Steel and Composite Structures, Vol. 3, No. 3 (2003) 153-168
DOI: http://dx.doi.org/10.12989/scs.2003.3.3.153

Large scale fire test on a composite slim-floor system

C. G. Bailey†

Manchester Centre for Civil and Construction Engineering, The University of Manchester, PO Box 88, Manchester, M60 1QD, UK

(Received May 3, 2002, Accepted May 29, 2003)

Abstract. This paper discusses the results and observations from a large-scale fire test conducted on a slim floor system, comprising asymmetric beams, rectangular hollow section beams and a composite floor slab. The structure was subjected to a fire where the fire load (combustible material) was higher that that found in typical office buildings and the ventilation area was artificially controlled during the test. Although the fire behaviour was not realistic it was designed to follow as closely as possible the time-temperature response used in standard fire tests, which are used to assess individual structural members and forms the bases of current fire design methods. The presented test results are limited, due to the malfunction of the instrumentation measuring the atmosphere and member temperatures. The lack of test data hinders the presentation of definitive conclusions. However, the available data, together with observations from the test, provides for the first time a useful insight into the behaviour of the slim floor system in its entirety. Analysis of the test results show that the behaviour of the beam-to-column connections had a significant impact on the overall structural response of the system, particularly when the end-plate of one of the connections fractured, during the fire.

Keywords: fire test; asymmetric beams; slim floor; structural behaviour; elevated temperatures; connections; composite slab.

1. Introduction

It is possible for the designer to adopted steel-framed systems for buildings that have sufficient inherent fire resistance, such that no applied fire protection, typically in the form of boards, sprays or intumescent coatings, is required to exposed parts of the steel structure (Bailey and Newman 1998). By removing the need to specify applied fire protection, material cost savings can be obtained, together with savings associated with fixing the protection. Possible savings can also be obtained by the elimination of the typical delay that installation has on the completion of the building. For steel beams, one such system involves embedding most of the beams cross-section within the supported concrete floor slab, with only the bottom flange exposed to any possible fire. This type of beam is commonly referred to as a slim floor beam.

In the UK the most common form of slim floor beams consists of an asymmetrical beam and rectangular hollow section edge beam (Fig. 1), marketed by Corus, with design guides published by the Steel Construction Institute (Lawson *et al*. 1997, Mullett 1997). These types of beams can readily achieve 60 minutes fire resistance, when subjected to normal office loading, without the need to protect

†Professor of Structural Engineering

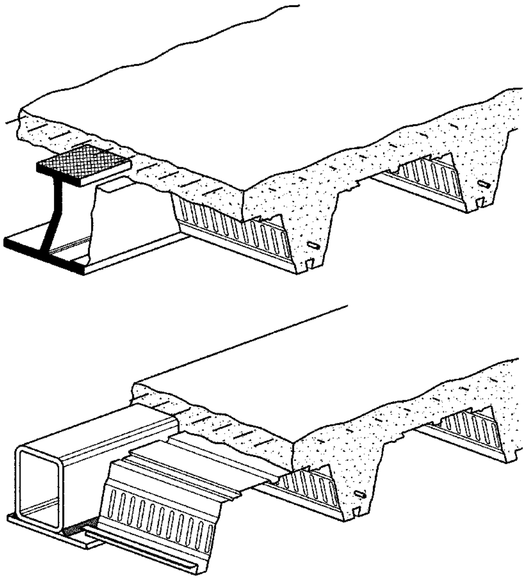

Fig. 1 Asymmetrical and rectangular hollow section slim floor beams

Fig. 2 Building under construction using asymmetric slim floor beams

the exposed bottom flange. For higher loads, or higher fire resistance periods, the exposed bottom flange will typically need to be protected. Optimum cross-sectional sizes were derived for the asymmetrical slim floor beam based on ultimate, serviceability and fire limit states. The use of rectangular hollow section slim floor beams has become a popular solution for edge beams due to their good torsional resistance, which is required during the construction stage. Slim floor beams can either be used with precast concrete floor slabs or composite floor slabs comprising steel deck, bar and mesh steel reinforcement and either lightweight or normal weight concrete. The use of asymmetric slim floor beams with a composite floor slab is shown in Fig. 2 for an office block under construction, which required 60 minutes fire resistance.

This paper presents the results from a large-scale demonstration fire test, designed to investigate the performance of a slim floor system, comprising asymmetrical beams and rectangular hollow section edge beams, supporting a composite slab. The aim of the test was to extend the knowledge gained from previous member and small-scale tests and to investigate, for the first time, the behaviour of the system in its entirety, including the behaviour of the beam-to-column connections.

Unfortunately a large number of thermocouples measuring the atmosphere temperature and the temperature through the cross-section of the beams malfunctioned during the test. This lack of test data significantly hinders the structural modelling of the test and prohibits the presentation of definitive conclusions on the structural behaviour of the system. However, sufficient test data and observations were obtained which indicate aspects of structural behaviour that were not identified from the previous small-scale tests. The limited test data allowed some tentative computer modelling to be conducted, which is presented in this paper and highlighted some of the important aspects relating to the integrity of the tested system.

Before the results and observations from the large-scale fire test are presented it is worth briefly reviewing previous experimental and theoretical research on the asymmetrical and rectangular hollow section slim floor beam system.

2. Previous experimental and theoretical work

Two standard fire tests were conducted in 1996 (Lawson *et al.* 1997) on prototype asymmetrical beams, where the beams spanned 4.5 m and the BS476 Part 20 (1990) standard time-temperature relationship was adopted. Fire resistance periods of 107 minutes and 75 minutes were achieved in these tests, with the beams subjected to a lower load than that generally assumed in design. Analysis of the test results, using purpose written thermal and mechanical models, allowed the models to be validated and design tables to be developed. These design tables show that beams subjected to normal office loading will generally achieve 60 minutes fire resistance.

Structural finite element computer models have been developed (Bailey 1999, Cai *et al.* 2002) which have shown to provide excellent predictions of the standard fire tests on asymmetric beams. Using the developed model, the effect of the beam-to column connections was investigated (Bailey 1999). This theoretical study concluded that connections, which are typically assumed in ambient temperature design to transfer zero moment, are beneficial to the survival of the beam in fire.

