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An analysis of the global structural behaviour of the Cardington steel-framed building during the two BRE fire tests

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Abstract

This paper presents the results of an analysis of the global structural behaviour of the 8-storey steel framed building at Cardington during the two BRE large-scale fire tests. These two tests (the Corner test and the Large compartment test), together with the four fire tests conducted by British Steel, formed the core programme of a multi-million pound research project sponsored by the UK's Department of Environment and the European Coal and Steel Community. These tests were carried out to investigate the performance of whole building structures under realistic fire conditions and to provide quality experimental information for the validation of various numerical models. The results of these two BRE tests were analysed using a specialist finite element computer program (to be referred to as FIREFRAME in this paper) and this paper presents the main findings of this study. Due to the difference in their fire intensities, the two BRE tests resulted in markedly different structural behaviour of the steel frame. This gives rise to an opportunity to check the capability of the computer program. The results of this analysis seem to indicate that FIREFRAME is capable of simulating flexural bending behaviour. However, in order for the program to simulate the Cardington frame behaviour during the Corner test, it requires a more advanced numerical procedure to deal with slab tensile membrane behaviour at large deflections. Test and computer simulation results suggest that columns may attain large moments as a result of being pushed by the adjacent hot beams, but as the test column temperatures were low, it was not possible to assess the column failure behaviour. Furthermore, computer simulations indicate that large sagging moments may develop in heated beams during cooling, but further research is required to check whether this would lead to beams failure.

Keywords: Whole building behaviour; Cardington tests; Fire resistance

1. Background to the Cardington project

The fire safety design of steel structures has undergone significant changes in recent years. It has advanced from the prescriptive approach based on the somewhat arbitrary standard fire resistance of individual structural members under idealised loading and boundary conditions, to a more rational approach which takes account of the more realistic fire and structural behaviour. This is evidenced by the publications of BS 5950 Part 8 [1], Eurocode 3 Part 1.2 [2] and Eurocode 4 Part 1.2 [3].

Using these rational design codes can often lead to a reduction in the required fire protection thickness to steel members, however, in most cases, since the temperature in a bare steel section will be higher than its limiting temperature, fire protection to structural steel members is still necessary. Owing to the high labour cost and increased construction time, the application of any fire protection can incur a high steel structure construction cost, thus steel's full potential can only be achieved when the fire protection is completely eliminated. In this sense, although the recent intensive studies are intended to gain a better understanding of the fire performance of steel structures, there is a strong economic incentive behind using unprotected steelwork.

Broadly speaking, the following three approaches may be considered in order to achieve the objective of eliminating the fire protection to steel structures:

1. To control the design fire size. By properly understanding the behaviour of real fires and by taking into account the contributions of other fire protection measures, such as fire alarms, sprinklers and other fire fighting methods, the design fire may become very

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small and the resulting damage to the unprotected steel structural members minimal. This forms part of the so-called “Natural Fire Safety Concept” [4].
2. To integrate the structural loadbearing and fire protection functions. In the traditional way of design, building and then fire protection of steel structures, fire protection is generally regarded as an extra finish whose function is purely to provide fire insulation to the steelwork. Recent development work on steel structural members has seen the integration of structural loadbearing and fire protection functions of concrete, in different forms of innovative steel-concrete composite construction. The paper by Bailey and Newman [5] summarises a number of practical approaches to achieve the required fire resistance using these innovative products.
3. To utilise the whole building structural behaviour. Up to now, the design of steel structures for fire safety is generally based on the assessment of individual structural members, i.e. each structural member should achieve the required fire resistance. However, it should be realised that for a building structure to remain stable under fire conditions, serving the principal need of containing a fire and preventing its spread, it is not absolutely necessary for every individual structural member to remain stable. Fire protection may be eliminated in some steel members if, in the absence of these members, an alternative load path can be provided with other undamaged structural members. To benefit from the better fire performance of a complete building structure, it is necessary to gain a sound understanding of the whole structural fire behaviour.

This paper is related to the third approach. The good whole building behaviour under fire conditions has been recognised for some time. It was emphatically confirmed by a fire accident that occurred in an unprotected and partly completed 14-storey steel framed office block at the Broadgate development in London [6]. Although this fire accident could not provide much quantitative information about the whole building structural behaviour, it was influential in the decision to carry out full scale fire tests in complete steel framed building structures [7,8], and this led to the studies reported in this paper.

In total, six compartment fire tests were completed, two by the Building Research Establishment and four by British Steel. The completion of these tests is now followed by extensive analysis of the test results by various organisations using different computer programs to study the various aspects of the whole building fire performance [9–13]. These analytical studies are being carried out with two principal objectives: firstly, to check and improve the capability of the different numerical models, and secondly, to use the validated numerical models to conduct parametric studies to provide data for the development of improved design guidance. This paper presents the results of the author’s analysis of the global structural behaviour using the computer program FIREFRAME [9].

2. Brief description of the computer program

This program was developed by the author [9] and was used to carry out preliminary analysis to assess the instrumentation need for the BRE’s fire tests. In this program, the following assumptions have been adopted:

1. Cubic beam-column elements are used to model skeletal frame members. Each structural member may be divided into a number of such elements. Each element has two nodes and each node 3 degrees of freedom (DOF) for 2-dimensional analysis or 6 DOF for 3-dimensional analysis, representing deflections and rotations.
2. The structural behaviour of a slab is represented by an effective width in the composite beam. Only complete shear interaction between the steel and concrete components is allowed for. In addition, the contribution of the reinforcement mesh in the slab is ignored.
3. Second-order effects are included by employing a geometric stiffness matrix. However, the effect of very large deflections, leading to second order membrane strains comparable to first order bending strains, is not included. Therefore, large deflection-induced membrane actions cannot be dealt with in this program.
4. The assumption of planes remaining plane after deformation is adopted, thus the program cannot deal with local buckling or lateral torsional buckling.
5. Temperature distribution over each finite element cross-section may be non-uniform.
6. The execution of the computer program stops when the determinant of the stiffness matrix becomes non-positive. The appearance of a non-positive stiffness determinant may be caused by the local failure of an element. Therefore, this program cannot simulate progressive collapse. Nevertheless, it can deal with the structural interaction between different members before this local failure.

The predictions of this program have been checked against the results of a large number of tests on steel/composite structural members and on simple steel frame assemblies [9]. It is regarded to be able to simulate flexural bending behaviour.

The limitations of this program are enumerated here so that the reported analytical results may be used to assess the stage of the Cardington building structural behaviour, and to indicate the requirements for developing future computer programs.

3. Brief description of the Cardington building and BRE tests

The Cardington fire tests on the steel framed building have been well promoted. This section provides only a brief description of the main features of the structure and the two BRE fire tests.

