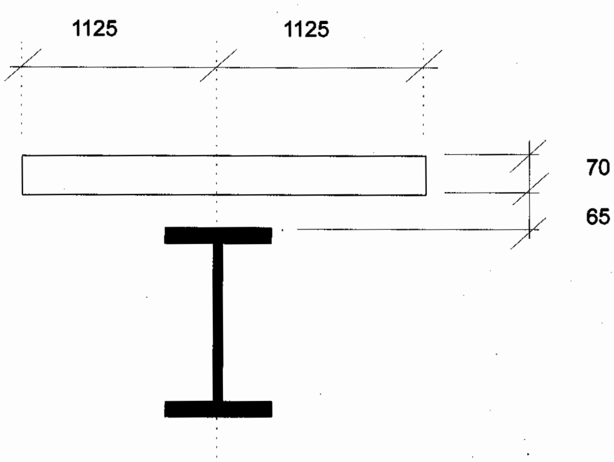


Figure 7.2 *Beam cross-section assumed in existing design methods*

In Test 1, only one beam and a small proportion of the composite floor were heated. Figure 7.3 shows a comparison between the vertical displacement recorded in the test and those predicted from modelling an isolated composite beam. The vertical displacements are similar up to approximately 250°C. However, after this temperature, the vertical displacements recorded in the test were much lower than those predicted using the model. As the temperatures increase, the displacements predicted by the model converge towards the critical temperature given in design codes for this type of beam. It is generally considered that the main reason for the test displacements being much lower, than the isolated beam, was due to the bridging action of the composite floor, spanning across the heated beam. This effectively redistributed load away from the weakening beam.

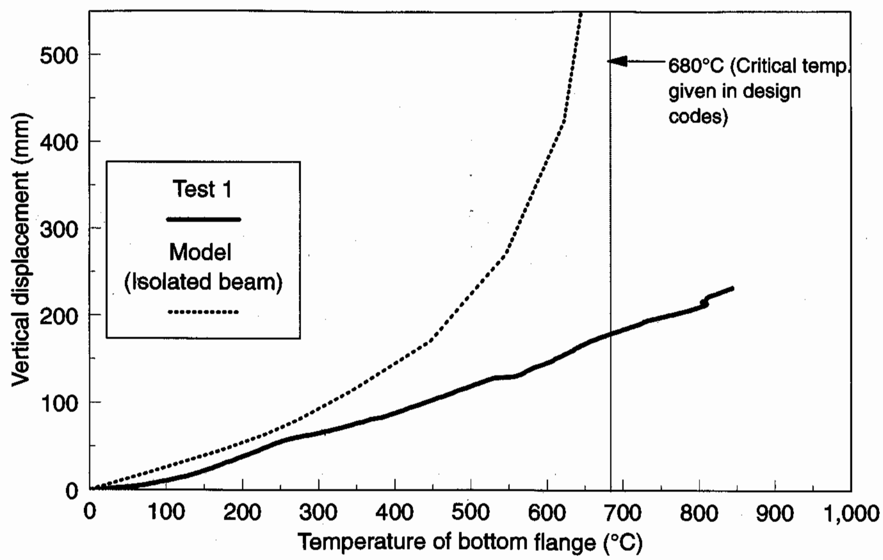


Figure 7.3 *Comparison between beam in Test 1 and beam modelled as an isolated member*

The comparisons between the beams shown in Figure 7.1 for Tests 5 and 6 and modelled isolated beams are shown in Figures 7.4 and 7.5 It can be seen that in both instances the actual test results are lower than predictions from the modelled isolated beams. Unfortunately, in Test 5 the steel temperatures did not exceed the critical temperature given the design codes. However, in Test 6 the steel temperatures reached over 1100°C with no sign of structural collapse. In addition, in Test 6, it can be seen that the rate of displacement decreased when approximately 730°C was reached. This was almost certainly due to the bridging/membrane action of the composite slab.

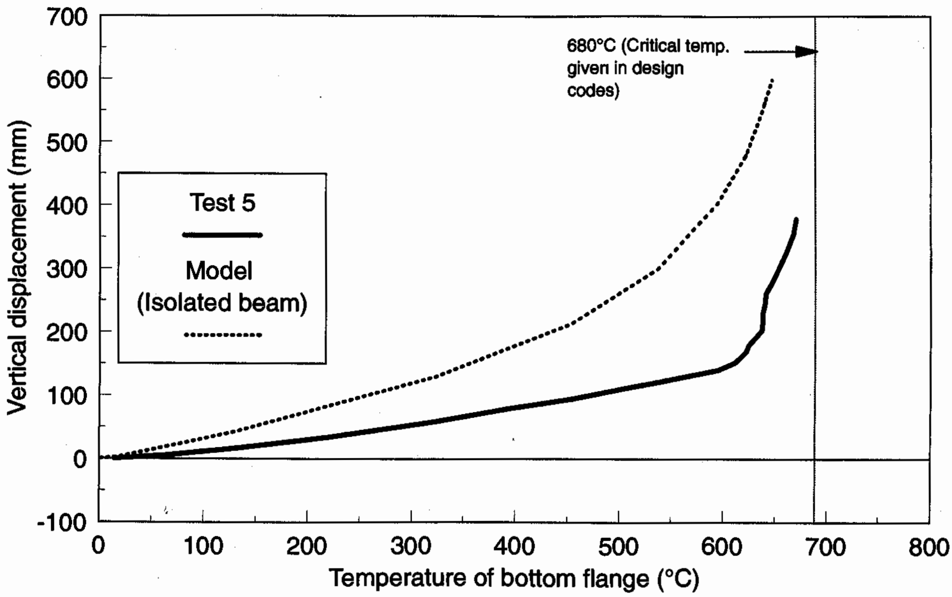


Figure 7.4 *Comparison between beam in Test 5 and beam modelled as an isolated member*

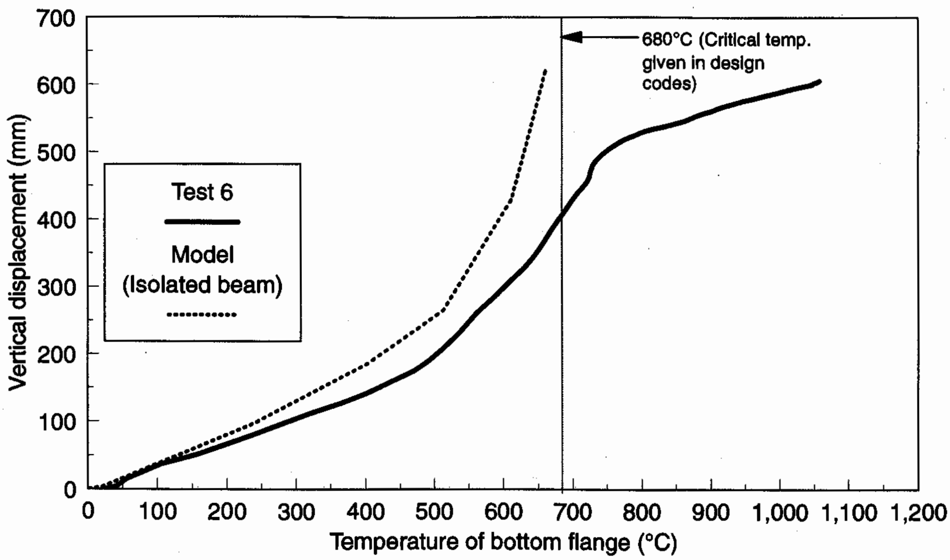


Figure 7.5 *Comparison between beam in Test 6 and beam modelled as an isolated member*

8 INTRODUCTION TO NUMERICAL ANALYSIS

Experimental testing of structures subject to thermal loading is extremely expensive to carry out. In other branches of structural analysis and design, cost benefits have been demonstrated in adopting the dual approach of numerical modelling with selective full scale testing. The full scale tests are used to examine the fundamental structural behaviour so that numerical models can be developed and calibrated. Once the numerical models are fully calibrated, they can be used to generate further data thus extending the database of available results.

Numerical modelling and experimental testing of steel structures subject to thermal loading has tended to concentrate on the behaviour of individual structural elements. However, since the Broadgate fire, it has been recognised that there are inherent benefits in considering the behaviour of steel framed building in fire as a whole rather than as a collection of individual elements. Therefore, attention has now switched to the behaviour of structural elements in frameworks.

The move to modelling the complete building structural behaviour in fire, particularly composite behaviour, is not straightforward as there are different problems to be tackled to those found in modelling individual element behaviour. Some of the problems that need to be overcome to successfully model the behaviour of composite steel framed buildings in fire are now discussed.