In 1996 a fire test was carried out at TNO (Mullett 1997) on a composite floor slab, spanning 5.6 m onto two rectangular hollow section slim floor beams, which spanned 4.6 m onto vertical ‘pinned’ supports. The test was carried out in TNO’s furnace and the standard time-temperature relationship was used. Similar to the approach for asymmetrical beams, the test results were used to validate thermal and structural models, which were then used to develop design tables.

To extend the knowledge gained from the previous fire tests, on the asymmetrical and rectangular hollow section slim floor beams, it was decided to carry at a large-scale test on the system as a whole. The primary aim of this test was to demonstrate that the system in its entirety, which included the effect of beam-to-column connections, has a greater inherent fire resistance compared to that shown from the previous small-scale tests. The knowledge gained from the test and supplementary computer modelling could be incorporated into future design, which will be a significant improvement on the current design approach which is predominately based on member behaviour.

3. The test structure

The tested structure was designed by the Steel Construction Institute and comprised four bays 6.1 m

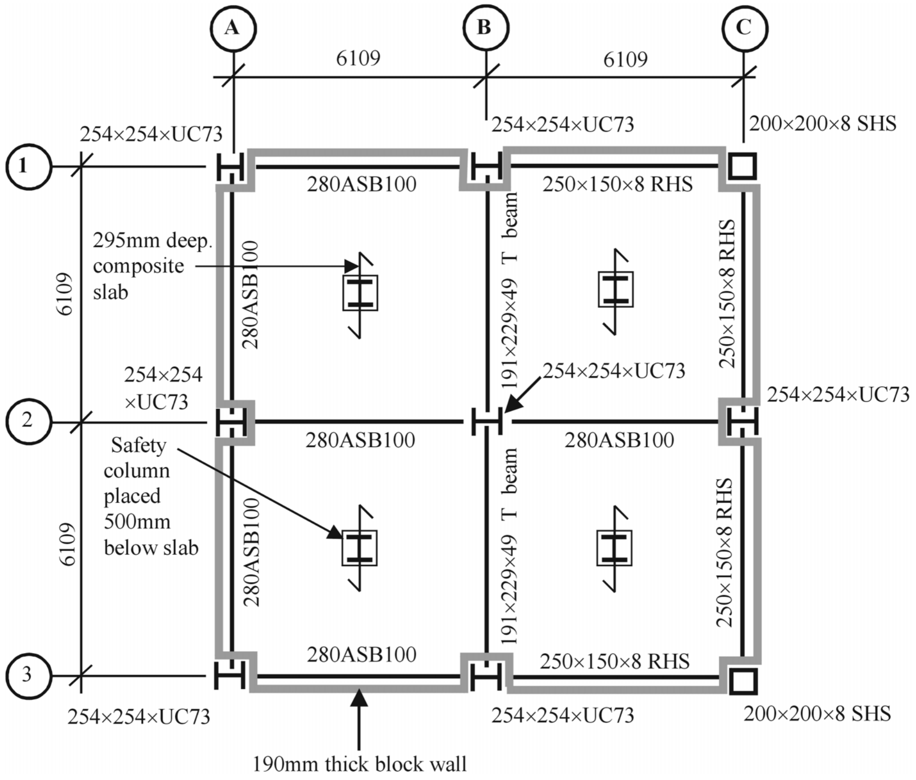

Fig. 3 Plan of fire test compartment

by 6.1 m (Fig. 3). The composite floor slab, which had an overall depth of 295 mm, consisted of a 225 mm deep steel deck with normal weight, grade C30, concrete. Reinforcing bars, 20 mm diameter, were placed in the centre of each trough of the deck (600 mm centres) and A142 (6 mm diameter bars at 200 mm centres) anti-crack mesh reinforcement was placed 20mm from the top of the slab. The reinforcement had a nominal yield strength of 460 N/mm^2. The floor slab was supported by a central asymmetric beam, designated as a 280ASB100 (Lawson *et al*. 1997), with a mixture of edge beams consisting of asymmetric beams or beams constructed from a rectangular hollow section (RHS) with a welded bottom plate (Mullet 1997). All the steel beams had a nominal yield strength of 355 N/mm^2. Fig. 4 shows the steel deck, bar and mesh steel reinforcement, and the asymmetric beam on gridline 2, before pouring of the concrete.

In the normal condition the asymmetric beam was designed as a composite beam following the design procedure based on test rests (Lawson *et al.* 1997). In the design of the internal composite asymmetric beams an effective width of span/8 was assumed, which is half the value taken for typical downstand beams that act compositely with the supporting floor slab. The compressive force in the concrete was also limited in the design by the longitudinal shear-bond between the steel beam and concrete encasement, which is enhanced by the rib pattern on the top flange of the asymmetric beam. Back analysis of the test results (Lawson *et al*. 1997) showed that a design value of 0.6 N/mm^2 could be assumed for the bond stress acting over an effective perimeter equal to the top flange and web of the beam. The asymmetrical edge beams were also designed as composite with an assumed effective breadth of span/16. The rectangular hollow section edge beam was assumed in the design to be non-

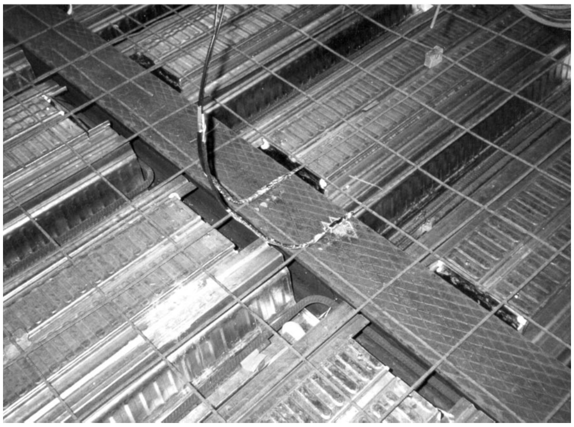

Fig. 4 Picture showing steel deck, reinforcement and asymmetric beam before pouring of concrete

Fig. 5 Picture showing timber cribs

composite.

The supporting columns were either a 254×254×73UC (Universal Column) or a 200×200×8SHS (Square Hollow Section) as shown in Fig. 3. The compartment wall was constructed using 190 mm thick block wall with a 265 mm vertical gap left between the wall and underside of the tested structure. The gap was filled with a flexible ceramic blanket, which contained the fire whilst allowing vertical movement of the tested structure without providing support from the block wall. The wall was constructed such that the external columns were outside the compartment and not exposed to the fire. The internal column had blocks placed between the flanges and was protected using ceramic fibre.