3.1. The test building

Fig. 1 shows the steel building structure, which was built inside a former airship hanger located at Cardington, Bedfordshire, UK. The test building is a steel framed construction, using in-situ concrete slabs supported by the steel decking and in composite action with the steel beams. It has eight storeys (33 m) and is five bays (5@9 m = 45 m) by three bays (6 + 9 + 6 = 21 m) on plan. The structure was designed as non-sway with a central lift-shaft and two end staircases providing the necessary resistance to lateral wind loads. The main steel frame was designed for gravity loads and the connections, which consist of flexible end plates for beam-column connections and fin plates for beam-beam connections, were designed to transmit vertical shear only. The building was designed as a real commercial office in the Cardington area and all the structural components were specified to meet the most up-to-date British and European Standards. Detailed construction details of the building structure, e.g. architectural and construction drawings, steel section sizes, are given in a test report by Bravery [14].

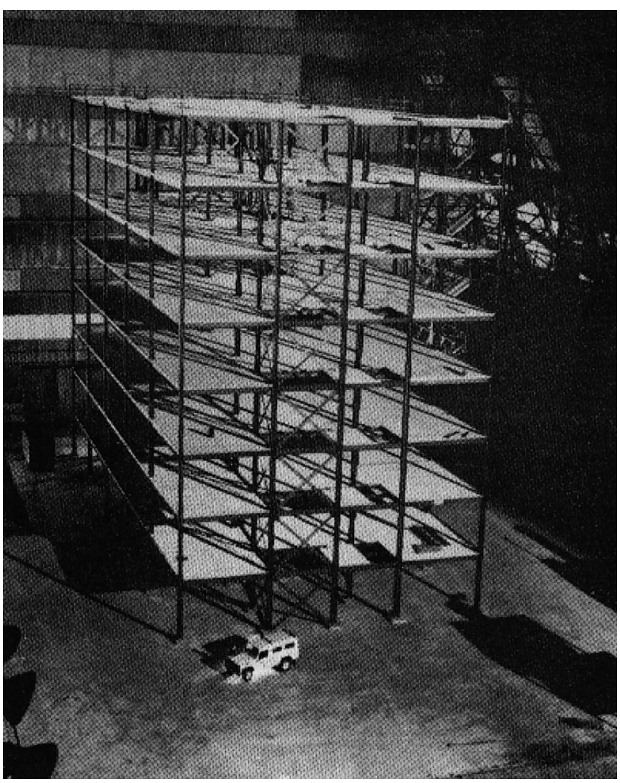

Fig. 1. The Cardington fire test building structure. © Building Research Establishment.

The building was designed for a dead load of 3.65 kN/m^2 and an imposed load of 3.5 kN/m^2. The floor construction is of steel deck and light-weight in-situ concrete composite floor, incorporating an anti-crack mesh of 142 mm^2/m (T6@200 mm) in both directions. The floor slab has an overall depth of 130 mm and the steel decking has a trough depth of 60 mm. As a consequence of mistakenly placing the reinforcement meshes directly on top of the steel decking, the anti-crack device was not effective and cracks appeared along all the primary steel beams.

Due to conservatism in design load specifications, only about 2/3 of the specified imposed load was applied during the fire tests. The imposed load was simulated using sandbags. Typically, 12 sandbags each of 1.1 ton were applied over an area of 9 m by 6 m, giving an uniform loading of 2.4 kN/m^2.

3.2. BRE fire tests and visual observations

The Building Research Establishment carried out two full scale fire tests in the building, being referred to as the Corner test and the Large compartment test. Fig. 2 shows the plan locations of these two tests. The details of these two tests, including precise locations of thermocouples, strain gauges and displacement measuring devices can be found in the two test reports by Lennon [15,16]. The following paragraphs summarise the main features.

3.2.1. Corner test

The corner fire test was carried out in one corner compartment of the building, enclosing a plan area of 9 m by 6 m. The identification of the steel beams and columns are illustrated inside Fig. 3. The fire load was provided with 40 kg/m^2 of wooden cribs of the floor area. A lightweight brickwork wall (gridline F) and two fire resisting partitions (gridline 3 and E) formed the boundary of the fire compartment. The remaining boundary wall (gridline 4) was constructed of a double glazed window of about 2.5 m high running the full length of 9 m and sitting on a 1.5 m high brick wall. The window was sealed when the fire was ignited, but when it became apparent that the fire was starved of oxygen, the stand-by fire brigade was asked to break the window twice during the fire test. When flashover eventually occurred, the double glazed window rapidly broke, creating an opening of the full area of the window. The recorded atmosphere temperatures in the middle of the compartment at various heights are presented in Fig. 3, showing the maximum temperature of over 1000°C and two local peaks as a result of forced opening.

Columns were heavily protected to prevent global

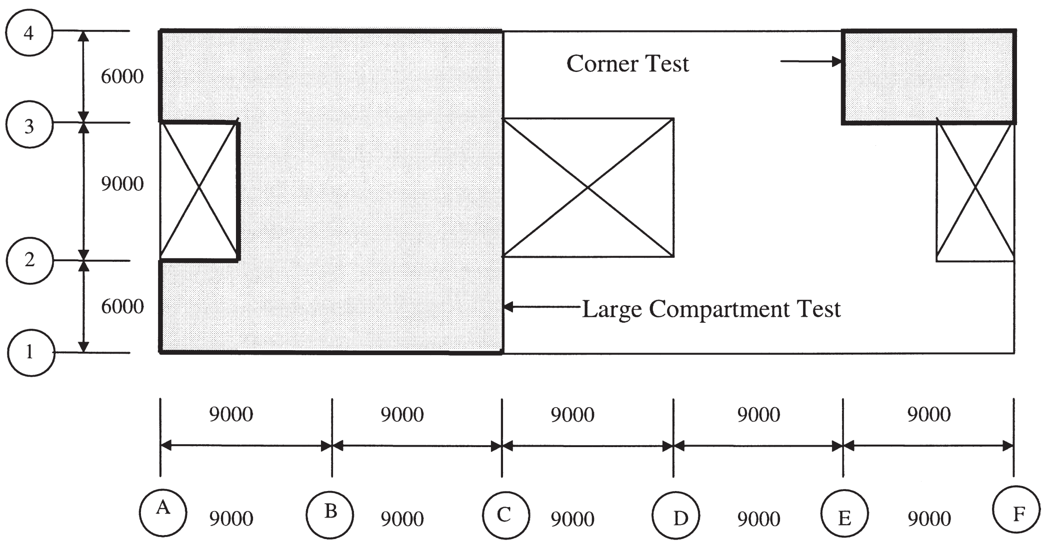

Fig. 2. Plan locations of BRE Corner and Large compartment fire tests.