8.1 Composite Steel Framed Building Behaviour Subject to Thermal Loading.

Composite steel framed building behaviour under thermal loading can be addressed on two levels. These are the local behaviour of the individual structural elements within the building, and the global behaviour of the overall building. Obviously, the effect of the local structural element behaviour affects the global behaviour, but the distinction is important as the failure of an individual element does not necessarily lead to failure of the whole structure.

The local structural element behaviour in fire is governed by three main factors. These are the temperature distribution in the structural element, the load carried by the structural element and the degree of restraint offered by the surrounding structure. A rise in temperature of a structural element will lead both to expansion of the element and degradation in material properties of that element. If the structural element is significantly restrained by the rest of the structure then the expansion will lead to an increase in the stresses in the element. This increase in stress along with the degradation of material properties can lead to failure.

If an individual structural element fails then the load carried by that element will need to be redistributed to the surrounding structure. The extent of the redistribution will depend on the temperature of the surrounding structural elements and the degree of redundancy in those elements.

8.2 Numerical Modelling Requirements

The most important aspect to be modelled by any developed numerical model is the amount of redistribution that can take place in the composite structure before possible structural collapse. This means that the developed numerical model must be able to simulate the correct load path through the structure and include the various modes of failure of the individual structural elements. This behaviour has to be simulated throughout the thermal loading phase and the unloading, cooling, phase.

In addition to the technical requirements of any numerical model, there are certain practical limitations to be placed on the development. These include the ease of use and the hardware requirements. For ease of use, the developed model ideally needs to be set up using a graphical user interface. To enable hardware requirements to be kept to a minimum, the number of nodes and elements, the element types used to represent the structural behaviour and the solution procedures used need to be as computationally efficient as possible. The element types used will be governed by the modes of failure that need to be modelled for each structural element. The solution procedures will be governed by the failure mode of the individual elements and the overall structure.

The main elements to be modelled are columns, beams and floor slabs. The connections between these main structural elements will also be important to some degree. It may, however, be difficult and the matter of some judgement to separate the behaviour of the connection from that of main structural element.

Failure of the main structural elements can take place in a variety of modes. This depends upon the loading in the element, the type of the element and the restraint conditions applied to the element. It will be important to set up any model so that the correct conditions are obtained in each structural element throughout the analysis. This is not as straightforward as might first appear due to the complex interaction that can take place when structural elements are acting compositely.

8.2.1 Column Behaviour

The behaviour of columns in fire is relatively simple to model compared to the behaviour of the composite beams and floor slab. Columns are usually only restrained at their ends. The main possible column failure modes are squashing, overall buckling, bending and lateral torsional buckling. It is relatively straight forward to follow the stress pattern due to static and thermal loading throughout the element using beam-column theory. Therefore, by following the spread of yield across the cross-section, it is possible to predict all of these failure modes using one dimensional beam elements.

However, it may also be possible to get a local buckling failure of a column in extreme circumstances. This failure mode cannot be modelled using existing beam-column theory and would entail modelling the cross-sectional behaviour of the column using shell elements for the flanges and web of the column. This is computationally very expensive if a large extent of the model is thermally loaded.

8.2.2 Beam and Slab Behaviour

The behaviour of the beam and floor slab has to be considered together due to the composite interaction between the elements and the dual function of the floor slab. The dual function of the floor slab is to enhance the load carrying capacity of the bare steel beam and to act as the main structural element that carries the floor loading back to the surrounding structure. This raises fundamental questions as to the modelling approach that needs to be adopted.

The main failure mode of the composite beam that will need to be predicted is bending failure. This can be modelled using simple beam-column theory. Overall buckling modes are not so important as these are restrained by the composite action of the floor slab. However, local buckling may be important, particularly in hogging moment regions of the composite beam. To predict this localised behaviour the beam may need to be modelled using shell elements for the flanges and webs.

The modelling of the floor slab needs to concentrate initially on load-carrying capacity but ultimately the prediction of overall failure will be important. Initially, floor slab behaviour will be governed by the flexural stiffness in the main load carrying direction. As the steel beams supporting the floor slab increase in temperature and lose stiffness, the floor slab has to span effectively over a longer distance. This leads to loss of flexural stiffness and the membrane stiffness of the slab becomes more important. Additionally, traditional composite slabs on profiled steel decking have different flexural stiffnesses in orthogonal directions.

The need to model both the flexural and membrane stiffness of the floor slab means that the behaviour needs to be modelled using shell elements. This leads to difficulties in numerical modelling due to the lack of knowledge of modelling non-linear anisotropic shells. These difficulties are compounded by the limited concrete models available for modelling reinforced concrete in tension, making floor slab behaviour difficult and computationally very expensive to model.

8.2.3 Connection Behaviour

Beam to column connections and beam to floor slab connection (via shear connectors) are the main connections that affect composite frame behaviour under thermal loading.

The degree of shear interaction throughout the thermal loading is perhaps the most difficult to quantify. This may be dealt with by considering upper and lower bound solutions, i.e. by considering that there is either full or no shear interaction between the beam and the floor slab throughout the thermal loading.

It is considered that the beam to column connections will have the greater influence on behaviour than beam to beam connections. This, of course, will be strongly dependent upon the type of connection adopted. Consideration has to be given to the axial restraint supplied to the end of the beam as well as the (more normal) degree of rotational restraint. This behaviour can be modelled by using separate nodes for the beam and column connected by spring elements.

8.3 Numerical Modelling Study

Important requirements of a numerical model to study composite buildings under thermal loading have been outlined above. Some of these are computationally onerous and, despite the ever-increasing power of modern computers, need to be examined to ascertain the most computationally efficient method of predicting the structural behaviour. Therefore, as part of the Cardington project, a numerical study was undertaken to ascertain the influence of the various factors on overall structural behaviour.

The study consisted of a comparison of existing numerical models against the experimental data and preliminary modelling to ascertain the areas in which the numerical models would need to be developed to predict the observed experimental behaviour. The study has been carried out by British Steel Swinden

Technology Centre (UK), TNO (NL), CTICM (F) and Sheffield University (UK). The study employed a number of modelling approaches to ascertain the influence of the most important factors on structural behaviour in fire.

In addition to this modelling study, work was also carried out to investigate the role of tensile membrane action as a means by which the floor can resist the applied loads.

8.3.1 Comparison of Test Results with Existing Models.

Existing models were generally developed to predict isolated element behaviour. Therefore most of the development work on existing models has taken place on 2-dimensional (planar) models. These models can take into account the effects of temperature distribution through the elements, composite behaviour including the effect of partial shear connection and the effect of different connection types between planar elements (rigid, pinned and semi-rigid). They are invariably based upon planar beam elements founded on simple beam-column theory. Examples of such programs are SISMEF and CEFICOSS. All these models have previously been found to satisfactorily predict the behaviour of single element tests.

An example of the results obtained from a typical 2-dimensional program are given in Figures 8.1 to 8.3 for the plane frame test (test 2). These results were produced with the program SISMEF . This code is capable of simulating the behaviour of steel, concrete and composite planar frames exposed to fire, includes different types of joints (rigid, simple and semi-rigid) and utilises beam elements. It also includes a special interface element to model the partial shear interaction between the concrete slab and the beam. Figures 8.1 and 8.2 show the comparison with the vertical displacements obtained in the test and the calculated behaviour assuming simple and rigid connections. It can be seen that for both beam types in the plane frame test the observed behaviour lies in between the two cases. However, the calculated horizontal displacements (Figure 8.3) are over predicted. Displacements in the test are less than in the model due to the in-plane restraint provided by the concrete slab. This effect cannot be modelled in 2-dimensions.

