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The inherent fire resistance of a loaded steel framework

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ABSTRACT

This paper describes a fire test on a full size, fully loaded, two-dimensional, mainly unprotected steel frame which achieved an Equivalent Fire Duration of 32.5 min. The test, the first of its kind in Europe, and the most recent in a large series sponsored jointly by the BSC and DoE conducted in the FRS Cardington Laboratory, showed that the performance of the frame was better than that of the individual elements. This is attributed to the beneficial effect of beam/column action on the beam and to the effect of blocking in the web of the column. At the end of the test the maximum steel temperatures were 775 and 600 °C for the beam and column, respectively. Because of non-uniform heating, the beam and columns were subjected to thermal bowing; the paper includes a discussion of this aspect with reference to some simple questions validated by FRS.

INTRODUCTION

Theoretical studies backed by experimental evidence show that it is feasible to design in steel using high temperature properties. [1]. In essence, this involves the determination of the temperature changes in any particular member, or sub-assembly, for comparison with a limiting temperature based on the required load-bearing capacity.

The traditional method of assessing structural stability in fire is based on standard fire exposure in the BS 476: Pt 8: 1972 test. An acceptance criterion is that load-bearing elements should satisfy the required periods of fire resistance prescribed by the 1985 Buildings Regulations Approved Document B2/3/4. This approach is essentially a classification system but has shown that in a growing fire environment worthwhile periods of fire resistance can be realised in fully loaded (BS 449: 1969) unprotected steel members when due attention is paid to design detail. However, the BS 476: Pt 8 fire test does not attempt to represent the heating conditions in a real fire, where the growth and decay stages could produce temperatures significantly different from those in the standard fire resistance test.

Therefore, a collaborative test programme was initiated jointly by Swinden Laboratories of the British Steel Corporation (BSC) and the Fire Research Station (FRS) of the Building Research Establishment, DoE, to examine the behaviour of unprotected steel elements in a series of well-documented natural fires. [2]. A fire compartment was built in the FRS Cardington Laboratory for the purpose of this work. The concept of the Equivalent Fire Duration was used to assess the ability of a structural element to withstand a natural fire. This involved relating the maximum average steel temperature achieved by a particular section in a specific Cardington experiment to an equivalent heating time in the BS 476: Pt 8 fire resistance test. The Equivalent Fire Duration is employed in fire engineering calculations.

Many buildings, such as ground and upper storeys in office, shop, factory, assembly and storage buildings up to 7.5 m in height may only require a 30 min fire resistance. In view of the options available for achieving a 30 min fire resistance with individual unprotected steel elements, the opportunity was taken at Cardington to establish whether their combined effect in a fully loaded structural framework would result in a similar improvement. The principal objective of this test, thought to be the first of its kind in Europe, was to generate data of value for the preparation of design guidelines and for use in the complementary development of analytical techniques to simulate the structural stability of steelwork in natural fires.

This paper describes a natural fire test on a fully loaded, two-dimensional framework. The structure comprised an unprotected steel beam which spanned the compartment at ceiling height and was attached at each end by bolted connections to blocked-in steel columns. The fire load comprised timber cribs and the ventilation openings were selected to ensure that the Equivalent Fire Duration exceeded 30 min.

THERMAL BOWING

In the experiment described, the steel members were subjected to non-uniform heating causing differential expansion and thermal bowing. In a flexural element the deflection due to thermal bowing alone can damage non-load-bearing partitions beneath it, or prematurely exceed the BS 476:Pt 8 deflection criterion. Thermal bowing in a column could induce eccentric loading leading to early failure. While there has been some investigation of the thermal response of such elements, there had been limited research into the structural response.

The rigorous analysis of the behaviour is complicated due to geometric and material non-linearities occurring simultaneously as heating proceeds. However, the FRS has developed a simplified theory which, to a first approximation, can be used to analyse thermal bowing [3].

The behaviour is related to equations that are based purely on geometry and assume bowing in a circular arc; they do not take account of deflections due to out of balance internal stresses arising from curvilinear temperature profiles across the section, or phase transformation. The equations for a non-loaded, simply supported beam and a fixed-base column are given in Fig. 1, from which can be seen that linear displacements are proportional to the square of the member length and inversely proportional to the depth of section. The equations have been validated with experimental data obtained from model steel beams heated electrically and a full size steel column in the BSC/FRS Cardington compartment in which temperature profiles were indeed curvilinear.