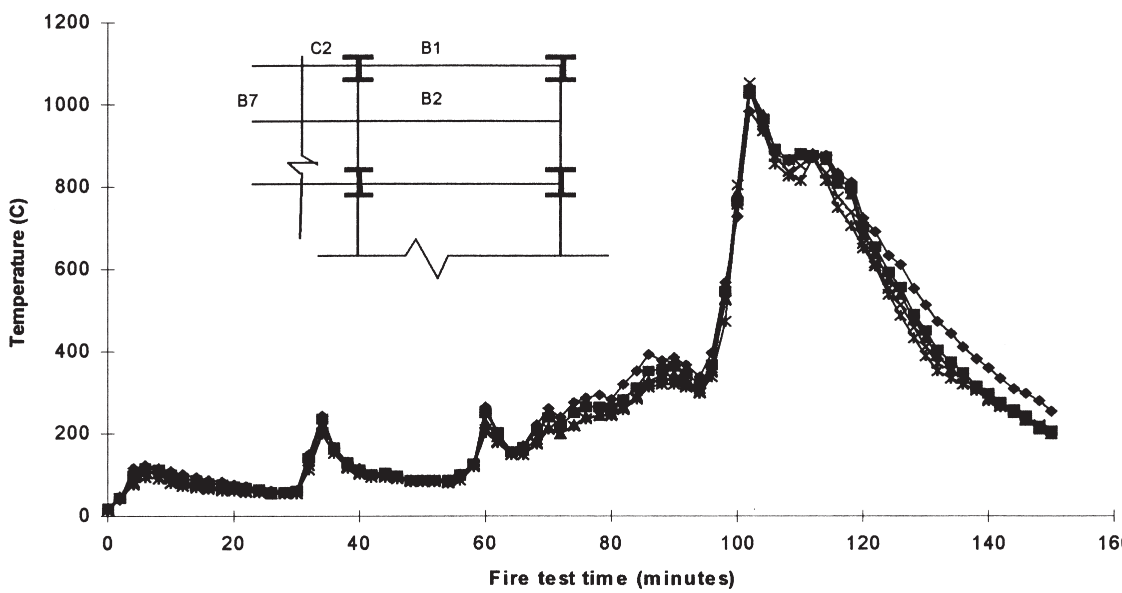

Fig. 3. Fire temperature-time relationships, Corner test.

structural instability and to limit fire damage to the fire test area. All steel beams were unprotected. The steel beam which was built into the lightweight brickwork wall (gridline F) was almost completely insulated, such that it attained very low temperatures and was unaffected by the fire. For the two steel beams which had partitions built underneath them (gridlines 3 and E), temperature distributions were non-uniform in the flange direction, thus lateral deflections due to thermal bowing were observed. However, since these beams were, on average, at low temperatures and were supported by the partitions, their structural behaviour was only slightly affected.

3.2.2. Large compartment test

As shown in Fig. 2, this test was carried out in a compartment occupying two bays (18 m) of the entire width of the building (21 m). Again, the fire load was provided with 40 kg/m^2 of wooden cribs of the floor area. The fire compartment was bounded with a lightweight brickwork wall (gridline A) at one end and fire resisting partitions at the other (gridline C). Each 2-bay side (gridlines 1 and 4) was bounded with two 6 metre wide single glazed assemblies with a 6 metre wide opening separating them. The opening was intended to simulate open windows in normal use and to avoid the need to break the windows during the fire test. Unfortunately, the wooden cribs were arranged in such a way that each pile had a rather high weight but was separated by a long distance from other piles. Even though the ceiling temperature reached the flashover temperature of about 600°C, each pile of wooden cribs was burning individually without joining together to form one combined large fire, in the true sense of flashover. As a result, a large part of each glazed assembly still remained in position, making the modelling of the fire behaviour difficult. Fig. 4 gives the locations of the fire affected primary beams and shows the recorded fire temperature-time relationships at different heights in the middle of the compartment, indicating a maximum fire temperature of only about 700°C.

Similar to the Corner fire test, all the steel beams were unprotected and all the columns were heavily protected to ensure overall structural safety. Fire damage was limited to the fire testing area.

4. Results of analysis

Due to their large quantity, it is not feasible to provide a detailed description of all the input information related to the analysis of these two tests reported in this paper. Interested readers should consult the test reports in references [14–16].

In order to save time and effort in preparing input data files, two-dimensional subframes were used to simulate the structural behaviour of the building. A subframe is usually formed of the structural member under consideration and its adjacent members.

Structural loading was assumed to be 4.9 kN/m^2, consisting of a self-weight of 2.5 kN/m^2 and an imposed load of 2.4 kN/m^2 simulated by sandbags. Nominal dimensions were used for all steel members. The effective concrete width was taken to be L/4 for internal steel beams and L/8 for edge beams, L being the beam span. Concrete depth was assumed to be 130 mm if the steel decking ribs were parallel to the steel beam and 70 mm if the steel decking ribs were at right angle to the steel beam. Steel yield stress was assumed to be 300 N/mm^2 and concrete cube strength 35 N/mm^2. The contributions of the anti-crack reinforcement mesh and steel decking were ignored.

Beam to column and beam to beam connections were assumed to be either rigid or simple joints.

4.1. Corner test

Although a comprehensive analysis of the Corner test results was carried out, only the following more important results are presented here. They include:

- the behaviour of the edge beam (B1 in Fig. 3) under direct fire attack,
- the behaviour of the secondary beam (B2 in Fig. 3) inside the fire compartment,
- slab load carrying capacity,
- the behaviour of the secondary beam (B7 in Fig. 3) immediately adjacent to the fire test compartment, and
- the change of bending moments in columns.