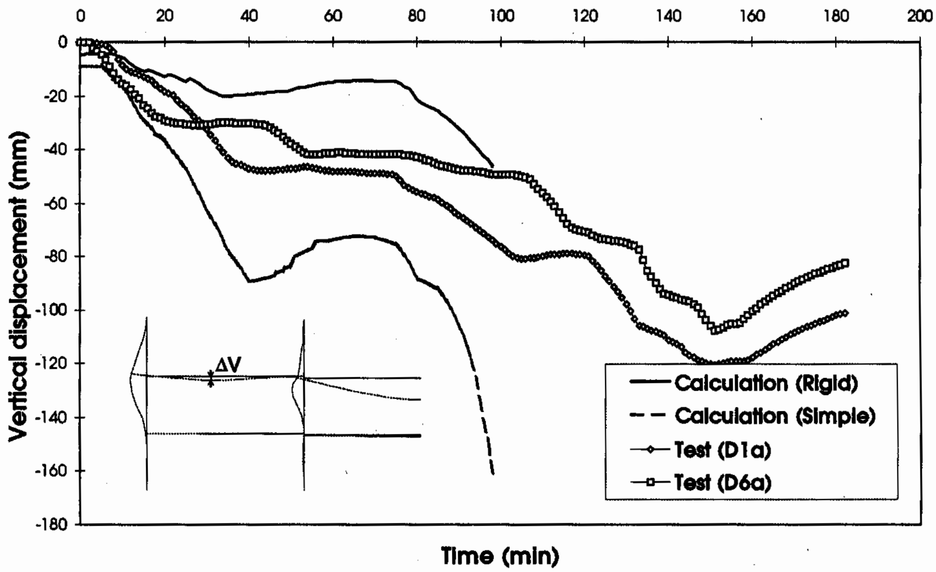


Figure 8.1 *Comparison of vertical displacement curves of experiment and calculations using 2-D programme SISMEF - 6 m span primary beams - Test 2*

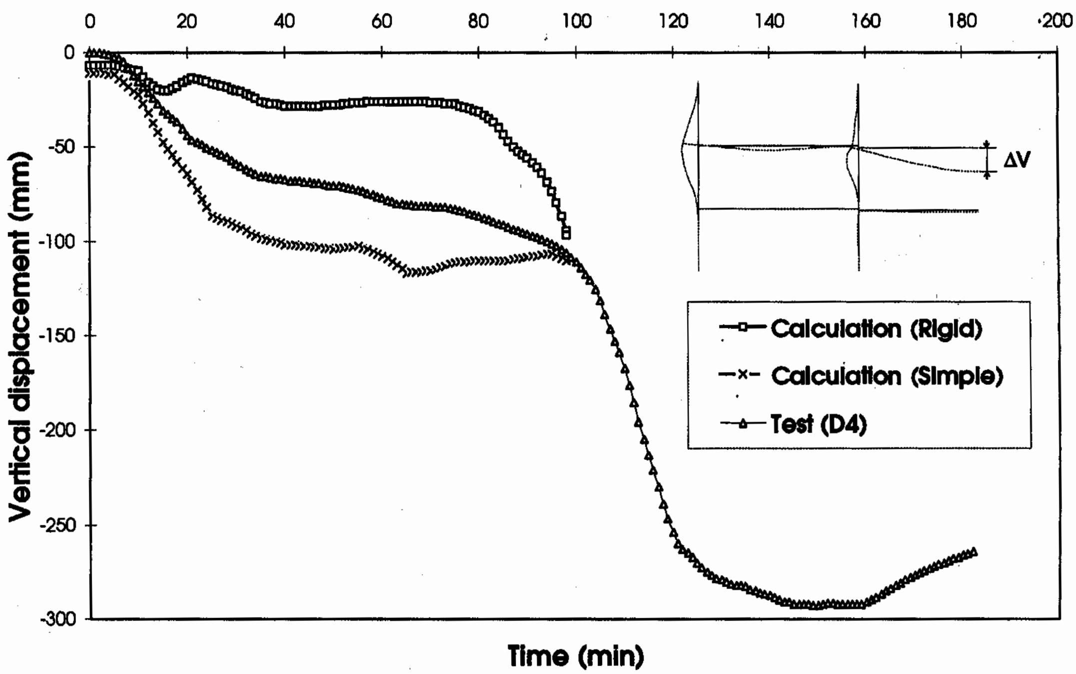


Figure 8.2 *Comparison of vertical displacement curves of experiment and calculations using 2-D program SISMEF - 9m span primary beams - Test 2*

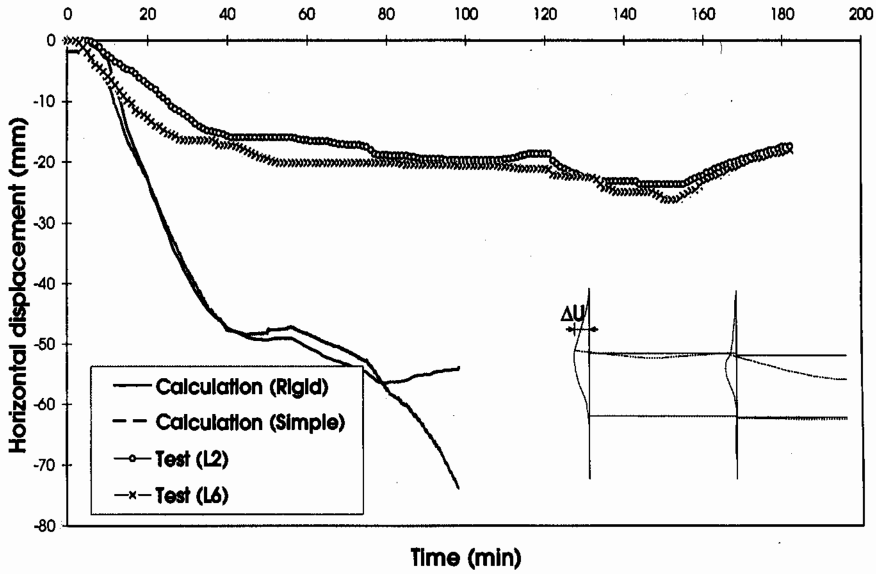


Figure 8.3 *Comparison of vertical displacement curves of experiment and calculations using 2-D program SISMEF - Test 2*

It was concluded that 3-dimensional models would be needed if the true behaviour of frameworks in fire was to be established. The move to 3-dimensional models for beams and columns is straightforward in that the planar type elements are simply converted to take into account the change in stress across as well as through the section based upon 3-dimensional beam-column theory. However, slab behaviour is more problematic as it necessitates a move to 3-dimensional shell elements. Therefore, most 3-dimensional models developed for fire at the beginning of the Cardington project tended to be based upon beam-column elements which were only suitable for looking at the bare steel framework behaviour.

An example of the results obtained for the 3-dimensional bare steel program, LENAS is given in Figure 8.4 for the first corner test (test 3). It can be seen that firstly the initial rate of vertical displacement is under predicted in the calculation but once 'runaway' in the model is reached the rate of vertical displacement is over predicted. Both observations can be explained by the absence of the concrete slab in the model. Firstly, the initial rate of vertical displacement is under predicted due to the thermal bowing of the slab being neglected and, secondly, 'runaway' is predicted in the model due to the membrane action in the continuous slab being neglected.

It was concluded, therefore, early in the numerical modelling study that existing 3-dimensional models would need to be developed further if the behaviour of composite structures in fire was to be predicted. A preliminary modelling study was undertaken to determine the areas for development.

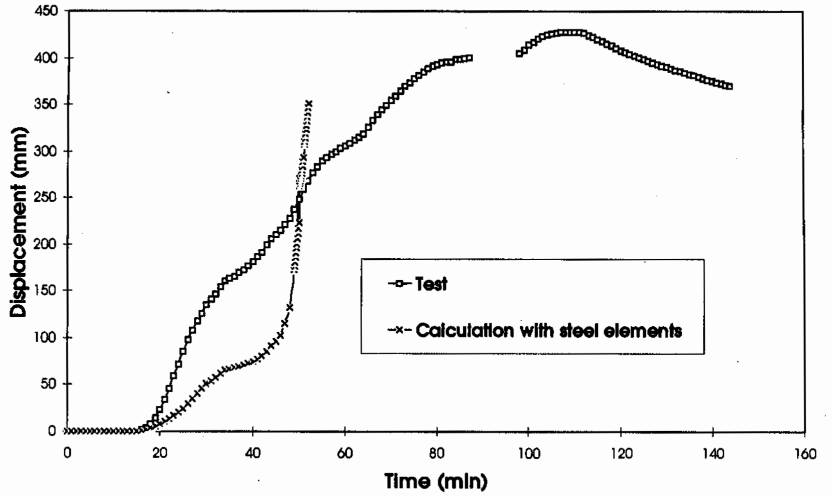


Figure 8.4 *Comparison of vertical displacement curves of experiment and calculations using 3-D program LENAS (steelwork only) - 9 m secondary beam - Test 3*

8.3.2 Preliminary Modelling Study

With the aid of the test observations, it was decided to study the effect of the following on the observed test behaviour:-

Concrete slab continuity
Beam local buckling
Beam-to-column/beam connection behaviour
Assumed boundary conditions/ extent of frame to be modelled

Three different modelling approaches have been adopted in examining the differences between developed numerical models and the observed test behaviour. Firstly, some of the important structural effects have been approximated using existing numerical models. Secondly, existing purpose written numerical models have been further developed to include some of the important modelling effects. Thirdly, general purpose finite element modelling packages have been used to examine the more complex structural effects.

Concrete slab continuity
As outlined previously, one of the problems with slab modelling is obtaining the correct nonlinear anisotropic behaviour. If it can be assumed that the slabs are essentially one way spanning then the out-of-plane effect of the slab can be modelled as a series of beam elements running perpendicular to the main steel elements. In-plane composite properties can be modelled with a beam element of appropriate effective width above the steel element.