An additional and important objective of the natural fire test was to relate the observed deflection behaviour

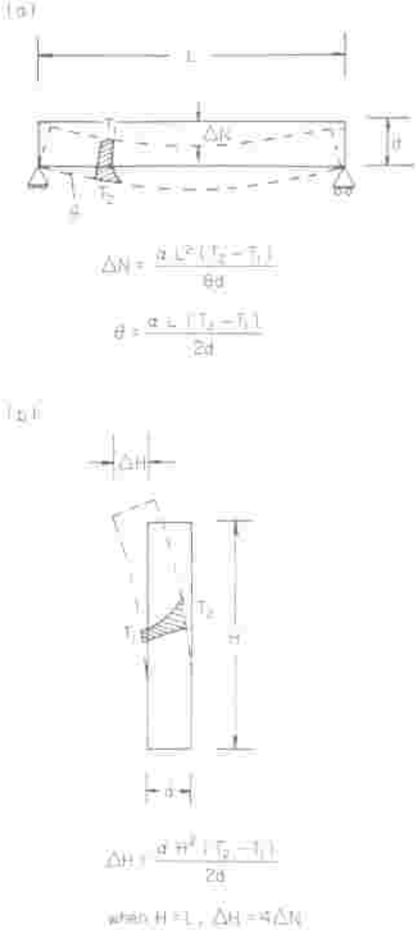

Figure 1. Thermal bowing relationships. (a) Simply supported beam and (b) fixed base column.

of the fully loaded two-dimensional steel framework with the simplified theory proposed by the FRS. In addition, the measured data would be used at a later date to validate a more rigorous mathematical analysis, such as by the finite element method.

THE CARDINGTON EXPERIMENT

Design of the compartment

The BSC/FRS compartment was built for elevated temperature studies of a number of individual columns and beams having different serial sizes. These steel members were either unprotected or partially protected against fire. Combustion gas temperatures could reach 1200 °C in the experiments. The floor area of 50 m² and ceiling height of 3.9 m was typical of an office in a multi-storey building. For the purposes of this present exercise, the individual steel sections had been removed with the exception of two partially built-in columns. A non-loaded 1.6 m length of unprotected

Figure 2. Front elevation of compartment—fire test in progress.

203 × 203 mm × 52 kg/m column was used to compare its heating rate with earlier fire tests, and is referred to as an indicative specimen in this paper. The front elevation of the compartment is shown in Fig. 2. Ventilation was provided by means of shutters placed within the long walls and was restricted to one-eighth of the total area on each wall to obtain as symmetrical a heating exposure across the loaded frame as possible.

The constructional materials had originally been selected so that they would withstand repeated exposure to high temperature. The inner leaf of the cavity walls comprised heat resisting brickwork whilst the concrete floor had a top screed of insulating concrete. The assembly of the frame necessiated the replacement of insulated roof slabs by 200-mm-thick precast concrete slabs that had already been in a fire test. It was recognised that the low thermal conductivity of the walls, not normally found in modern buildings, would raise the maximum average combustion gas temperature. Earlier tests had shown, for example, that the introduction of a plasterboard lining to the walls reduced measured combustion gas temperatures by as much as 150 °C [2].

Steel framework

Section sizes. The steel framework selected for testing under load was typical of that used in a building of two or three storeys in height. It comprised a 4.55 m length of 406 × 178 mm × 54 kg/m (7.6/10.9* mm) BS 4360 Grade 43A universal beam section and two 3.53 m lengths of 203 × 203 mm × 52 kg/m (9.2/12.5 mm) Grade 43A universal column section. These serial sizes of section were chosen because they had previously been included in separate BS 476:Pt 8 fire resistance tests, culminating in the following observations.

The heating rate of a steel member is quantified by the Hp/A ratio, where Hp is the perimeter of the steel exposed to heat, and A is the cross-sectional area. The standard fire tests have shown that on fully loaded, simply supported beams exposed to fire attack on three sides, an Hp/A value of about 100 per metre is necessary to obtain a 30 min fire rating at which point the temperatures in the lower flange at the limit of deflection (span/30 on a 4.5 m span) are around 630 °C. As the 406 × 178 mm × 54 kg/m has an Hp/A ratio of 190 per metre it heats up more rapidly. However, as shown in Table 1, the imposition of rotational end restraint increases the fire resistance of this member to 30 min.

The corresponding Hp/A ratio for free-standing fully loaded unprotected columns is less than 50 per metre and the mean temperature at which the column can no longer support the load is 580 °C; this means that a 203 × 203 mm serial size of column would require a flange and web thickness of 45 mm to satisfy the requirement. A relatively cheap method of improving the fire resistance of lighter/free-standing columns is to block-in the volume between the flange and the web with conventional lightweight concrete building blocks [4]. Standard fire tests have demonstrated that fully loaded (BS 449) 203 × 203 mm × 52 kg/m columns will achieve 30 min fire rating by this method with no load carrying contribution from the blocks, and typical results are also included in Table 1.

Design. The complete assembly shown schematically in Fig. 3, comprised the test frame surrounded by load reaction frames which gave a closed loading system so that only dead loads were transmitted to the floor of the laboratory. The assembly was held in position by a combination of external bracing and a subsidiary steel framework contained within the compartment designed to prevent lateral and sway instability. The test frame, shown in more detail in Fig. 4a, was centrally positioned inside the compartment parallel to the short walls.