Ventilation to the fire compartment was provided by two large openings along gridline 3. The size of these openings was varied during the test using manually operated moveable screens. The purpose of varying the ventilation openings was to try and follow, as closely as possible, the time-temperature response of the standard fire test (BS476, 1990). The procedure of varying the ventilation creates an artificial fire scenario that has no relation to reality. However, the purpose of the test was not to investigate the fire behaviour but instead to investigate the structural response of the system. By forcing the fire to follow the standard time-temperature relationship, it would be possible to relate the structural behaviour to the behaviour previously observed in standard fire tests and allow the test results to be compared against current design approaches, which are based on fire resistance periods. Four small ventilation holes were also provided in the compartment wall along gridline 1 to prevent the possibility of oxygen starvation in this area, with the aim of creating a reasonably uniform fire temperature throughout the compartment.

The imposed load was provided by 49 sandbags, each weighing 10.79 kN, spread equally over the floor area. The number of sandbags and resulting imposed load was limited by the physical space available on the floor slab. The total applied load, including the self-weight of the floor slab, was 6.88 kN/m^2.

Current fire design codes (BS5950-8, 1990, ENV1993-1-2, 1995 and ENV1994-1-2, 1994) measure the performance of steel structures by the ratio of the load applied to the member during a fire, divided by the capacity of the member at ambient temperature. The UK Code BS5950-8 (1990) defines this ratio as the load ratio, whereas the Eurocodes (1994, 1995) adopt the terminology, load level, load intensity or degree of utilisation. Adopting the common design assumption that the slab and supporting beams are simply-supported, the load ratio (using UK terminology) of the middle asymmetrical beam was 0.35. The edge ASB spanning perpendicular to the floor slab had a load ratio of 0.19, whereas the RHS beams spanning perpendicular to the floor slab had a load ratio of 0.35. To put these values in perspective, using current design methods (Lawson *et al*. 1997 and Mullet 1997) the maximum load ratio for 60 minutes fire resistance is 0.47 for the ASB and 0.46 for the RHS edge beam. Therefore, even without including the expected beneficial effect of connection continuity, the beams used in the test structure should have in excess of 60 minutes fire resistance.

The fire load was 55 kg/m^2 (990 MJ/m^2) and consisted of timber cribs placed uniformly within the compartment. The fire load was significantly higher than the design 80% fractile value of 511 MJ/m^2 for offices, given in the Eurocode (prEN1991-1-2, 2001). However, as previously discussed, the purpose of the test was not to investigate the behaviour of realistic fires, but to investigate the structural behaviour of the system when subjected to a severe fire. Fig. 5 shows the timber cribs and a view inside the compartment prior to the test. Four timber cribs were placed in cages, as shown in Fig. 5, to allow the mass loss of the crib to be recorded during the fire.

Four protected steel columns were placed in the centre of the four 6.1 m by 6.1m bays (Fig. 3), with a gap between the top of the column and underside of the slab of approximately 500 mm. These columns were specified for safety reasons, and would provide additional support to the slab should the vertical displacement at the centre of the bay reach 500 mm.

4. Test results and observations

The test was carried out at the Building Research Establishment (BRE) Cardington laboratories in October 1998 (Fig. 6), with the construction and installation of the instrumentation managed jointly between BRE and Corus. The atmosphere temperatures were measured at 16 locations within the compartment (Fig. 7). Unfortunately a number of thermocouples measuring the atmosphere temperature malfunctioned during the test, and only measurements at locations A1, A6, A11 and A12 were recorded. The maximum atmosphere temperature of 1118 °C occurred at location A6 and was recorded 60 minutes after ignition. The maximum and average atmosphere temperatures are shown in Fig. 8, together with the time-temperature response used in a standard fire test. Inspection of Fig. 8 shows that

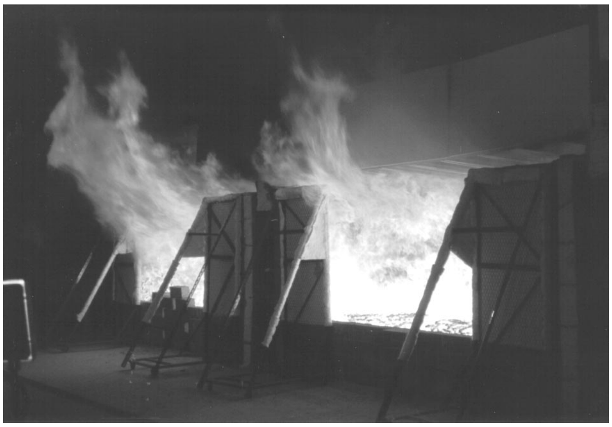

Fig. 6 Fire test in progress

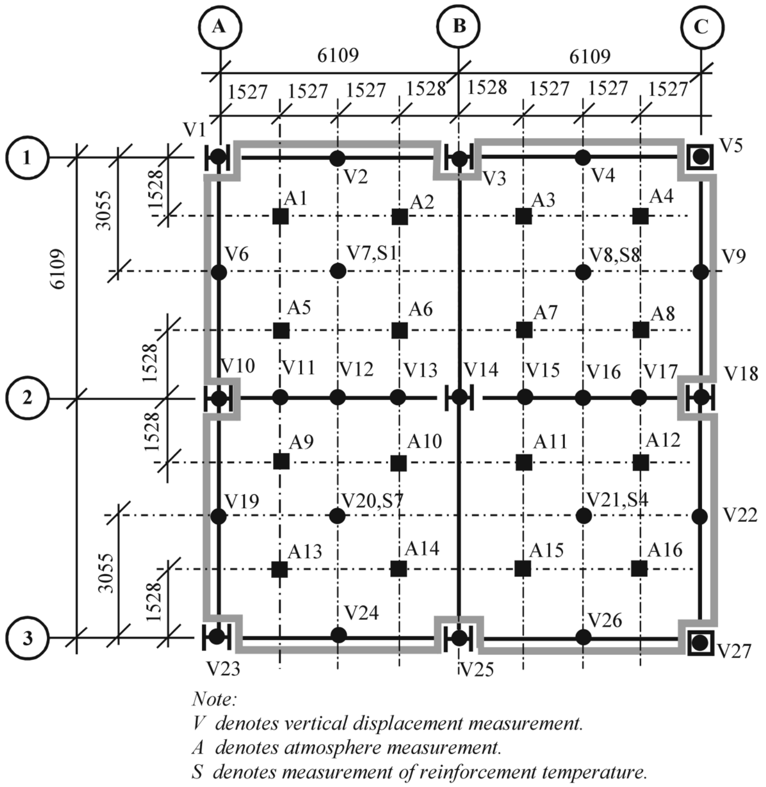

Fig. 7 Location of instruments measuring atmosphere temperatures and vertical displacements