An example of this assumption using the first modelling approach is given using the 3-dimensional code, LENAS, modelling the first corner test (Test 3). There is no concrete material model in this code so the concrete properties in tension and compression are assigned to the slab elements by considering whether the slab element is in a hogging or sagging region. An example of the results obtained for the first corner test is given in Figure 8.5. It is clear that much better agreement is obtained than for the bare steel structure (Figure 8.4). However, the best result is obtained for the assumption of simple joints between the steel to the concrete elements and this assumption is difficult to justify from an engineering viewpoint.

Additionally, comparison of displacements obtained across the test compartment for both the first corner test and the demonstration test (Test 6) show erratic agreement with primary beam displacements tending to be underestimated and secondary beam displacements being over predicted. Over prediction of the secondary edge beam displacements in Test 6 is due to the omission of the windposts in the model. However, the overriding reason for erratic prediction is probably due to the slab elements being decoupled and due to the simplifying assumptions regarding concrete modelling.

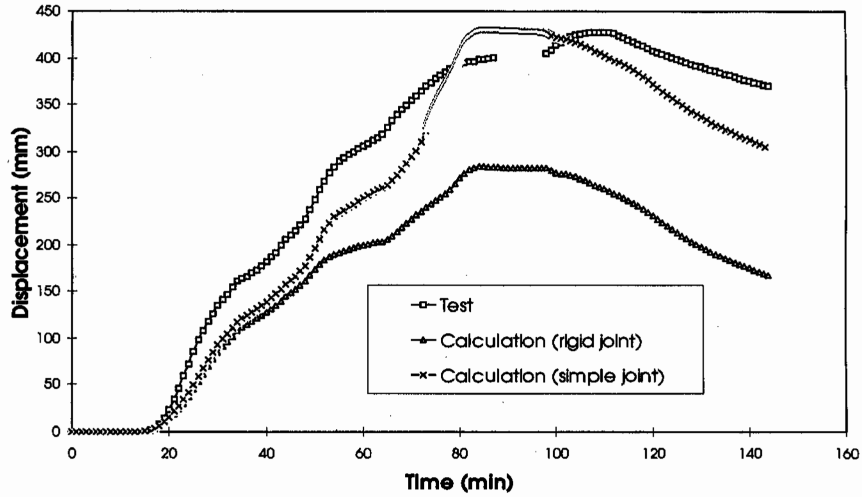


Figure 8.5 *Comparison of vertical displacement curves of experiment and calculations using 3-D program LENAS (slab representation included) - 9 m span secondary beams - Test 3*

An example of the second modelling approach is given by the purpose written program VULCAN. This approach utilises isotropic shell elements to model the concrete slab behaviour. Initial studies were carried out using a crude representation of the concrete tensile cracking which was governed by bending stresses only. However, the model has been shown to illustrate good correlation with the observed behaviour in some instances. Both the concrete model and slab element modelling have been improved during the course of the study with the development of a layered slab element capable of modelling anisotropic slab behaviour. Examples of the displacements obtained with both modelling approaches are given in Figure 8.6, again against the first corner test for comparison purposes.

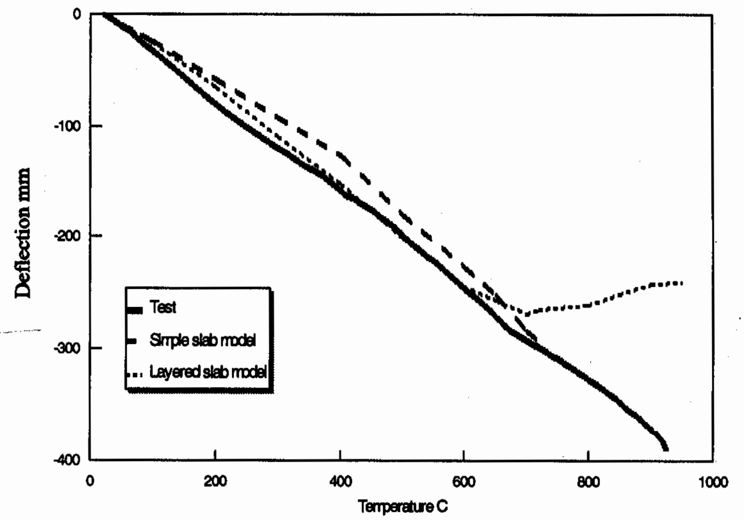


Figure 8.6 *Comparison of temperature vertical displacement curves of experiment and calculations using 3-D program VULCAN using different slab representations - 9 m secondary beam - Test 3*

An example of the third modelling approach is given by the modelling of the first corner test using the general purpose finite element program, DIANA. This approach uses 8-noded shell elements with the ribs of the concrete slab being modelled by means of beam elements underneath the shell elements. The material model considers concrete cracking and assumes that the slab behaviour is dominated by tensile cracking of the concrete. Both static secant stiffness and dynamic solution procedures have been used to obtain good agreement across the first 3 tests. An example of the agreement obtained in the first corner test (Test 3) is given for the secondary beam in Figure 8.7. This modelling procedure is the most rigorous approach to concrete slab modelling and gives the best overall agreement in the early stages of the tests. However, it is computationally very onerous especially when the dynamic procedure has to be used for the more extensive tests. Research to find the most accurate but computationally efficient global solution to concrete slab degradation behaviour in fire is currently proceeding at several research organisations.

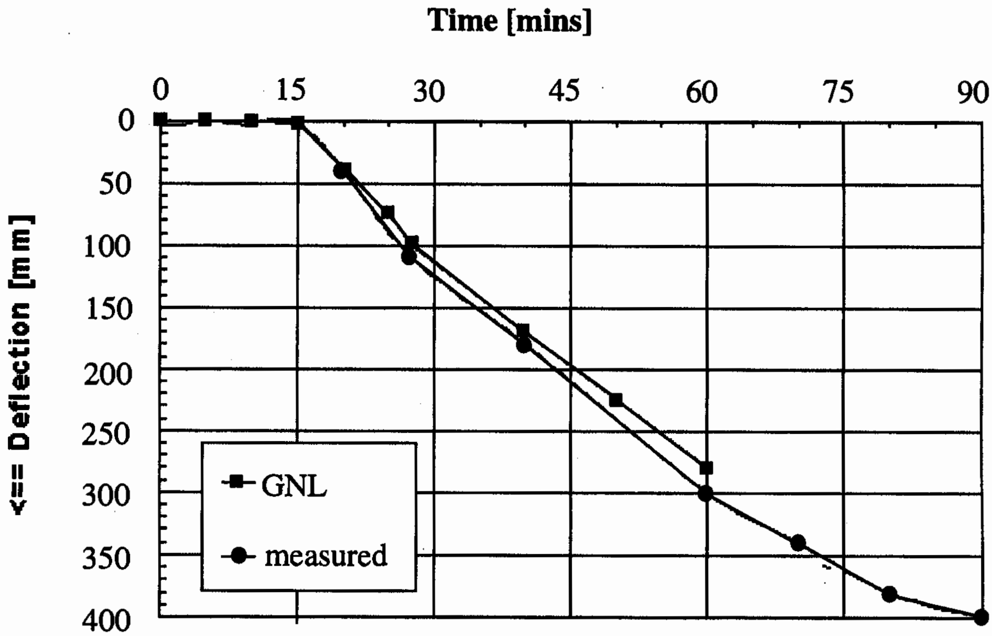


Figure 8.7 *Comparison of vertical displacement curves of experiment and calculations using 3-D element program DIANA using shell slab representation with concrete cracking - 9 m secondary beam -Test3*

Beam local buckling

Local buckling of the lower flange of the steel beams was observed in nearly all of the Cardington tests. Most of this buckling occurred in the connection areas especially in connections between beams and columns. However, it may be possible that the buckling observed is due to a localised squash failure once the beam has reached its limiting temperature locally or indeed that the buckling may be of limited structural significance. All the results presented in the section on concrete slab continuity use beam elements and show reasonable agreement with the experimental results especially for secondary beams. However, it is important to understand the relative contributions of different factors and it may be possible that underestimating the stiffness of the slab could be offset by overestimating the stiffness of the steel beam. This is important if parametric studies are to be carried out.

Beam elements cannot predict local buckling as their formulation assumes that the beam retains its I-shape at each cross-section. If local buckling is to be predicted then, as mentioned in the earlier section, the web and bottom flange of the beam need to be modelled using shell elements. An example of such is an analysis using ANSYS to represent the bare steel beam in the restrained beam test (Test 1). The vertical deflection of the ANSYS (shell formulation) model is given in comparison with a SISMEF (beam formulation) model in Figure 8.8. The concrete slab is not modelled in each case which accounts for the much greater deflections in comparison with the test result. The difference in deflection between the two analyses would suggest that local buckling is not important in this case. However, buckling was only picked up just inside the test furnace suggesting that this is a material type failure at the limiting temperature of the beam.