Each column, which extended above the beam, was pin-jointed at the base (Fig. 4b) by means of a 60 × 150 × 25 mm bearing strip, fixed to a 1.5 m length of 533 × 210 mm × 92 kg/m section which also formed the lower load spreader of the column load reaction frame. The webs of each column were protected by autoclaved aerated concrete blocks with a density of 677 kg/m^3 (3.8% water content by weight) built between the flanges using an ordinary mortar mix followed by a 28-day drying out period. The test beam remained unprotected, but four 1200 × 5550 × 150 mm precast concrete slabs, which formed part of the compartment roof, were attached to the top flange by welded 12 mm-diameter threaded bars. The slabs were separated by a gap of 25 mm to prevent composite action with the beam, and the gap was filled with ceramic fibre blanket.

* Web and flange thickness, respectively.

Table 1. Fire resistance of steel beams and columns loaded to maximum allowable stress (BS 449: Pt 2: 1969)

Section	Exposure (sides)	Hp/A (per m)	Condition	Fire resistance (min)
406 × 178 mm × 54 kg/m	3	190	Unprotected	20
			30% rotational restraint*	30
203 × 203 mm × 52 kg/m	4	180	Unprotected	15
		85	blocked-in-web	36

*This means that 30% of the maximum free bending moment is applied at each end of the beam.

The test frame was designed to the maximum allowable stresses in accordance with BS 449: Pt 2: 1969. The test beam was assumed to be simply supported but laterally unrestrained due to the slab construction adopted. The test columns were considered to be pin-jointed at the top and bottom and the eccentric loads due to the beam connection and the straightness of the columns were taken into account*. The beam/column bolted connections (Fig. 4c) were designed to resist the shear forces and it was assumed that tensile forces in the bolts generated by the bending moment could be transmitted; however, M20 Grade 4.6 bolts were replaced by Grade 8.8 counterparts to provide improved resistance to loss in strength at elevated temperature.

The load reaction frame comprised a 533 × 210 mm × 92 kg/m universal beam as a crosshead, each end of which was attached to two vertical 152 × 89 mm channel section tie members that straddled the blocked-in columns and were fixed to the lower load spreaders, which were themselves braced and bolted to the concrete floor. A maximum axial compressive load of 552 kN was applied to each test column by an hydraulic ram and load cell placed between the top bearing plate and the upper load spreader of the column reaction frame. The test beam was loaded to 39.6 kN at four positions along the span using two jacks and two load spreaders in a similar manner.

The test loads on the beam and column were maintained constant throughout the fire test. With the exception of the test frame the remainder of the structure inside the compartment and lengths of the external braces in front of the ventilation openings were fire protected.

Instrumentation

The distribution of the combustion gas temperature in the compartment during the fire was monitored by 3 mm diameter chromel/alumel thermocouples with Inconel sheaths and insulated hot junctions. The measurements were concentrated at various positions around the test frame with the thermocouples placed between 100 mm and 300 mm from the steelwork. Combustion gas temperatures were also measured at locations half way between the test frame and each of the short walls of the compartment.

Similar thermocouples were used to measure the steel temperature profiles in the flanges and webs of the test sections at five different heights on the columns and nine positions along the beam. The tip of each thermocouple was inset into the steel section by 3 mm. The temperatures of the top and bottom bolts of the beam/column connection were monitored with thermocouples inserted to a depth of 22 mm from the face of the exposed bolt head and 12 mm from the ends of the threads that were surrounded by mortar and blockwork.

Deflection measurements (Fig. 5a) were made at the centre of the loaded beam using two separate potentiometric transducers mounted independently of the test frame on the roof of the compartment. A similar device was located between the crosshead and the top of one blocked-in column. Three linear displacement transducers with shaft extensions were installed to follow the lateral bowing of each blocked-in column at three heights, also shown in Fig. 5b.

*The frame had previously been subjected to a proving test involving a less severe fire.

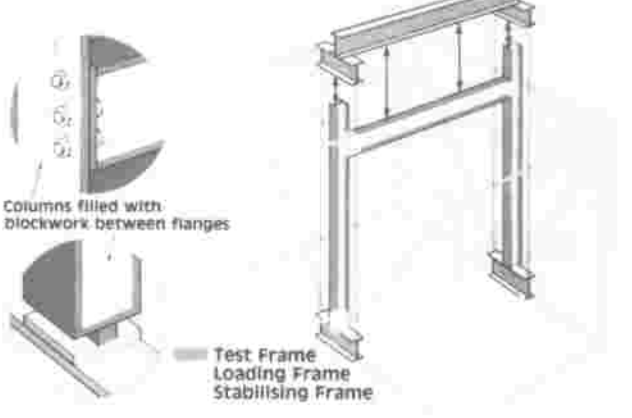

Figure 3. Schematic layout of the loaded frame used in the Cardington test.