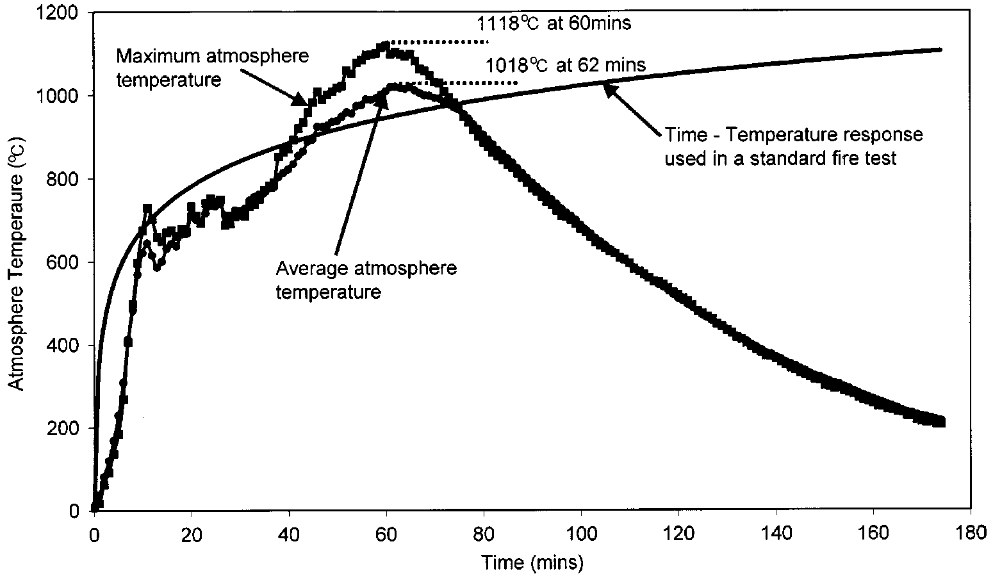

Fig. 8 Average and maximum atmosphere temperature

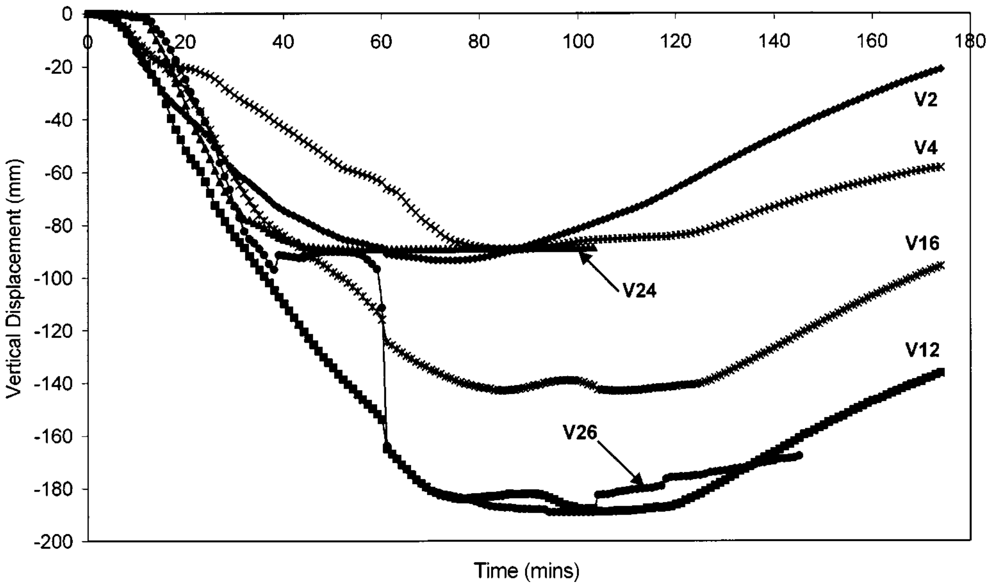

Fig. 9 Vertical displacement at centre of beams (refer Fig. 7 for location)

between 18 and 28 minutes the average temperature fluctuated between the trends of increasing and decreasing. This behaviour was caused by the manual operation of changing the ventilation openings, which was deemed necessary during the test to try and follow the standard time-temperature response. Considering the crude method adopted for artificially controlling the fire the correlation between the test temperature and the standard relationship is fairly good up to the maximum temperature recorded.

The vertical displacement of the beams on gridlines 1, 2 and 3 are shown in Fig. 9, with the vertical displacement of the slab at locations V7, V8, V20 and V21 shown in Fig. 10. Comparison of the central vertical displacement of the two asymmetrical beams on gridline 2 shows that the beam between gridlines A and B (location V12) is higher than the beam between gridlines B and C (location V16). Due to the reasonable symmetry of the structure and applied static load, it is thought that the difference in vertical displacement was due to the varying temperatures across the fire compartment. Unfortunately, due to malfunction of a significant number of thermocouples measuring the atmosphere temperature and temperature through the beams, there is insufficient test evidence to support this theory. An interesting observation is the increase in vertical displacement of the edge beam on gridline 3, between gridlines B and C (location V26, Fig. 9) from 96 mm to 167 mm between 59 and 61 minutes. After 61 minutes, the displacement at location V26 continued to increase, at a slower rate, up to 184 mm (recorded at 78 minutes) and then remained fairly constant.