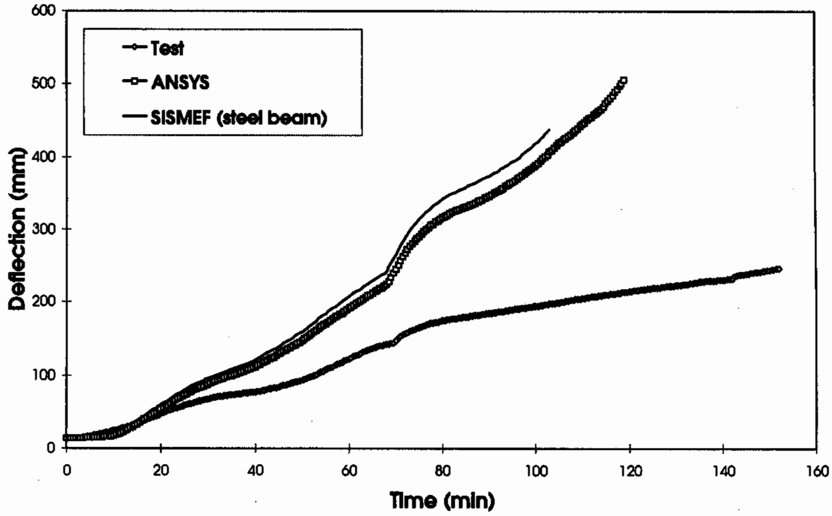


Figure 8.8 *Comparison of vertical displacement curves of experiment and calculations using 3-D finite element program ANSYS (shell model) and 2-D program SISMEF (beam model) - 9 m secondary beam - Test 1*

Further work has been carried out using the general purpose finite element code, ABAQUS. This work has studied the influence of factors such as connection behaviour and degree of restraint on the occurrence of local buckling. The work has mainly concentrated on the restrained beam test. A comparison of the experimentally obtained temperature-deflection curve with those obtained from shell and beam models with different degrees of restraint is given in Figure 8.9. These results suggest that local buckling does have an influence on overall beam behaviour. However, the influence is predictable and should be able to be included in beam models by the use of appropriate axial springs. This work is currently undergoing development.

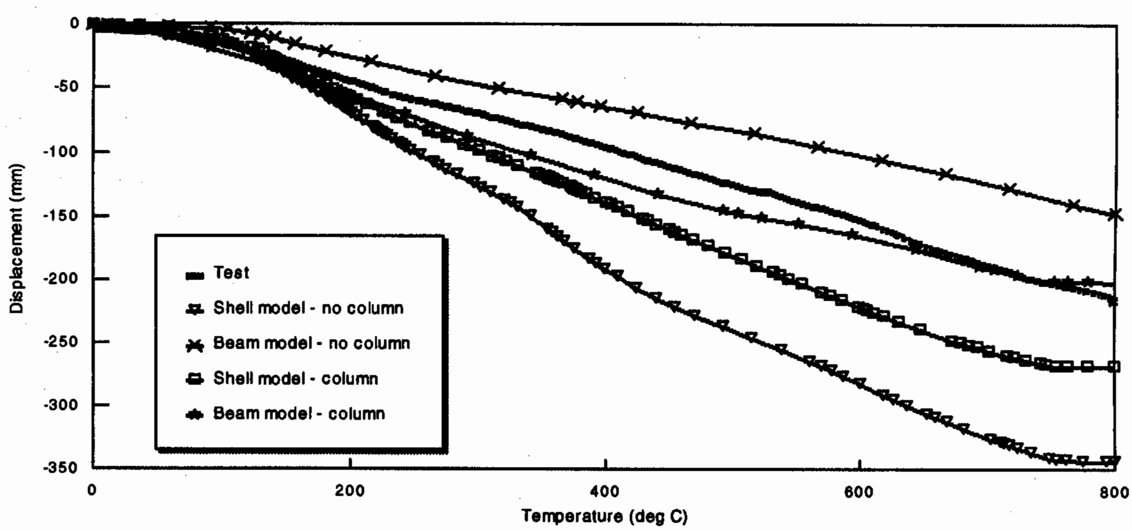


Figure 8.9 *Vertical displacement curves of experiment and calculations using 3-D finite element program ABAQUS (shell and beam models) using different degrees of restraint - 9 m secondary beam - Test 1*

Beam-to-column/beam behaviour

The possible influence of connection rotational behaviour on overall structural performance in fire has been referred to in section 8.3.1. These figures suggest that the degree of connection rotational restraint has a significant influence. However, research carried out using the program VULCAN suggests that connection rotational performance has less of an effect, Figure 8.10. Additionally, both these approaches use beam elements and as such assume that the end of the beam remains plane in bending.

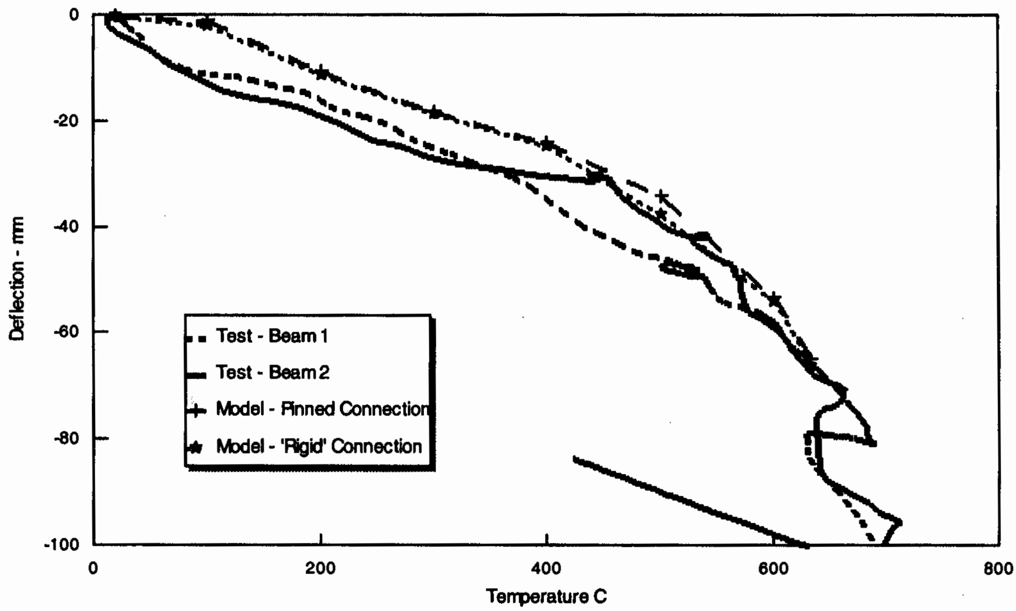


Figure 8.10 *Comparison of vertical displacement curves of experiment and calculations using 3-D program VULCAN (beam models) using different degrees of rotational restraint - 6 m primary beam - Test 2*

Work to study the possible influence of the connection behaviour on beam local buckling has been carried out using shell models for the beam in the general purpose finite element code, ABAQUS. The study has looked in some detail at the degree of axial restraint provided and the effect of the partial depth end plate supporting only the web of the section. The connection is represented as a series of springs between the column and beam web in tension and by a series of gap elements of zero gap representing the connection in compression. The gaps between the beam flanges and the column are represented as a gap element with an initial gap equal to the thickness of the partial depth end plate. This is illustrated schematically in Figure 8.11. Comparisons of the effect of including this behaviour are given in Figure 8.12 for the restrained beam test. Three different events attributable to connection behaviour can be discerned from these analyses. Firstly, the restraint to beam web expansion provided by the partial depth end plate leads to web buckling at a bottom flange temperature around 75°C. This gives rise to increasing rotation until the bottom flange of the beam contacts the column at a bottom flange temperature around 150°C. This gives rise to increasing bottom flange stresses and a local buckling failure of the bottom flange at around 250°C. As temperature continues to increase a final 'buckle' develops just inside the heated portion of the test furnace. This result is in complete agreement with the test observations and clearly suggests that the connection behaviour does have an influence on local buckling behaviour at the end of the beam. Work is continuing into ways of including this effect into simplified beam models.