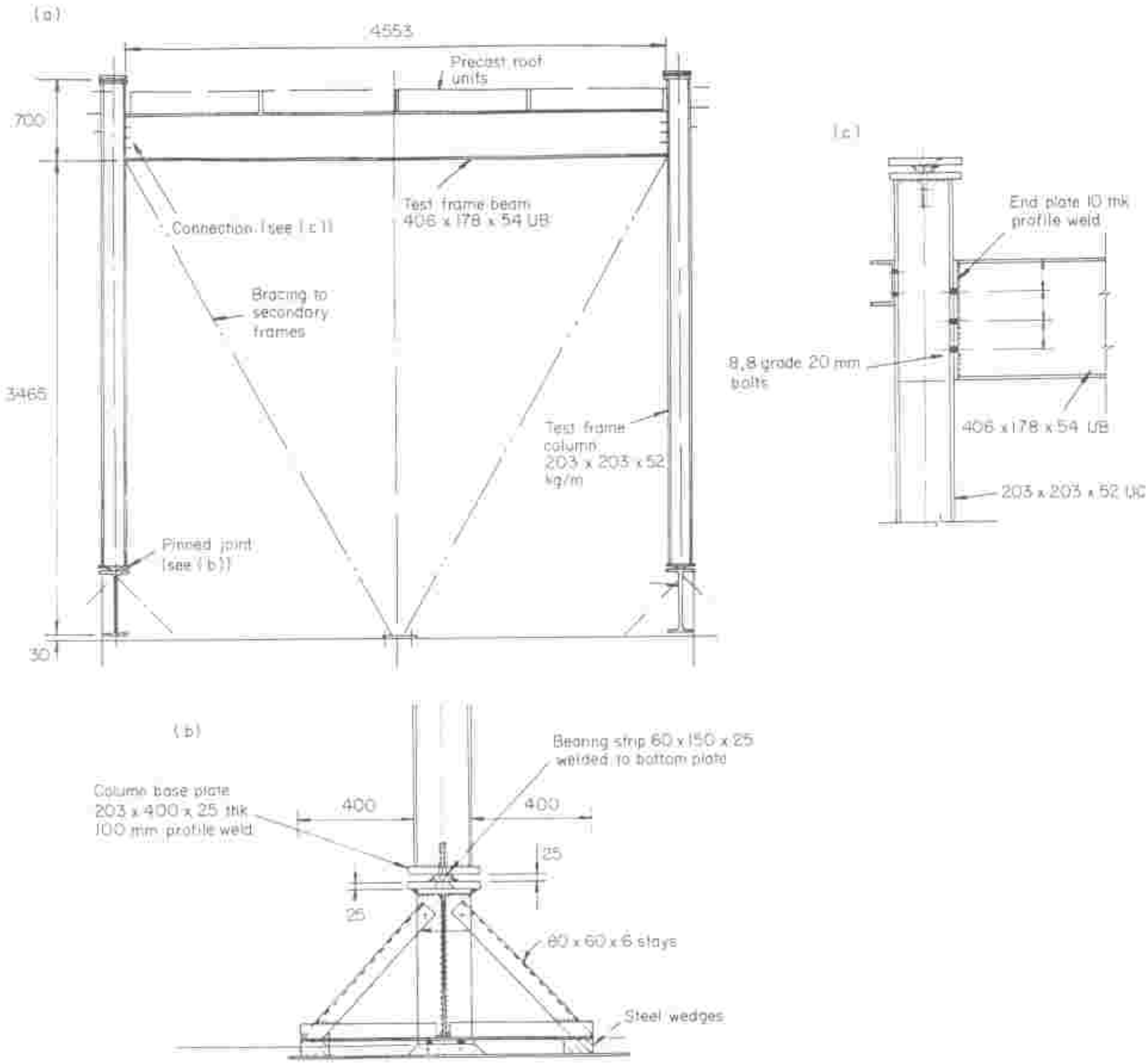

Figure 4. Principal features of test frame. (a) Front elevation, (b) column joint detail and (c) connection detail.

The outputs from the separate measuring devices were fed back to a Compulog 4 computer controlled data acquisition system.

Fire load and ventilation

The combustion of wood with a fire load density of 25 kg/m^2 with the one-eighth ventilation of two walls in the compartment was considered to be of sufficient severity to ensure that the loaded beam would reach its limiting temperature of 630 °C during the fire. This decision was based on earlier studies of the temperatures attained by unprotected steelwork in natural fires.

The total fuel load of 1320 kg comprised 50 × 50 mm × 1 m softwood sticks having an average moisture content of 15% which were divided into 12 equally spaced cribs. It was considered advisable to control the rate of burning to establish a realistic temperature distribution over the cross-section of the blocked-in columns. This was achieved by spacing the sticks 50 mm apart with alternate rows at right angles to each other, but with the third layer from the bottom replaced by 100 × 100 mm bundles placed 100 mm apart to reduce the total surface area of the fuel, so as to reduce the rate of burning and increase the fire duration.