Compared to the displacement measurements at location V26, similar behaviour was recorded at the centre of the slab between gridlines B and C and gridlines 2 and 3 (location V21), as shown in Fig. 10. At location V21 an increase in displacement of 150 mm occurred between 59 and 61 minutes, with the displacement then becoming fairly constant just over 500 mm. This strongly suggests that the slab was supported by the protected steel column, placed at this location to provide additional support to the slab should its displacement exceed 500 mm. The additional support, at location V21, provides an explanation of why the rate of displacement at location V26 (edge beam) decreases after signs of 'runaway' between 59 and 61 minutes.

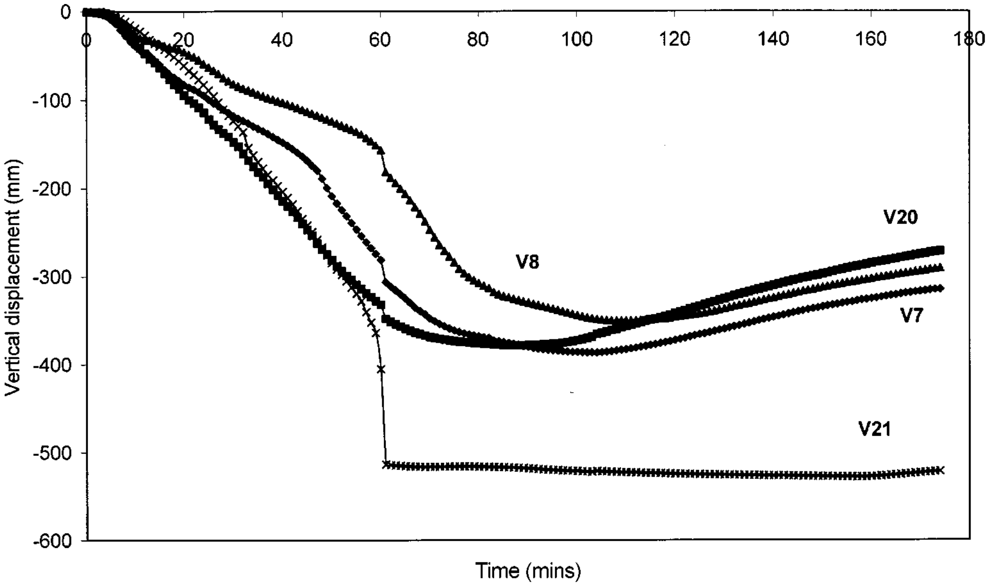

Fig. 10 Vertical displacement at centre of floor slab (refer Fig. 7 for location)

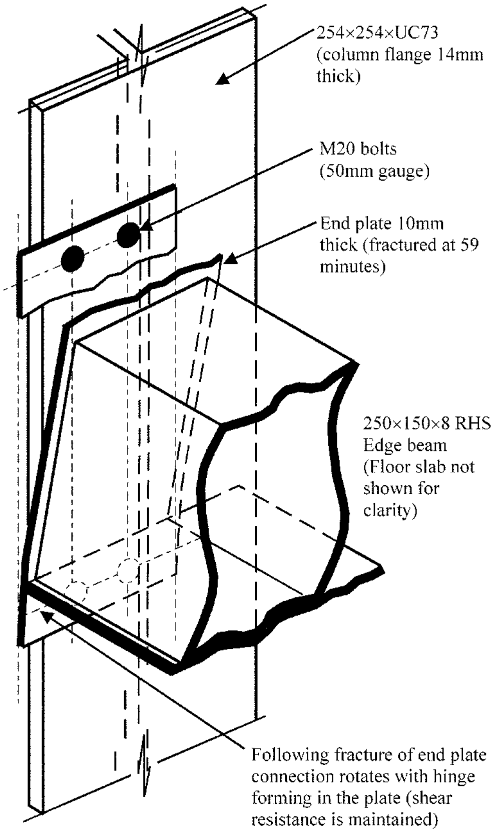

Fig. 11 Fracture of connection end-plate

Fig. 12 Fracture of end-plate (protection to outside of beam removed)

The ‘runaway’ displacement observed at locations V21 and V26 was found to be caused by fracture of the connection endplate between the beam on gridline 3 and the column on gridline B3. The connection consisted of a 10 mm thick extended endplate with four M20 bolts, with a 50 mm gauge width (Fig. 11). The endplate fractured across its full width, 10 mm above the top of the beam, as shown in Fig. 12. Evidence from the video recording indicates a large ‘bang’ occurring at 59 minutes after ignition, which could be associated with fracture of the endplate resulting in a significant increase in vertical displacement. Although the endplate fractured, vertical shear resistance was maintained with the connection transforming from an effective semi-rigid connection to a simply-supported connection, with rotation occurring in the end-plate between the bottom of the beam and the bottom row of bolts (Fig. 11). The behaviour of the beam as the connection fractured was, however, influenced by the ‘safety’ steel