4.1.1. Edge beam B1

The edge beam B1 was designed as a simply supported beam of 9 metres but behaved as a three span continuous beam as a result of the supports provided by the two wind posts above the fire compartment. Fig. 5 compares test results with the predicted deflection-temperature relationships for both the simply supported beam and the three span continuous beam. It indicates that the simply supported beam would have failed at a temperature much lower than the test maximum temperature and has a different behaviour from that recorded. Conversely, the predicted behaviour of the

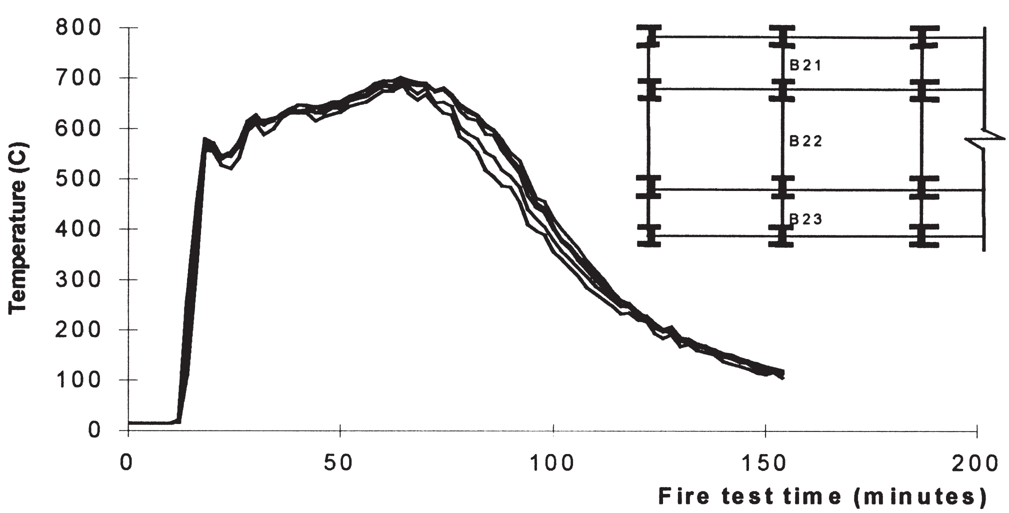

Fig. 4. Fire temperature-time relationships, Large comp. test.

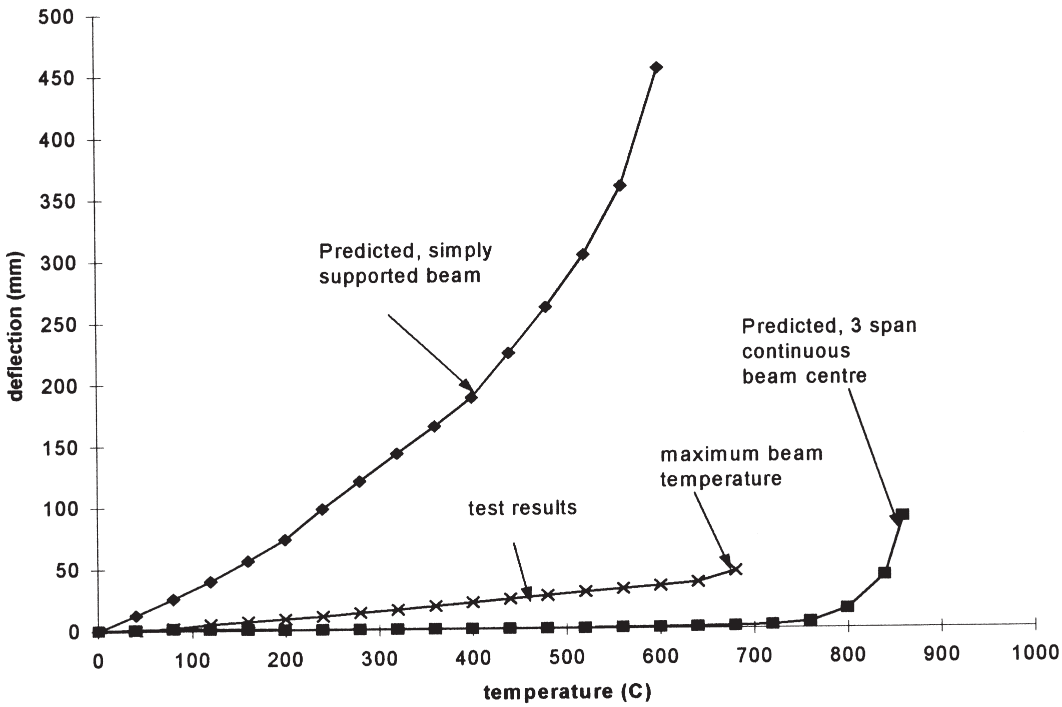

Fig. 5. Temperature-deflection relationships for beam B1, Corner test.

three span continuous beam was much closer to the test result and there was no indication of beam failure at the recorded maximum steel temperature. Wind post tightening during the fire test was thought to contribute to the difference between the recorded behaviour and the predicted continuous beam behaviour.

Clearly, the behaviour of this beam indicates that non-structural members can strongly influence the structural behaviour of primary loadbearing members. Although the benefit may be minimal for the ambient temperature design, the contributions of non-structural members to the fire performance of primary structural members may be taken advantage of in the fire resistant design of primary loadbearing members.

4.1.2. Internal beam B2

This beam was the main interest of the test. Fig. 6 compares the simulated temperature-deflection relationship with the recorded behaviour for B2. In the simulation, the five bay continuous beam model was adopted. The remote ends of the continuous beam were assumed to move freely in the horizontal direction. Fig. 6 shows that during the whole fire test, the beam/slab deflection-temperature behaviour was almost linear with no sign of run-away deflection. The predicted behaviour was reasonably close to the test results until about 500°C. However, it deviates from the test results after 500°C and indicates run-away beam deflection and failure at about 750°C, suggesting that the computer program was not capable of simulating the full range behaviour of B2.

Under fire conditions, the deflection in the steel beam is comprised of two parts: the thermal bowing deflection and the mechanical deflection. The thermal bowing deflection is due to non-uniform temperature distribution in the steel beam. The mechanical deflection is the increase in the beam deflection under constant load at decreasing stiffness due to reduced steel strength and stiffness at high temperatures. It is expected that at low temperatures (less than 500°C), the beam deflection is controlled by thermal bowing. At higher temperatures, mechanical deflection dominates and the beam deflection increases at a faster rate with a rise in the beam temperature.

This behaviour was reproduced in the computer simulation. In contrast, test results show an almost constant rate of beam deflection increase with increasing steel temperature, suggesting very little change in the beam stiffness. Since the steel beam was at very high temperatures and its flexural stiffness became very low, the test behaviour could only be sustained if the composite slab was supporting the steel beam.