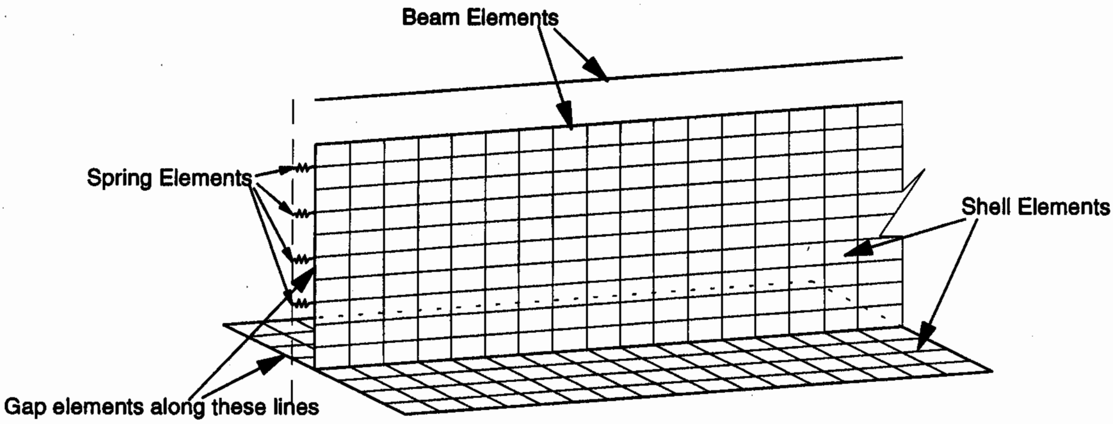


Figure 8.11 *Schematic representation of connection modelling to ABAQUS shell model*

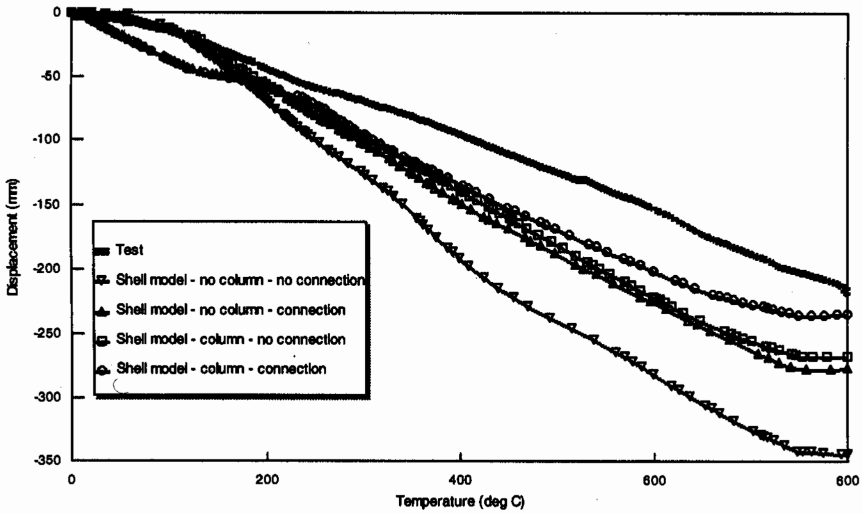


Figure 8.12 *Comparison of vertical displacement curves of experiment and calculations using 3-D finite element program ABAQUS (shell model showing effect of connection modelling and different degrees of restraint - 9 m secondary beam - Test 1*

Assumed boundary conditions

The influence of the degree of restraint provided to the restrained beam in Test 1 has been demonstrated by the above analyses. A study using VULCAN to investigate the extent of the frame to be modelled was carried out for the restrained beam test. Comparison of the temperature deflection curves for different models (Figure 8.13) illustrates that greater thought needs to be given to the boundary conditions incorporated within different models. These results need to be extended to shell representations of the beam in order to determine the influence of the assumed restraint on beam local buckling.

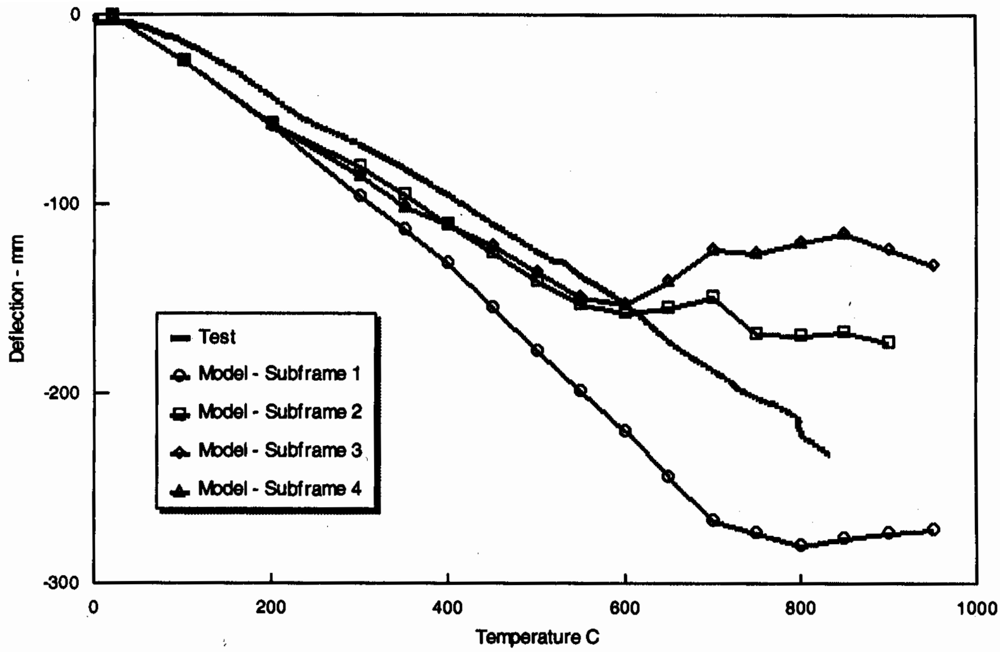


Figure 8.13 *Comparison of vertical displacement curves of experiment and calculations using 3-D VULCAN (beam models) using different sized subframes - 9 m secondary beam - Test 1*

8.3.3 Conclusions

Clearly all the factors studied have some degree of influence on the overall frame behaviour, often in a complex interactive manner. Different modelling approaches have shown that all the important effects can be successfully included to predict the behaviour of steel frameworks in fire. Effort is now being focussed on the most computationally efficient method of including the important structural effects. Once this has been done, parametric studies of different structural arrangements under different fire scenarios can be carried out so that detailed design guidance can be formulated.

8.4 Tensile Membrane Action

Membrane action in the composite slab becomes more significant as both the slab and supporting steel beams rise in temperature and lose their flexural stiffness and strength.

Depending on the boundary restraint conditions of the slab, it is possible for either compressive membrane action (dome effect) or tensile membrane (catenary action) to develop. Both types of membrane action develop after the formation of yield lines in the slab and result in the slab having a larger load carrying capacity compared to the theoretical pure flexural strength, which can be calculated using standard yield line theory.

Compressive membrane action relies on the in-plane restraint to the slab at its boundaries and can therefore only develop in slabs with fixed boundary conditions. It occurs immediately after the formation of yield lines at moderate slab deflections and is very sensitive to the amount of in-plane restraint to the slab.

Tensile membrane action can develop in slabs with either fixed or simply supported boundaries. In a slab with fixed boundary conditions, tensile membrane action develops, following compressive membrane action, at very large deflections. The tensile force developed in the reinforcement is anchored to the slab supports. In a slab with simply supported boundary conditions, the supports cannot anchor the tensile forces in the reinforcement. Instead, the slab develops a compressive stiff ring beam around its edges which support the internal tensile forces. With the development of tensile membrane action the ultimate load carrying capacity of the slab is reached when the reinforcement fractures.

It is possible that tensile membrane action occurred in a number of the fire tests on the Cardington frame. However, it was difficult to identify the formation of any yield line pattens which should, theoretically, occur before the development of tensile membrane action. To identify the importance of membrane action BRE are conducting an experimental study on the behaviour of composite slabs constructed similar to those used in the Cardington frame. The purpose of the investigation is to study the mechanism of tensile membrane action and to quantify the ultimate collapse strength for this type of composite slab.

9 DISCUSSION

9.1 Observed Behaviour

The Cardington fire tests were carried out because standard fire tests, although excellent at comparing the performance of different structural components, are not a realistic representation of real fires or the actual structural behaviour of buildings.

The tests on the Cardington frame enabled the differences between the standard fire test and actual behaviour to be studied. This led to the conclusion that the interaction with the cooler structure surrounding the heat affected members within the fire compartment, was extremely beneficial to the heated members. The maximum steel temperature reached during the six fire tests at Cardington was in excess of 1100°C. This occurred with no signs of structural collapse. Using modern fire codes, which are based on the standard fire tests, failure (or structural collapse) was calculated to occur at a critical temperature of approximately 680°C; it is clear that the current level of safety is extremely high. Future work will concentrate on quantifying the safety level and developing definitive design guidance, which will incorporate a more logical and economical approach to structural fire design.