FIRE SEVERITY

The wooden cribs were ignited simultaneously at an ambient temperature of 0 °C and the fire increased in intensity until a maximum temperature of 830 °C was recorded by the thermocouples at the centre of the compartment after 17 min. At this time the average temperature in the vicinity of the columns was 758 °C

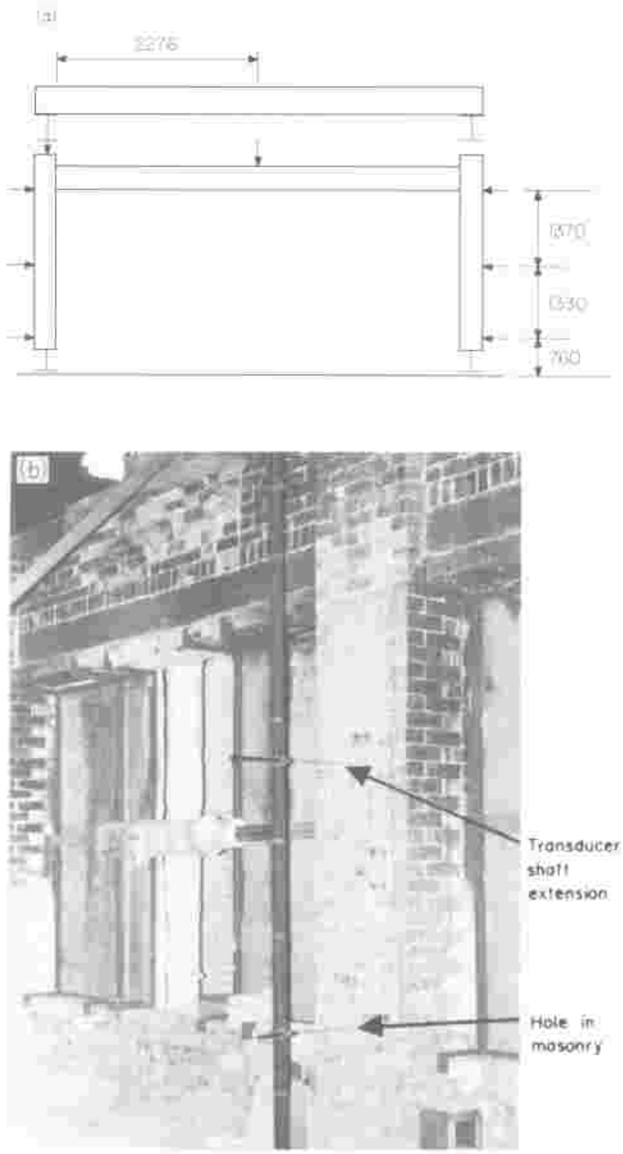

Figure 5. Measurement of deflections. (a) Measurement positions and (b) monitoring horizontal column.

at a height of 3300 mm, 800 °C in the middle position and 780 °C at the 1200 mm level. The development of the combustion gas temperature with time along the beam is represented in Fig. 6 for a number of time periods throughout the test. As a result of equal ventilation from both walls of the compartment, the hottest region of the fire occurred at the centre of the beam. Once the peak temperature had been reached the heating became more uniform.

The combustion behaviour depends upon the balance between the rate at which heat is produced in a fire and lost from the compartment. The BSC/FRS study confirmed and extended the results of other investigations [5] which showed that fire severity increased with the fire load density and with a decrease in the extent of ventilation.

A recent approach for estimating the Equivalent Fire Duration (Te) in the BS 476: Pt 8 test, by making an allowance for the thermal properties of the boundary surfaces, has been prepared [6]. The relationship is Te = $q_t cw$ (min) where, q_t is the fire load density in MJ/m^2 of compartment bounding surface; c is a conversion factor related to the thermal properties of the boundary materials and w, is a ventilation factor which allows for the shape of the opening. The conversion factor, c takes one of three specific values depending upon the thermal inertia of the compartment. As $q_t = 462.5$ MJ/m^2, $w = 0.97$ and c was calculated to be 0.07 min/(MJ/m^2), the relationship predicted an Equivalent Fire Duration of 31.4 min.

STEELWORK TEMPERATURES

The steel temperature profiles recorded from the loaded test frame are typified by the curves of Fig. 7. The heating rate was fastest at the centre of the unprotected beam. Maximum temperatures of 775, 777 and 577 °C were measured in this locality in the lower flange, centre of the web and upper flange, respectively, after 20 min. The corresponding temperatures in the lower flange and web close to the connections were 671 and 702 °C, the web heating up more rapidly since it was thinner than the flange. The maximum temperatures in the steel occurred approximately 3 min after the combustion gas temperatures reached their peak due to out-of-phase radiation from the flames and heated surfaces of the compartment in the decay stage of the fire.

With regard to the blocked-in columns, the exposed flanges facing into the compartment (Fig. 8) heated up faster than the exposed flanges facing towards the walls and this is mainly due to the difference in the radiation configuration factor. Thus for one column, a maximum temperature of 606 °C was measured on the inward facing flange after 20 min, by which time the outward facing flange reached 514 °C. Due to the protection provided by the blockwork, the centre of the web only attained a temperature of 251 °C after 20 min.