4.1.3. Flexural strength of the composite slab

The composite floor slab was constructed of in-situ concrete supported on steel decking. Recorded results show very high temperatures in the steel decking, reaching the maximum of about 800°C. The steel decking was also observed to have debonded from the concrete slab in most areas. Thus it may be assumed that the steel decking contributed very little to the slab strength at the maximum fire severity. Furthermore, since the concrete in the steel decking trough also reached quite high tem-

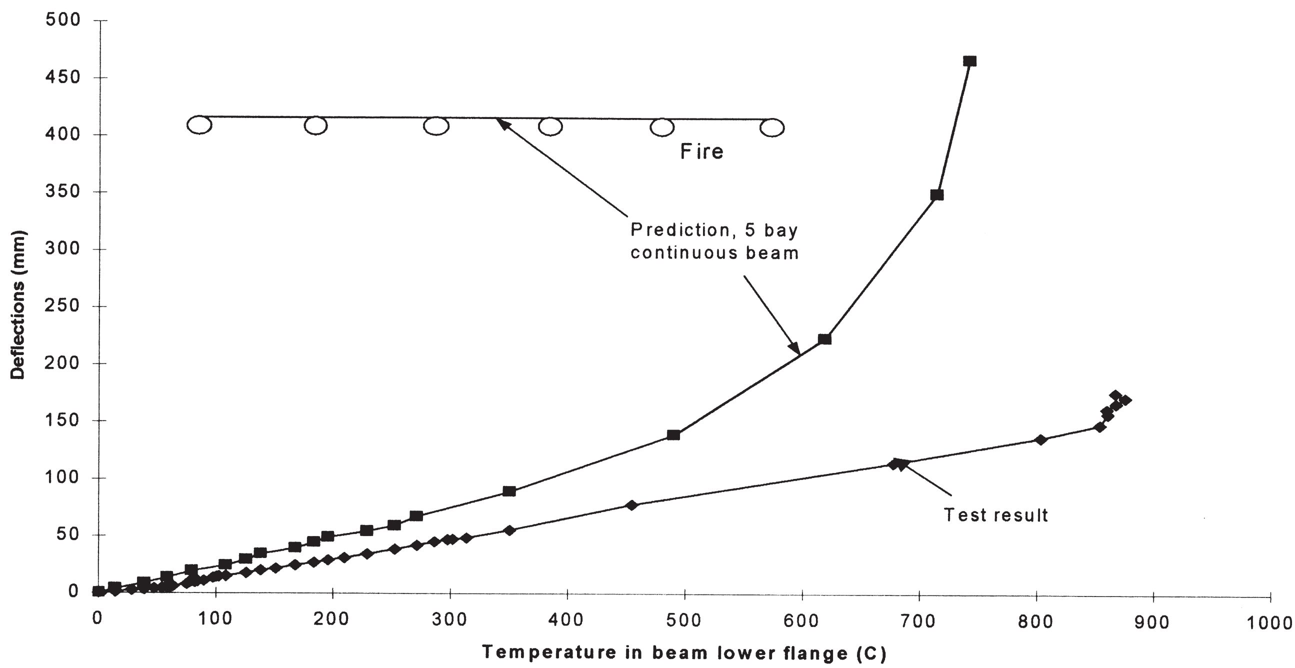

Fig. 6. Temperature-deflection relationships, beam B2, Corner test.

peratures, the slab hogging moment resistance was low. The slab may therefore be analysed as simply supported, and its total flexural bending strength may be calculated as follows:

Contribution from the simply supported reinforced concrete slab:

Reinforcement yield stress:	600 N/mm^2
Reinforcement force:	600 N/mm^2 × 142 mm^2/m = 85200 N/m
Depth of concrete block in compression:	85200/(0.67*35*1000.0) = 3.6 mm
Depth of concrete cover to reinforcement:	50 mm
Length of level arm:	50 − 3.6/2.0 = 48.2 mm
Slab sagging moment capacity:	85.2 kN/m*48.2/1000.0 = 4.11 kN.m/m
Flexural strength of slab according to yield line theory [17]:	17.1 × 4.11/(6 m × 6m) = 1.95 kN/m^2

Contribution from flexural strength of the composite beam:

Steel temperature:	900°C
Reduced steel strength [1]:	300 N/mm^2*0.08 = 24 N/mm^2
Tensile force in 356x171UB51:	24*5150 mm^2 = 123600 N = 123.6 kN
Effective width of concrete:	9000/4.0 = 2250 mm
Depth of concrete block in compression:	123600/(2250*0.67*35) = 2.34
Depth of 356 × 171UB51:	303.8 mm
Length of level arm:	303.8/2.0 + 130 − 2.34/2.0 = 280.73 mm = 0.281 m
Sagging moment capacity of composite beam:	0.281 × 123.6 = 34.73 kN.m
UDL on composite beam:	8*34.73/(9 m*9 m) = 3.43 kN/m
Equivalent UDL on slab:	3.43/3.0 = 1.14 kN/m^2

Total flexural strength of floor slab:

1.95 + 1.14 = 3.09 kN/m < applied load = 4.9 kN/m^2

It is possible that other means, e.g. the slab hogging moment capacity and the steel decking, may contribute to resisting the applied load, however, the total flexural bending load carrying capacity would still not be sufficient. In any case, if the slab was nearing its collapse, an accelerating rate of beam/slab deflection would have been observed. The observation that the slab showed no sign of imminent collapse indicates that the beam/slab load carrying mechanism changed from flexural bending to a different one. It is thought that at the large deflections experienced in the test, tensile membrane action was the likely alternative load carrying mechanism [18].

Thus, this analysis suggests that flexural bending may not be the only load carrying mechanism in the whole range behaviour of a complete building structure. In order to accurately simulate the history of the building structural behaviour, large deflection and post-failure analyses should be carried out. In the improved analysis, the two-way slab behaviour should also be correctly modelled.

4.1.4. Cold beam B7

The behaviour of this cold beam was analysed to assess whether additional consideration should be given to cold beams when carrying out fire resistant design for hot ones. This was done by checking the bending moment change in the cold beam which is immediately adjacent to the hot one. Due to the unknown concrete strains, the test bending moment values in this beam could not be accurately determined from measured steel strains alone. Instead, Fig. 7 presents a comparison between the observed and predicted beam curvatures at the indicated locations. The predicted beam curvature was calculated as the predicted beam moment divided by the initial beam rigidity. Fig. 7 shows a qualitative agreement between the predicted behaviour and test results, and if the reduced transient beam rigidity at high temperatures had been used, the agreement would have been better.

The more important point from Fig. 7 is the qualitative changes in the beam bending moments. An increase in the negative curvature in Fig. 7 indicates an increase in the beam hogging moment. When the temperature distribution in B2 was non-uniform, the restraining cold beam (B7) developed hogging moment which increased with a rise in the hot beam temperature gradient and decreased with a reduction in the temperature gradient and bending stiffness of B2. This behaviour was reproduced in the computer simulation and was also followed by the test results. In addition, at high hot beam temperatures, the restraining effect became negligible.

Thus, this analysis suggests that fire has minimal effect on beams outside the fire compartment and also confirms the deterioration in the stiffness of the hot beam B2.