The main structural observations from the fire tests on the Cardington building are discussed below:

9.1.1 Steel Columns

In Test 2, the heated columns were fire protected over most of their length, with the passive protection being stopped 200mm below the deepest connecting beam (Figure 9.1). During the test, the exposed part of the steel column failed in compression when it reached a steel temperature of approximately 670°C. The shape of the squashed column was governed by the bottom flange of the connected beams (which were of differing depths) bearing onto the column flange, as the curvature of the beams increased (Figure 9.2). Although this behaviour did not result in overall structural failure, the local compression deformation of the column did cause damage to the floors above the fire compartment. It was decided to protected the columns over their full length (including the connections) in all the remaining tests and therefore avoid local failure of the columns.

Consideration of Figure 9.1 raises the question of to what extent should the column length be protected. However, it can be argued that the passive protection only needs to be extended to the underside of the smallest connecting beam. The temperature of the exposed part of the column will be fairly low in relation to other exposed parts of the structure due to the mass of steel in this area. In addition, the restraint provided by all the connecting beams will be beneficial to the squashing resistance of the exposed part of the column. Future research, in terms of small-scale tests may address the extent to which passive protection needs to be applied to the column. Until this research is conducted, it is recommended that columns in multi-storey buildings of more than two storeys, should be fire protected over their full length. Alternatively, the columns could be designed to achieve the required fire resistance, e.g. concrete filled hollow steel sections.

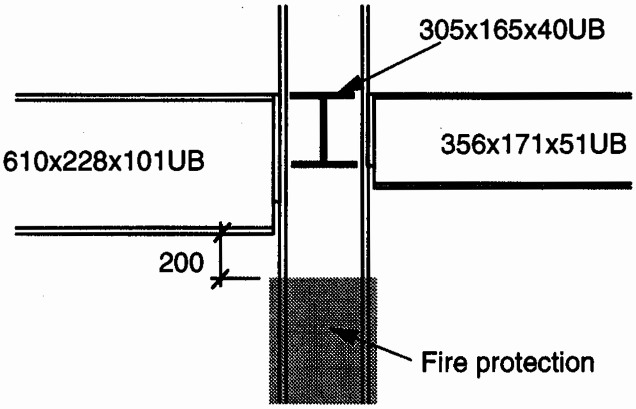


Figure 9.1 *Extent of column fire protection in Test 2*

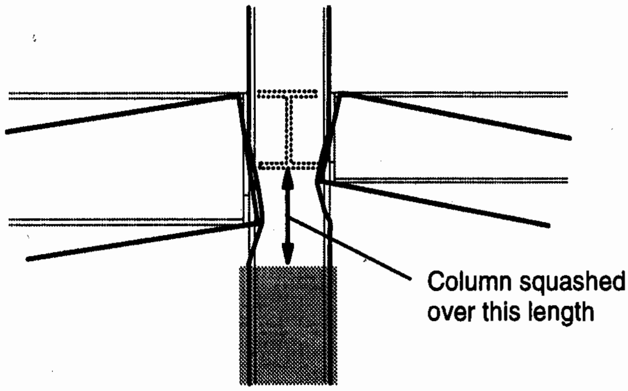


Figure 9.2 *Part of exposed column which squashed during Test 2*

9.1.2 Composite Beams

The steel beams act compositely with the concrete slab in normal and fire conditions. The internal beams and many of the perimeter steel beams were unprotected in all of the tests, and no signs of structural collapse was evident. The maximum recorded internal steel temperature was over 1100°C.

Internal beams
Many internal beams showed signs of local buckling in the lower flange and part of the web in the proximity of the connections. This was caused by the restraint to thermal expansion from the much cooler structure surrounding the fire compartment, and was accentuated by the negative moment caused by thermally induced curvature and connection restraint. Due to local buckling, it is difficult to quantify the transfer of moment from the beam into the connections. Therefore, at present, it is advisable to design unprotected beams in multi-storey buildings as simply-supported in the fire condition. However, it is apparent that some other beneficial mechanisms must have come into play in order that the fire performance of the beams was so much improved. It is suggested that this was due to catenary action of the beams at large deformations. In this case, the connections act in tension, rather than bending.

Perimeter beams
In Test 3 the perimeter beams were protected whereas in tests 4,5 and 6, they were left totally exposed. In the case of the unprotected perimeter beams, the attached windposts acting in tension above the fire compartment were beneficial, resulting in very small vertical displacements of these beams. Therefore, if used, the windposts may be included in the fire design to justify the use of unprotected perimeter beams. In other cases, it is probably necessary to fire protect edge beams.

9.1.3 Connections

Apart from Test 2, all beam-to-column connections were protected as a consequence of the columns being protected over their full height. During the heating phase of the fire, both the end-plate and fin-plate connections performed adequately (including those in Test 2) with structural integrity being maintained. During the cooling phase some of the end-plate connections fractured down one side of the beam, as shown in Figure 9.3. In the fin-plate sections a number of bolts sheared. This type of behaviour in the connections was attributed to the high tensile forces induced during the cooling phase of the fire. This was caused by the steel beam cooling down (over many hours) from a plastic state (typically recognised by the presence of local buckling). This tensile strain on cooling was relieved by the plate fracture or bolt shear of the connection.

In all instances shear capacity was maintained, either by the residual strength of the connection or from the shear strength of the slab. The observed behaviour of the connections during the cooling phase of the fire, suggests that it is desirable to design connections to have large ductility so that shear capacity is maintained when subjected to high tensile forces. End plate connections act more reliably than fin plates in this situation.

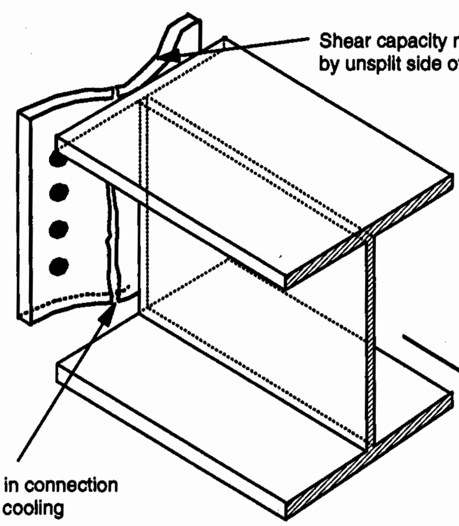


Figure 9.3 *Fracture of end-plate during the cooling phase of the fire*

9.1.4 Composite Floor Slab

The composite floor slab performed very well during all of the tests. This confirmed the observations from previous small-scale fire tests and from experience of real fires in buildings, which have all shown that floors of this type of construction have good inherent fire resistance.

The bridging/membrane characteristics of the composite slab had a beneficial effect on the structural performance of the unprotected beams during the tests, when the beams had lost a large proportion of their bending resistance. In the first instance the slab bridges across the weakening heated steel members by utilising its full moment capacity. As the displacements increase further, the slab then acts as a tensile membrane through the mesh reinforcement. Both types of behaviour are shown diagrammatically in Figure 9.4. In the case of tensile membrane action, if the supports have no horizontal restraint (i.e. the slab is at an edge of a building) then a compressive membrane ring will eventually form around the area of slab in the cooler part of the compartment, as shown in Figure 9.5.

To include the beneficial effects of the load carrying capacity of the floor slab in structural fire design, its membrane action needs to be fully understood and the span (or size of fire compartment) limits defined. This task is being addressed at present by a number of research organisations.

This action only occurs when a local area of the slab is heated and there is sufficient restraining effect around the perimeter of the heated area. It is not clear how large the heated area can be in order that this action is effective in supporting the beams to well beyond their critical temperature. The slab above protected beams is effective in resisting compression.

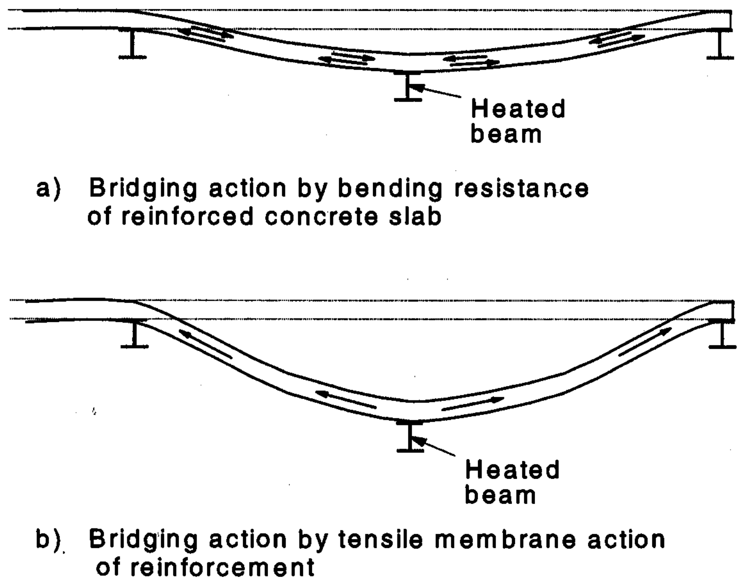


Figure 9.4 *Bridging across a heated beam by the slab acting in bending and membrane action*

Although extensive cracking was observed in the concrete above the fire compartment, its integrity as a compartment floor was generally maintained during the heating phase of the fire. In Test 6, large cracks occurred around the internal column. Investigation of this area showed that the mesh reinforcement was not lapped correctly, and was just 'butted' together. Therefore, it is recommended that, to ensure integrity of the compartment, and for the capability of utilising tensile membrane action, care should be taken to ensure that the reinforcement is correctly lapped in accordance with normal specifications.