The time equivalent concept can be equally applied to relate the maximum temperature achieved by a steel member in a natural fire to the time taken to reach the same temperature in the standard test. This is normally carried out by graphical means. For example, a maximum average steel temperature of 768 °C was recorded on the indicative column (Hp/A = 180 per metre), which is equivalent to about 33 min exposure in the standard fire (adjusted to an ambient temperature of 20 °C). An alternative approach, where BS 476: Pt 8 experimental data are not available is to predict the heating rate of the member by mathematical modelling. This simulation is now possible for unprotected steelwork. On this basis the time taken by the

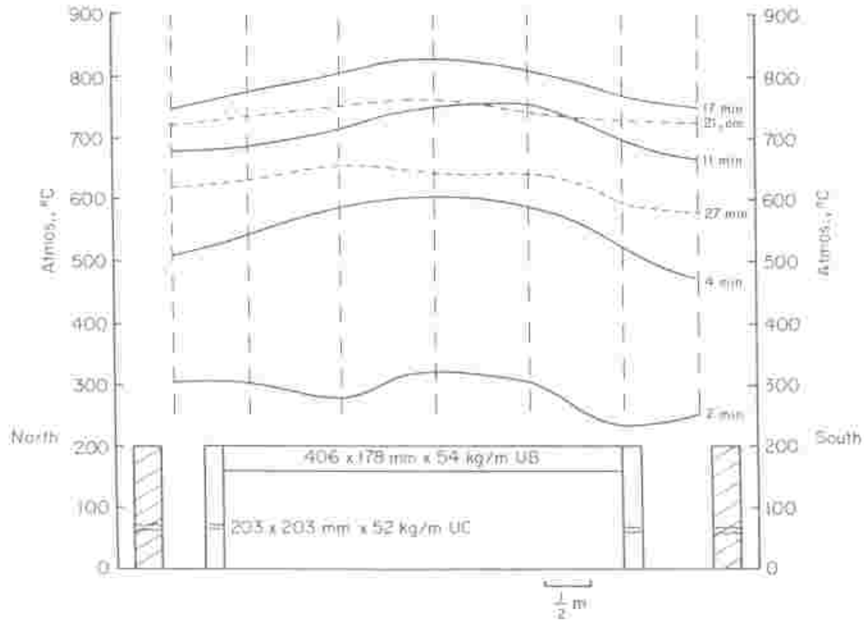

Figure 6. Combustion gas temperatures established in the vicinity of the beam.

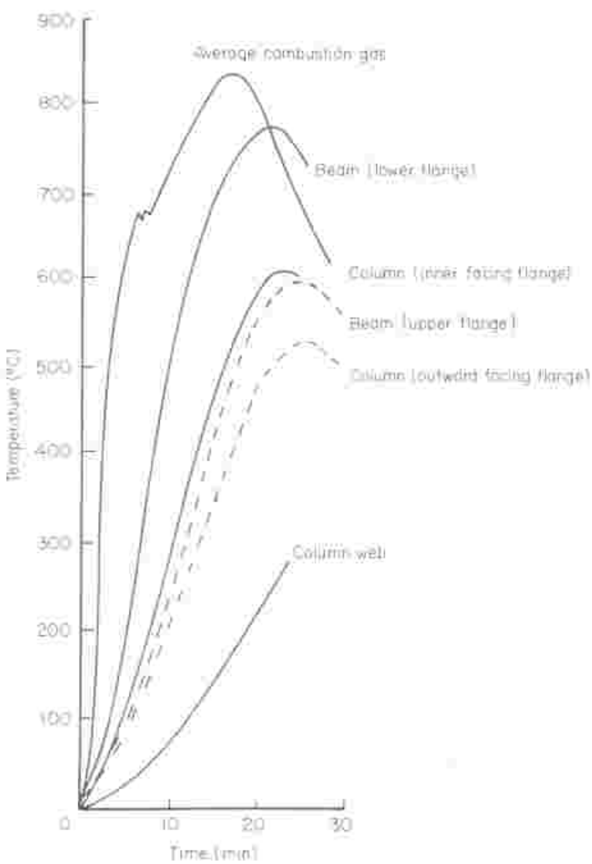

Figure 7. Steel temperatures at centre of beam and at mid-height of blocked-in columns.

406 × 178 mm × 54 kg/m beam in the standard fire to reach the maximum average temperature recorded in the Cardington fire test was calculated to be 32 min. These observations all emphasised that the test framework satisfied a 30 min fire rating, thereby indicating the potential benefit to fire resistance from structural continuity.