4.1.5. Columns

In the so-called "Plane frame" fire test previously conducted by British Steel, the internal columns were fire protected only to the level of the assumed false ceiling position, leaving a column length of about 400 mm unprotected. Fire test observations afterwards showed that the entire unprotected length was completely squashed. Although the steel structure remained stable after the fire attack, the fire damage extended to adjacent bays and to all the floors above the fire testing compartment. Consequently, columns in all other fire tests were heavily protected to prevent global structural failure and to limit the extent of fire damage. As a result, their temperatures were low (maximum about 200°C) and the test results are not sufficient to quantify their structural behaviour at the fire limit state.

Nevertheless, the test results are able to give an indication of the column behaviour and to confirm the validity of the computer program to predict structural interaction before any local structural member failure. For example, Figs. 8 and 9 show the observed and calculated major axis bending moment changes in column C2 at the two indicated locations. The analysis was carried out using a subframe enclosing the fire testing storey and all the columns above and below the fire testing storey. Calculated results are given for both simply supported and fixed remote column boundary conditions in the adopted subframe.

A number of interesting points may be noted from the results in Figs. 8 and 9. Firstly, the predictions of the computer program follow the test results quite well, the difference between them may largely be attributed to the difficulty in using a set of precise temperatures in the beam supported by the partitions (gridline E). Secondly, a high

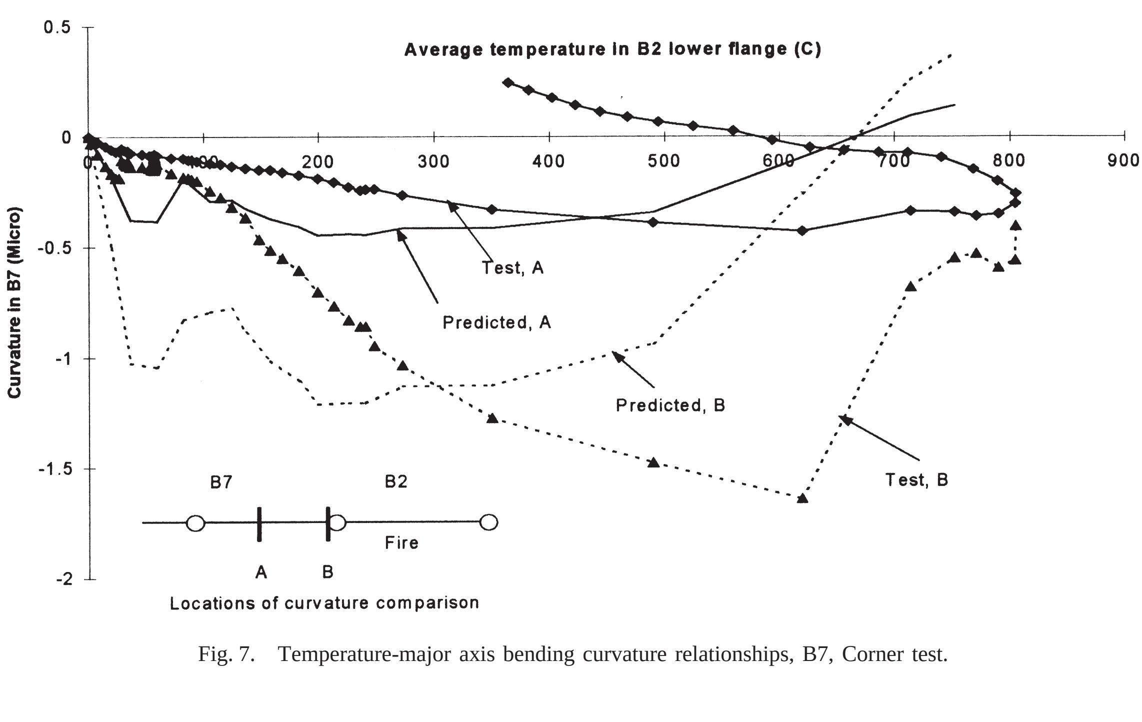

Fig. 7. Temperature-major axis bending curvature relationships, B7, Corner test.

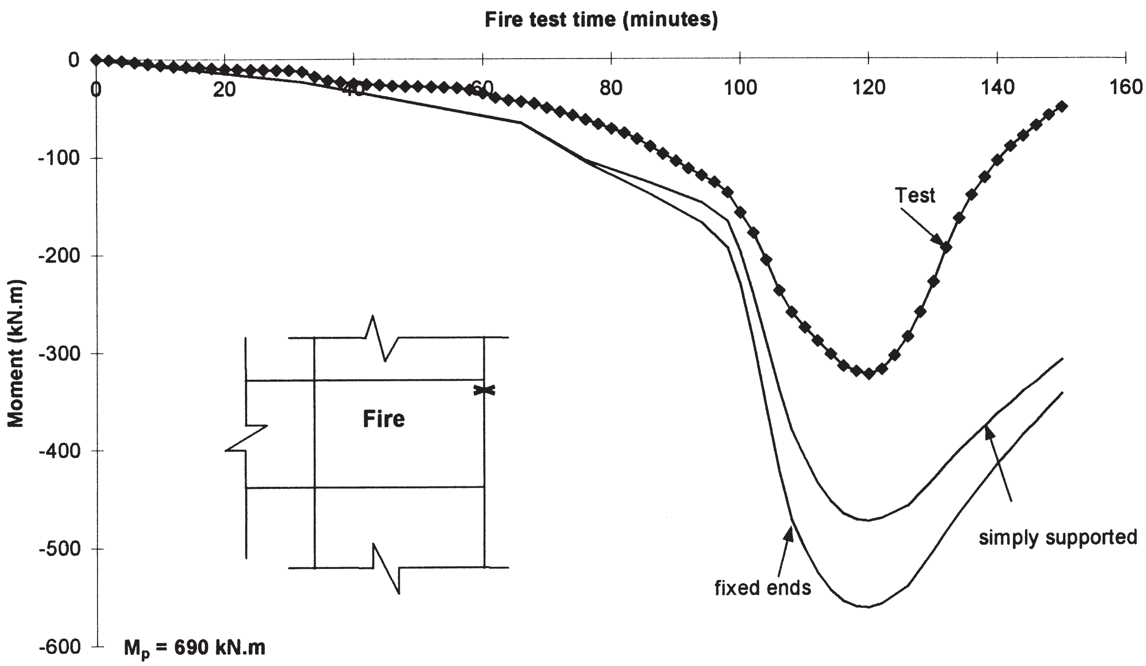

Fig. 8. Comparison for column major axis bending moment, Corner test.