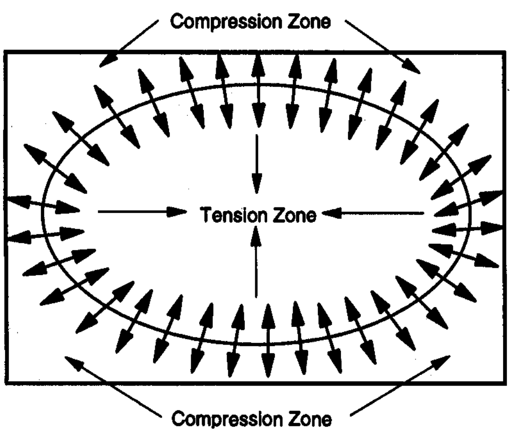


Figure 9.5 *Tensile membrane action of slab without horizontal restraint to edges*

9.1.5 Compartment Walls

In Tests 4 and 5, internal compartment walls were constructed using steel stud partitions with fire resistant board. In Test 4, the wall was placed under unprotected beams. Due to the shielding effect of the wall, the vertical deflection of these beams was very small and the integrity of the compartment wall was maintained. In Test 5, the compartment wall was placed 'off-grid' and the deflection of the slab caused integrity failure of the compartment wall.

Therefore it is advisable to place compartment walls under beams which, as shown in Test 4, leads to limited vertical deflection of these beams.

9.2 Analytical and Numerical Modelling

The results of the preliminary analytical modelling have been very encouraging. Different modelling approaches have shown that all the important effects can be successfully included to predict the behaviour of steel frameworks in fire. Work is continuing on the systematic assessment of different modelling approaches to ascertain computationally efficient methods of representing the important structural framework behaviour. This work will result in robust analytical models which can be used to carry out parametric studies of different structural and fire scenarios to extend the data base of results.

10 THE FUTURE

Following the fire test programme further research has commenced in order to develop a more detailed understanding of the observed behaviour and to develop design guidance. Two groups have been set up: the first, the "science" group has the task of building finite element models of the structure with the aim of understanding the various modes of behaviour and their relative importance. The second, the "design" group, has the role of developing design aids and disseminating the lessons from Cardington.

The development of design guidance will be in two phases. The first phase is to look carefully at what happened at Cardington and, based largely on observation, give advice and make limited recommendations. Many phenomena were observed in the Cardington tests which go some way to answering questions posed by industry and regulatory authorities. Guidance will be given on subjects such as thermal expansion, heat conduction through walls and floors, and on the design of compartment walls. Recommendations will also include reinforcement detailing and advice on the selection of connections. In the short term, advice will focus on the ability of composite secondary beams to resist fire attack (in the absence of passive protection) and on the fire protection criteria for columns.

The development of comprehensive design rules will necessarily follow the advanced finite element work being carried out by the "science" group. Some difficult problems have to be solved, interestingly, for the steel industry, many of these problems relate to modelling concrete behaviour in fire.

The ultimate aim of the research is to reduce unnecessary amounts of applied fire protection in steel framed buildings in order to improve efficiency, competitiveness and value for money to clients. A key aspect of this lies in quantifying the level of safety required and ensuring that it is met by means of reliable design techniques.

11 REFERENCES

1. STEEL CONSTRUCTION INDUSTRY FORUM
Structural fire engineering, Investigation of Broadgate Phase 8 fire
Steel Construction Institute, 1991

2. BRAND VERHALTEN VON STAHL UND STAHLVERBUND KONSTRUCTIONEN
(Fire behaviour of steel and composite construction)
Verlag TUV Rheinland, 1986.

3. BRITISH STANDARDS INSTITUTION
BS ENV 1994: Eurocode 4: Design of composite steel and concrete structures
Part 1.2 Structural fire design (including UK NAD)
BSI, 1998

4. BRITISH STANDARDS INSTITUTION
BS 5950: The Structural Use of Steelwork in Buildings
Part 1: Code of Practice for Design in Simple and Continuous Construction, BSI, 1990
Part 3: Design in Composite Construction, Section 3.1 Code of Practice for Design in Simple and Continuous Composite Beams
BSI, 1990

5. THE BUILDING REGULATIONS
(a) Approved Document B, Fire Safety
Department of the Environment and The Welsh Office, HMSO, 1991
(b) The Building Standards (Scotland) Regulations
Scottish Office, HMSO, 1990
(c) The Building Regulations (Northern Ireland)
Department of the Environment, HMSO, 1990

6. BRITISH STANDARDS INSTITUTION
BS ENV 1991: Eurocode 1: Basis of design and actions on structures
Part 1: Basis of design, and
Part 2.2: Actions on structures exposed to fire
BSI, 1994

7. INTERNATIONAL STANDARDS ORGANISATION
ISO 834: Fire resistance tests - elements of construction
ISO, 1985

8. PETTERSSON , MAGNUSSON, THOR
Fire engineering design of steel structures
Swedish Institute of Steel Construction
Stockholm, 1976

9. BRITISH STANDARDS INSTITUTION
BS DD 240: Code of practice on fire safety engineering
BSI, 1996

10. BRITISH STANDARDS INSTITUTION
BS 5950: The Structural Use of Steelwork in Buildings
Part 8: Code of practice for fire resistant design, BSI, 1990

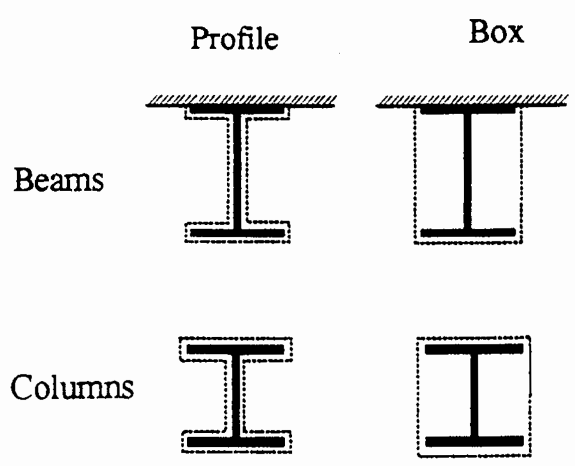
11. BRITISH STANDARDS INSTITUTION
BS ENV 1993: Eurocode 3: Design of steel structures
Part 1.2 Structural fire design (including UK NAD)
BSI, 1998

APPENDIX A Section Factors

Table A.1 *Section factors for steel sections used on the Cardington frame*

		Section Factor (m $^{-1}$)	
		Profile*	Box*
305x165x40 UB	9.0m internal secondary beam	210	150
356x171x51 UB	9.0 external beam and 6.0m primary beam	185	135
610x229x101 UB	9.0 primary beam	145	110
305x305x198 UC	ground to second floor columns	75	50
305x305x137 UC	second floor to fifth floor columns	105	70
254x254x89 UC	fifth floor to eigth floor columns	130	90

* *'Profile' and 'box' is defined for the Cardington building as shown;*



APPENDIX B Test Loading

Table B.1 *Characteristic design loads (excluding dead loads)*

Type of load	Design characteristic load (kN/m^2)	Load applied during fire tests (kN/m^2)
Raised floor	0.4	0.4
Services	0.25	0.25
Ceiling	0.15	0.15
Partitions	1	1
Imposed	2.5	0.83
Total	4.3	2.63

Table B.2. *Load Ratios for main beam sections*

Section size	Location	Load Ratio	Limiting temperature to BS5950 Part 8 (°C)
305x165x40 UB (grade S275)	9.0m secondary beam	0.4	680
356x171x51 UB (grade S355)	6.0m primary beam	0.39	685
610x229x101 UB) (grade S275)	9.0m primary beam	0.26	747

Note: The self-weight of the beams plus 2.06kN/m² for the self-weight of the composite slab was used in calculating the Load Ratios.