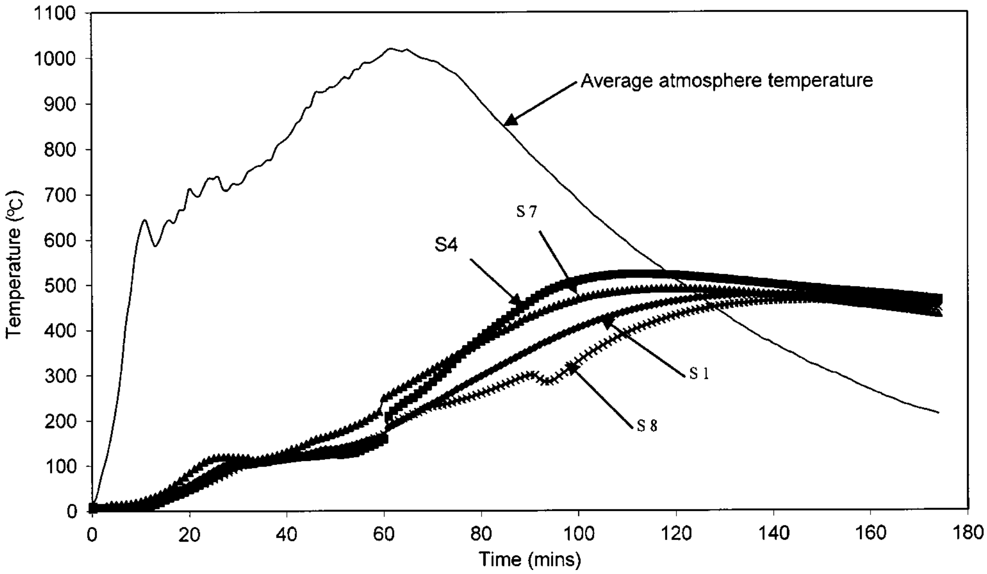

Fig. 13 Reinforcement temperature for 20 mm bar

Fig. 14 Picture showing damage to underside of the composite slab

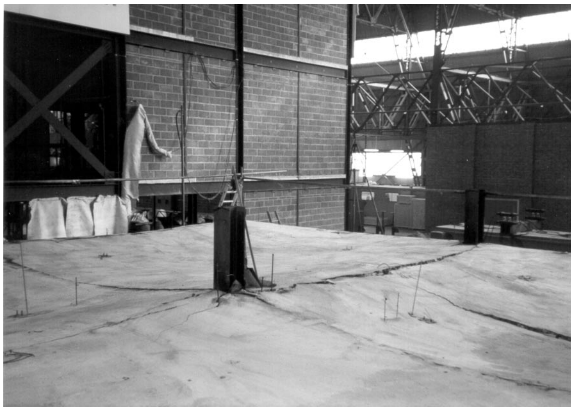

Fig. 15 Picture showing cracks on top surface of the floor slab after removal of sandbags

column placed inside the compartment and provided additional support when the slab reached a displacement of 500 mm.

The temperature of the 20 mm reinforcing bar, placed in the troughs of the slab, was measured at 8 locations, with four of the locations coinciding with the centre of each bay, as shown in Fig. 7. The reinforcement temperature recorded at the centre of each of the four bays is shown in Fig. 13. The reinforcement temperatures continued to rise after the maximum atmosphere temperature was reached, due to the high thermal conductivity of concrete. The maximum reinforcement temperature of 523 °C occurred at location S4. Using the design rules in ENV1994-1-2 (1994), a reinforcement temperature of 523 °C results in a 40% reduction in tensile strength of the bar.

The underside of the composite floor slab is shown in Fig. 14. The steel deck remained intact but had generally debonded from the concrete. Significant cracks were observed over the supporting beams on the surface of the floor slab, following removal of the sandbags and instrumentation, (Fig. 15). The cracks over the supporting beam are thought to be due to the large hogging moments occurring in this area. A significant crack was also observed running from gridlines 2 to 3 and positioned between gridlines B and C. This crack was caused by the high shear, experienced when the safety steel column provided support to the slab as it reached a vertical displacement of 500 mm.

Unfortunately, the test was limited in that the thermocouples measuring the temperature distribution through the beams malfunctioned during the test. However, since the atmosphere temperature in the test was reasonably close, during the heating stage, to the standard fire curve (Fig. 8), it is worth conducting some computer simulations, where the temperature gradient through the beams cross-section is assumed to be similar to that recorded from previous standard fire tests.

5. Modelling of the test

Modelling of the test was carried out using a purpose written finite element program, developed by the Author (Bailey 1999, Bailey *et al*. 1999). The asymmetric beams are represented within the model as one-dimensional elements, with 7 local degrees of freedom at each node. Geometric and material non-linearity is included within the model. The model has previously been validated to large displacements, both at ambient and elevated temperatures, including the accurate modelling of

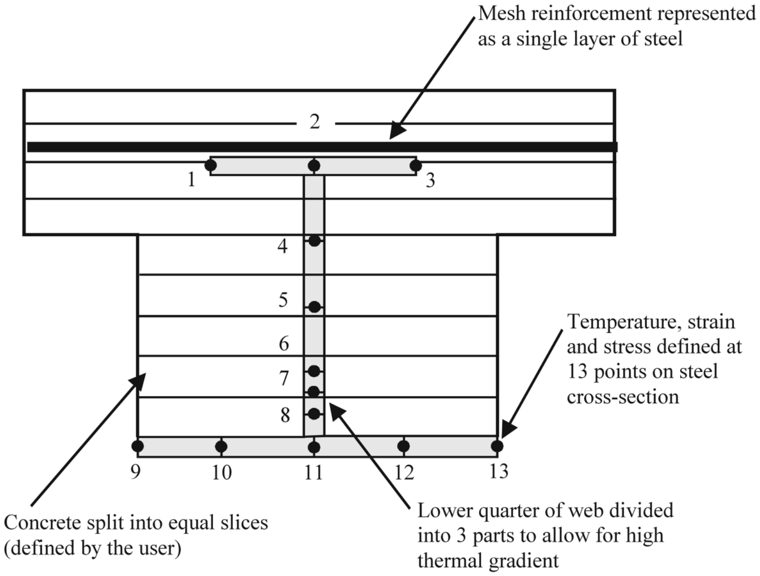

Fig. 16 Division of cross-section in segments for modelling

asymmetrical beams in a standard fire test.

To allow for the high thermal gradient through the asymmetric beam, the cross-section is divided into segments, with a greater number of segments in the lower quarter of the web (Fig. 16). The temperature, strain and resulting stress are defined at the 13 points shown in Fig. 16. The temperature-strain-stress relationship for the steel is defined (Bailey *et al.* 1996) using the Ramberg-Osgood representation (curve-fit) of the published test data on tensile steel specimens. The modelled composite cross-section assumes an effective width of span/8, corresponding to the conservative value defined in design guides (Lawson *et al.* 1997), which was derived following tests at ambient and elevated temperatures. The concrete component of the cross-section is divided into equal segments, which can be defined by the user. A previous parametric study (Bailey 1999) has been carried out, based on the results of two asymmetric beams tested in a standard furnace, to define the optimum number of segments for the concrete component. The temperature, strain and stress are defined at the centre of each concrete segment using the material model presented in ENV1994-1-2 (1994). It is assumed that the stress-strain-temperature relationship for concrete in tension is 10% of the compression values. Full composite action is assumed between the steel beam and concrete. Thermal expansion of the steel and concrete is included, using the relationships presented in ENV1994-1-2.

Semi-rigid connections can be modelled using spring-elements, with any user defined temperature-moment-rotation relationship. Parametric studies have previously been conducted (Bailey 1999) to highlight the beneficial effect of connections. This study has shown that including the behaviour of connections, which are assumed to transfer zero moment in ambient design, can be beneficial to the survival of the beam in a fire.