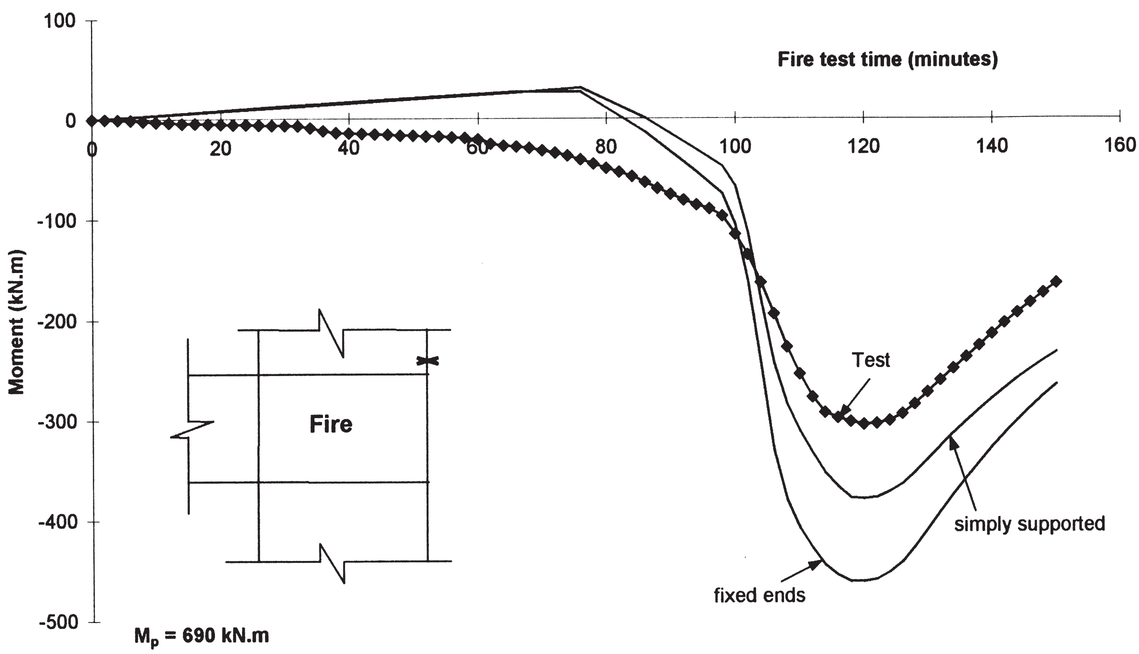

Fig. 9. Comparison for column major axis bending moments, Corner test.

level of additional bending moment was produced in the columns. Further examination indicates that this additional bending moment was mainly a result of beam thermal expansion. Finally, the column response was not very sensitive to the remote column boundary condition, justifying the use of subframes in carrying out the analysis.

The increase in column bending moment, as a result of being pushed by the adjacent beam thermal expansion, may require consideration in the column fire resistant design, especially bearing in mind the importance of column stability. However, a separate study by the author [19] seems to indicate that, at higher column temperatures, as the column deflects away from the expanding beam, the additional column bending moment would reduce to a negligible level due to the hot beam changing its role from pushing to pulling the column.

This may be explained using Fig. 10. Assuming a continuous column, simply supported at the ends, of floor height L is pushed to the left by a displacement of Δ_h and the column has a uniform bending stiffness $(EI)_c$, the bending moment in the column may be calculated using the following equation:

$$M = \frac{3(EI)_c}{L^2}\Delta_h$$

The displacement Δ_h may be divided into two parts, Δ_b being the thermal expansion of the adjacent beam, and Δ_c being the lateral deflection of the column, such that:

$$\Delta_h = \Delta_b - \Delta_c$$

Therefore, when the column is at low temperatures, its lateral deflection Δ_c is small, giving a large net displacement Δ_h, leading to a large column bending moment. This is thought to have occurred during the Cardington fire test. At high column temperatures, as the column approaches its failure, its lateral deflection Δ_c becomes large, giving a small net displacement Δ_h (since the beam axial stiffness will be much higher than the column bending stiffness, the net displacement Δ_h will always approach zero), leading to a small column bending moment.

Attempts were made to correlate theoretical predictions of the column compressive force increases with test results. However, as the column temperatures were low and also non-uniform both in the cross-section and along the length, the additional column axial forces were low and the results of the analysis were not conclusive.

4.2. Large compartment test

Although the scale of this fire test was large, the observed fire behaviour was not as severe as that of the Corner test, therefore, this test actually provided less information on the steel frame structural behaviour. Nevertheless, the behaviour of the primary beams (B21/B22/B23 in Fig. 4) is worthy of further discussion.

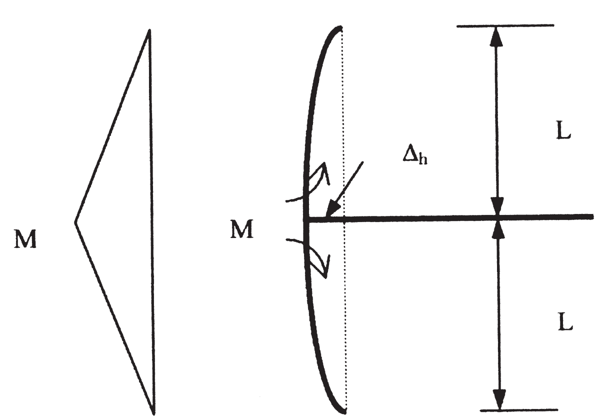

Fig. 10. Column bending moment due to lateral deformation.

4.2.1. Primary beam deflection

Fig. 11 compares the observed and predicted deflection history of beams B21/B23, with similar results being obtained for beam B22. The predicted results were obtained using a subframe constructed by the fire testing storey and two cold storeys above and two below the test storey. From Fig. 11, it is noted that: (1) The beams were still under flexural bending and there was no sign of any run-away deflection; (2) Non-symmetrical behaviour was observed due to non-uniform heating condition in the test compartment. This could not be simulated but the difference between the behaviour of beams B21 and B23 was small; (3) Predicted beam behaviour show substantial beam-column connection rigidity since the computer program predicted much higher simply supported beam deflections than the test results.

4.2.2. Beam bending moment

Owing to the large scale of this fire test, a decision was made before the fire test only to strain gauge selected columns. Thus it was not possible to compare the predicted and observed forces in the beams. However, the computer program produced some interesting results for the bending moments when the hot beam was cooling down.

An example is shown in Fig. 12 for the bending moment variations in beams B21/B23 at two locations. As expected, heating of the beam gave rise to an increase in the beam hogging moment due to temperature gradient. As the temperature gradient decreased and the steel beam lost some of its rigidity, the hogging moments started to decrease at about 400°C. At the maximum temperature of about 600°C, the overall beam moments were comparable to their initial values. However, when the beam started to cool down, sagging moments started to increase at a rate similar to the initial rate of hogging moment increase. As a result, the beams seem to have residual bending moments much higher than their initial values. Whilst the beams in the test building had higher bending moment capacity than the theoretical residual bending moments, it is not clear whether, under other circumstances, e.g. higher initial sagging bending moments, steel beams may fail during cooling.