The utilisation of rotational end restraint to improve the fire resistance of steel beams necessitates that the connections have adequate performance during fire. Uniaxial tensile tests carried out by BSC at elevated temperatures showed that for ISO metric precision bolts with a strength Grade 8.8, the maximum allowable working stress in tension (or shear) would be

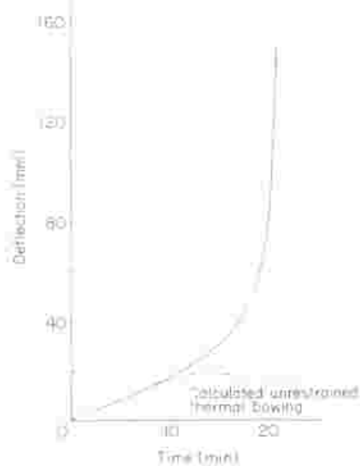

Figure 8. Central vertical deflection of beam.

sustained at temperatures below atbout 550 °C. The load was removed from the structure after 22 min into the natural fire test. At this time the temperatures reached by the thread beneath the bolt heads were 397 °C for the upper and 441 °C for the lower bolt. The reduction in temperature along the thread which extended into the blockwork was approximately 100 °C. Apart from slight distortion of the end plate the integrity of the connection showed no requirement for additional fire protection. The benefit afforded by protecting the webs of the columns with blockwork was also clearly apparent in the real fire situation—a BS 476: Pt 8 fire resistance test on an identical column but without blockwork gave a fire resistance of 15 min.

DEFLECTION BEHAVIOUR

The deflection of the structure was more complicated than the behaviour of isolated elements due to the effect of structural continuity and the non-uniform fire exposure which caused thermal bowing. The variation of downward mid-span deflection of the beam with time is shown in Fig. 8. The rate of deflection increased up to approximately 40 mm/min. with the rise in steel temperature. The load was removed after 22 min when it could no longer be applied with safety. At this point the total deflection of the beam exceeded (span/32). The theoretical deflection due to thermal bowing is also included in Fig. 8.

At failure the beam exhibited considerable twisting as well as vertical deformation (Fig. 9), together with tilting of the concrete slabs attached to the upper flange. Subsequent examination revealed the presence of a plastic hinge approximately 600 mm from each end of the beam and some plastic distortion of the welded end plates at the top of the connection, where the bolts were in tension. If the beam had been connected to a normal floor system such twisting would not be expected to occur and a longer period of stability could result.

The blocked-in columns expanded axially to reach a maximum extension of 20 mm after 15 min. The distance between the columns increased during the test due to the axial expansion of the beam and rotation of the ends of the beam due to thermal bowing. The average lateral displacements measured on the columns at different heights with time are shown in Fig. 10. The unrestrained theoretical deflections due to thermal bowing are also included for comparison.

The FRS theories for calculating the angular rotation and lateral deflection of steel members heated non-uniformly can be related to the degree of fixity imposed on the frame. The concepts are illustrated schematically in Fig. 11.

Consider, for example, a non-loaded single bay portal frame having rigid beam/column connections in which the inner flanges of the two columns and beam are hotter than the outer flanges (as in the Cardington portal frame test). If the columns were not position or direction-fixed (a hypothetical condition), the deflected form would be as in Fig. 11a.

If the column feet are now position-fixed but not direction-fixed (as in the Cardington portal frame test)

Figure 9. Condition of the test beam on completion of the fire test.

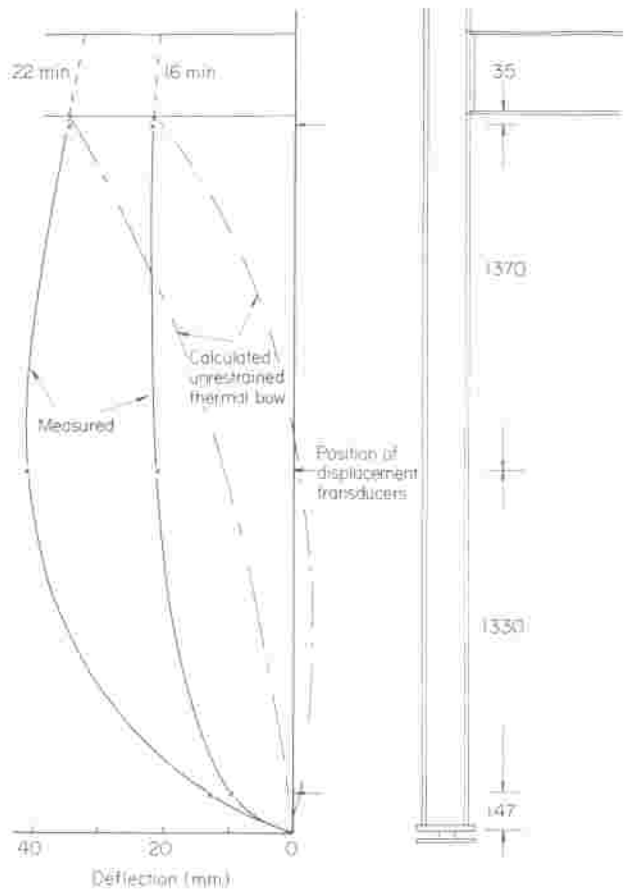

Figure 10. Average lateral deflection of column in fire test.