To model the asymmetrical beam along gridline 2, a two-dimensional slice was assumed through the building (Fig. 17). The temperature distribution through the cross-section, for a given atmosphere temperature, was obtained from the previous results on a similar type of beam tested in a standard furnace. This approach introduced slight errors into the modelling, since the time-temperature

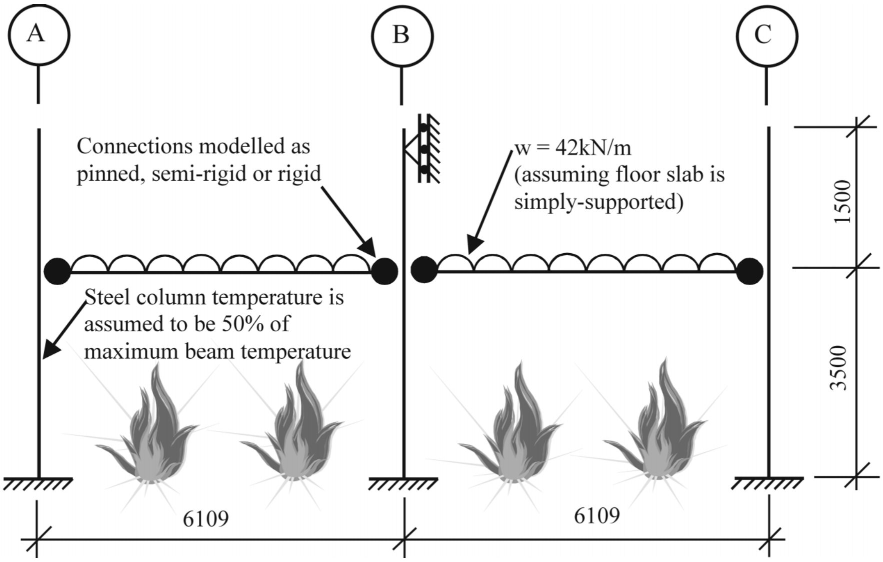

Fig. 17 Modelling of steel beams along gridline 2

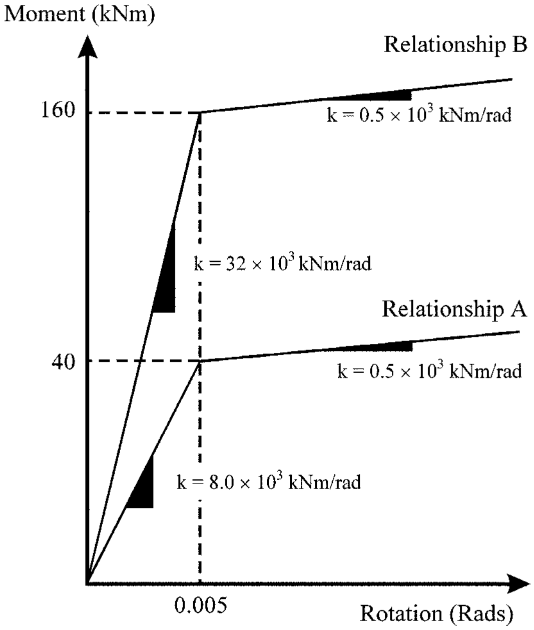

Fig. 18 Moment-rotation relationship

relationship between the large-scale test and standard fire test were not identical (Fig. 8). Computer simulations were conducted, to model the asymmetric beam along gridline 2, with the connections represented as ideally pinned, semi-rigid or ideally rigid. The moment-rotation characteristics for the semi-rigid connection are shown in Fig. 18, where relationship ‘A’ was derived from cold tests, as discussed by Bailey (1999), and relationship ‘B’ was defined, to investigate the sensitivity of the

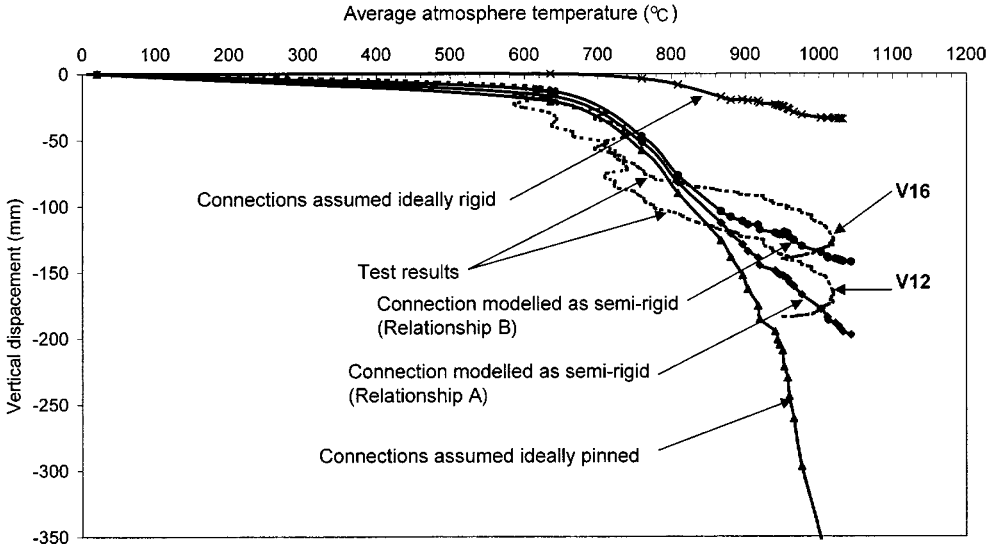

Fig. 19 Comparison between computer model and test results for vertical displacement of asymmetrical beams along gridline 2

connection behaviour, as an increase in initial stiffness of four times relationship ‘A’. No tests at elevated temperatures have been conducted on the behaviour of connections used for slim floor beams. It is, however, assumed that the moment-rotation relationship of the connection for slim floor beams is not influenced by temperature, which is a reasonable assumption since the entire connection is embedded within the concrete slab.

The predicted response of the vertical displacement at the centre of the two asymmetrical beams along gridline 2 is shown in Fig. 19. In the computer modelling, the temperature distribution through the beams cross-section was assumed to be constant along the beam, resulting in identical structural response of the two beams. As previously discussed the vertical displacement of the two beams along gridline 2 (locations V12 and V16, Fig. 7) was different, which is assumed to be caused by different temperature distributions. However, this assumption cannot be confirmed due to lack of test data. The predicted structural response (Fig. 19) shows that the connections were beneficial to the survival of the beams, and that the typically adopted design assumption of treating asymmetrical slim floor beams as simply-supported in fire conditions is significantly conservative. The accuracy of the modelling, compared to the test results, seems reasonable but definitive comparisons and thus conclusions on the models accuracy cannot be presented due to the lack of test data defining the temperature distribution through the cross-section.