4.2.3. Column behaviour

The column behaviour was also comprehensively analysed, but the results were not conclusive due to the very low column temperatures. Similar to columns in the Corner test, substantial additional bending moments were observed at the recorded column temperatures.

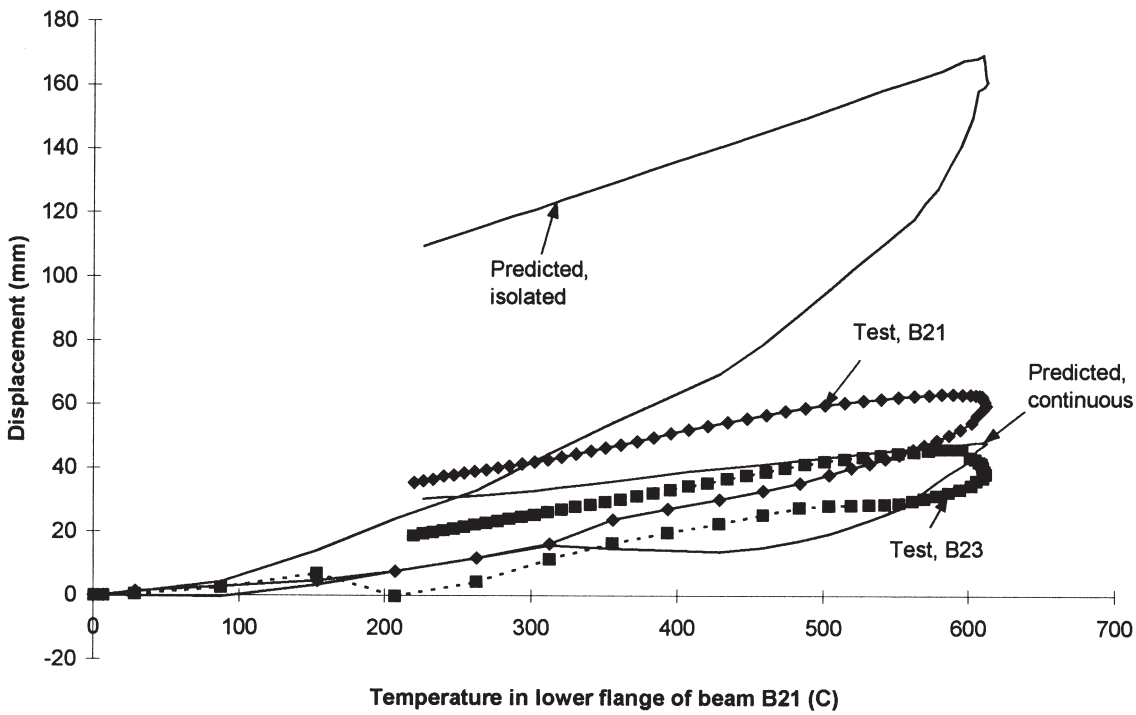

Fig. 11. Comparison for displacements for B21/B23, Large comp. test.

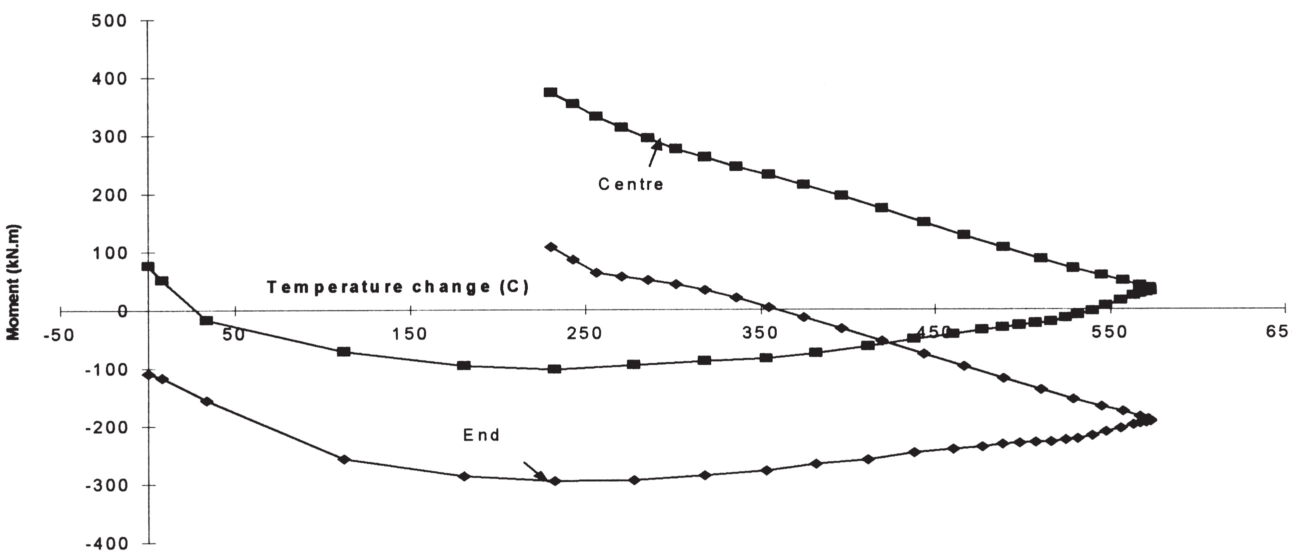

Fig. 12. Predicted major axis bending moment in beam B21, Large comp. test.

5. Conclusions

This paper presented an overview of the two BRE large scale fire tests in the 8-storey steel framed building at Cardington, and some results of an analysis of these tests using a finite element computer program developed by the author [9]. The Cardington tests provided a unique opportunity to study the behaviour of a complete building structure under realistic fire conditions. The test results are still being intensively analysed by a number of researchers, thus it is not possible to draw a comprehensive list of conclusions at this stage. This paper presents the results of the author's analysis and the following tentative conclusions may be drawn:

1. During the Corner test, the Cardington steel building structure exhibited a large deflection behaviour that could not be modelled by pure flexural bending. It is concluded that numerical procedures capable of dealing with membrane action in the floor slabs at large deflections should be included.

2. Large additional bending moments may generate in columns as a result of being pushed by the adjacent hot beam's thermal expansion. However, it is not clear whether this effect would still be present when the column approaches failure in fires.
3. Members regarded as non-loadbearing at the ambient temperature condition may contribute to the fire resistance of primary loadbearing members. This benefit may be taken advantage of in the fire safety design of the concerned primary loadbearing member.
4. Large sagging bending moments (exceeding the initial value) may develop when a hot beam cools down. However, it is not clear whether this may result in beam failure during cooling.

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