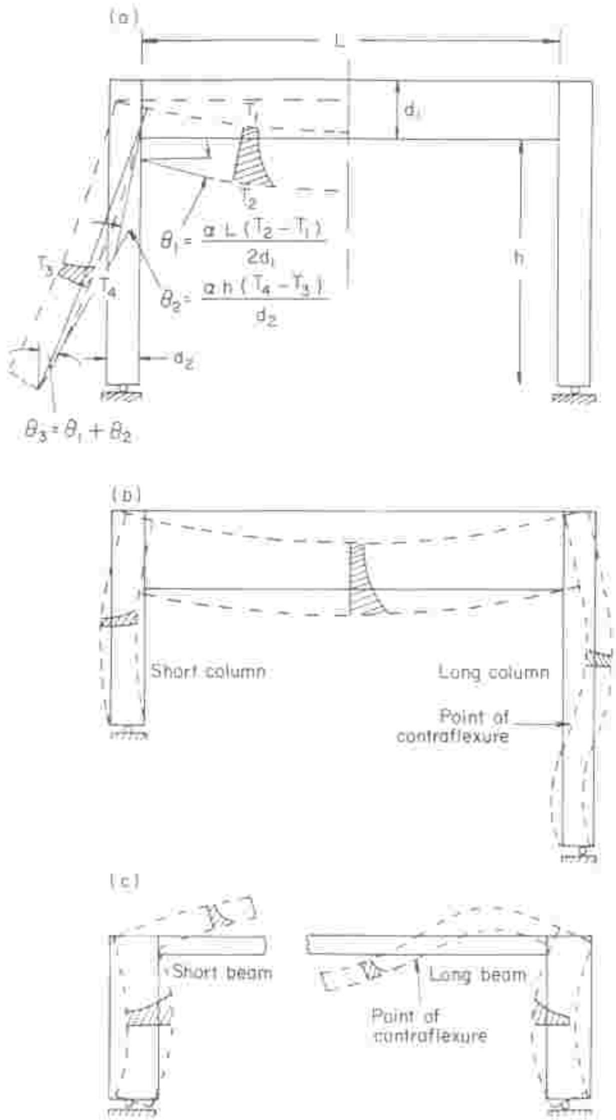

Figure 11. Deflection shapes of portal frame due solely to thermal bowing. (a) Unrestrained column feet, (b) beam dominant, column feet position fixed and (c) column dominant, column feet position fixed.

the deflected shape will depend on the relative stiffness (EI) and length of the beam and columns. If the beam is thermally dominant (i.e. it has a large EI/L and $(T_2-T_1)/d$ values) the deflected shape would be as in Fig. 11b. A point of contraflexure may occur in a long column. Figure 11c shows the deflected shapes for the column-dominant condition for long and short beams. Whether a member is dominant or not (or remains so as heating continues) will depend not only on its EI/L and $T_2-T_1)/d$ values but also on which member first becomes plastic such that a plastic hinge forms, and on the nature and magnitude of the imposed loads.

A comparison of the total frame displacement measured during the Cardington fire test with the thermal bowing calculations is illustrated in Figs 8 and 10. In the initial stage of the test (when the beam had not suffered extra deflection due to reduction in modulus) the predicted thermal bowing of the beam corresponded closely with measured values. Later, the beam became thermally dominant (Fig. 11) forcing the columns to bow outwards as it deflected downwards.

CONCLUSIONS

- A steel frame comprising an unprotected 406 × 178 mm × 54 kg/m beam bolted to two blocked-in columns and subjected to its maximum design load (BS 449: 1969) had an Equivalent Fire Duration of 32 min in the natural fire test. The response of the structure was more complex than that of the isolated elements. Bowing was clearly observed in the columns and the beam exhibited twisting as well as vertical deformation. The observed survival time was greater than that expected from the individual steel beam thereby indicating the potential benefit from continuity. The attainment of a 30 min fire resistance for the frame means that such construction could meet the provisions set out in Approved Document B/2/3/4 issued under the 1985 Building Regulations for ground and upper storeys in office, shop, factory assembly and storage buildings up to 7.5 m in height, and in non-institutional residential buildings up to two or three storeys, depending on floor area.
- The simple beam-to-column connections using a flush end plate on the beam and six Grade 8.8 bolts provided sufficient strength and stiffness to maintain adequate continuity throughout the test. It is possible that the continuity was enhanced by a combination of general expansion and thermal bowing. The connections provided 30 min fire resistance without the need for additional fire protection.
- The behaviour of the blocked-in columns appeared to be unaffected by the end moment, and the autoclaved aerated concrete blocks were not dislodged.
- The first test emphasised the importance of thermal movement when considering the structural response of frameworks in fire particularly where non-uniform heating occurs. The thermal bowing equations proved to be useful diagnostic tools in structural analysis demonstrating, for example, the thermal dominance of the beam over the blocked-in columns.
- It is thought that this is the first test in Europe in which a full size loaded frame has been tested in a fire compartment.
- Because of the complex nature of the frame behaviour in fire, mathematical modelling of this situation is at the limit of current analytical capabilities. As further work progresses in this area the Cardington results will be of value in refining and validating the computational procedures.

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