It is worth presenting the modelling of the slim floor edge beam along gridline 3, between gridlines B and C, where the connection between the beam and column B3 failed during the test. The comparison between the model and test results is shown in Fig. 20, which suggests that the connections were significantly influencing the behaviour of the beam, with constant displacement between 800 and 1000 °C, until one connection failed and the connection characteristics changed from semi-rigid to pinned. The beam did not reach displacements over 190 mm, since the safety columns supported the centre of the slab once the end-plate of the connection fractured.

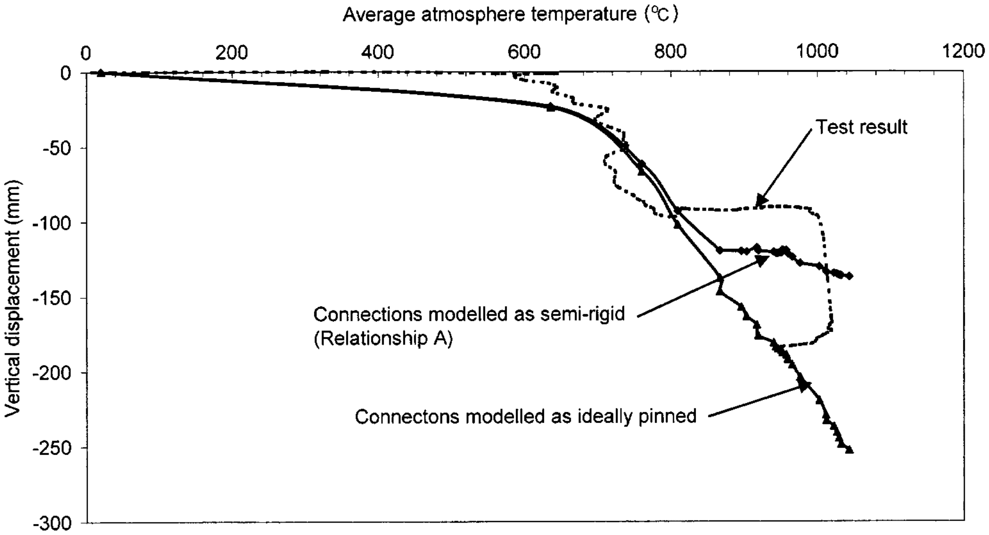

Fig. 20 Comparison between computer model and test results for vertical displacement of RHS slim floor beam along gridline 1

6. Discussion and conclusions

It is not possible to obtain definitive conclusions from the test, due to the malfunction of the thermocouples measuring the atmosphere temperature and member temperatures. In addition the 'safety' columns placed inside the fire compartment supported the slab once it reached 500 mm, after 62 minutes, effectively terminating the test.

The limited test results obtained, and subsequent computer modelling, showed that the beam-to-column connections, which were assumed to transfer zero moment in normal design, were significantly beneficial to the survival of the beams and the system as a whole. The effect of the connections could be included in future design allowing either the fire resistance period to be increased, or the beam section size to be reduced. The test, however, did show the fracture of an end-plate connection between a rectangular hollow section and a standard 'H' column. This effectively converted the connection from semi-rigid to pinned. It is thought that fracture of the connection would not have led to ultimate failure of the system, since vertical shear at the connection was maintained. However, this was not proved from the test since safety columns supported the floor slab once the vertical displacement exceeded 500 mm. If future designs are to rely on moment transfer through beam-to-column connections, at the fire limit-state, then the behaviour of the connection during a fire needs to be considered. No previous research has been conducted into connection behaviour for slim floor beams at elevated temperature, and this is an area that requires future investigation.

Acknowledgements

The Author would like to acknowledge the contribution from Mr Tom Lennon of BRE, Dr Stephen Hicks of SCI and Dr Brian Kirby of Corus who were also involved in the design, construction and execution of the test discussed in this paper.

References

Bailey, C.G., Burgess, I.W. and Plank, R.J. (1996), "Analysis of the effects of cooling and fire spread on steel-framed buildings", *Fire Safety Journal.* **26**, 273-293.

Bailey, C.G. and Newman, G.M. (1998), "The design of steel framed buildings without applied fire protection", *The Structural Engineer*, **76**(5), March.

Bailey, C.G. (1999a), "The behaviour of asymmetric slim floor steel beams in fire", *Journal of Constructional Steel Research*, **50**, 235-257.

Bailey, C.G., Lennon, T. and Moore, D.B. (1999b), "Full scale fire test on the new UK slim floor system", *Proceedings of The Second International Conference on Advances in Steel Structures*, 15-17 December, Hong Kong, China.

BS476 (1990), "Method for determination of the fire resistance of elements of construction: Part 20", London, British Standards Institution.

BS 5950 (1990), "Structural use of steelwork in buildings: Part 8: Code of practice for fire resistant design", London, British Standards Institution.

Cai, J., Burgess, I.W and Plank, R.J. (2002), "Modelling of asymmetric cross-section members for fire conditions", *Journal of Constructional Steel Research*, **58**, 389-412.

Eurocode 1 (2001), "Actions on Structures, Part 1.2: General Actions Actions on structures exposed to fire", Final Draft (Stage 49) prEN1991-1-2..

Eurocode 3 (1995), "Design of steel structures: Part 1.2: General rules, Structural fire design", ENV 1993-1-2, Brussels, European Committee for Standardisation.

Eurocode 4 (1994), "Design of composite steel and concrete structures: Part 1.2 General rules", Structural fire design, ENV 1994-1-2, Brussels, European Committee for Standardisation.

Lawson, R.M, Mullett, D.L and Rackham, J.W. (1997), "Design of asymmetric slimflor beams using deep composite decking", *SCI Publication P175*. The Steel Construction Institute, Ascot.

Mullett, D.L. (1997), "Design of RHS slimflor edge beams", *SCI Publication P169.* The Steel Construction Institute, Ascot.

DN