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Performance of steel beams at elevated temperatures under the effect of axial restraints

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Abstract. The growing use of unprotected or partially protected steelwork in buildings has caused a lively debate regarding the safety of this form of construction. A good deal of recent research has indicated that steel members have a substantial inherent ability to resist fire so that additional fire protection can be either reduced or eliminated completely. A performance based philosophy also extends the study into the effect of structural continuity and the performance of the whole structural totality. As part of the structural system, thermal expansion during the heating phase or contraction during the cooling phase in most beams is likely to be restrained by adjacent parts of the whole system or sub-frame assembly due to compartmentation. This has not been properly addressed before. This paper describes an experimental programme in which unprotected steel beams were tested under load while it is restrained between two columns and additional horizontal restraints with particular concern on the effect of catenary action in the beams when subjected to large deflection at very high temperature. This paper also presents a three-dimensional mathematical modelling, based on the finite element method, of the series of fire tests on the part-frame. The complete analysis starts with an evaluation of temperature distribution in the structure at various time levels. It is followed by a detail 3-D finite element analysis on its structural response as a result of the changing temperature distribution. The principal part of the analysis makes use of an existing finite element package FEAST. The effect of columns being fire-protected and the beam being axially restrained has been modelled adequately in terms of their thermal and structural responses. The consequence of the beam being restrained is that the axial force in the restrained beam starts as a compression, which increases gradually up to a point when the material has deteriorated to such a level that the beam deflects excessively. The axial compression force drops rapidly and changes into a tension force leading to a catenary action, which slows down the beam deflection from running away. Design engineers will be benefited with the consideration of the catenary action.

Key words: fire engineering; fire resistance; steel beams; fire tests; with resiseants; fire element analysis; catenary action.

1. Introduction

The prescriptive approach to the design of structural steelwork for fire resistance, which involves the application of a prescribed thickness of fire protection material to limit steel temperatures within required fire resistance periods, is progressively being replaced by a fire engineering approach. This requires the designer to calculate the response of loaded structural elements to increasing temperatures, allowing fire to be considered as one of the limit states for which the structure is initially designed (Allam *et al*. 1998).

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The fire engineering approach demands a better understanding of the factors that govern the behaviour of both individual steel members and the structure as a whole in fire conditions. This consideration of fire resistance by design and construction is beginning to be a normal method used in building codes to achieve fire safety. It should take into consideration the fact that the basic objective of structural fire safety can be satisfied only if control of the movement of the fire is achieved and if structural stability is provided. When subject to fire, an unprotected steel structure will lose its stiffness and strength as a result of deterioration in its material properties.

The majority of medium-rise steel-framed buildings at present use non-composite pre-cast concrete floor slabs carried by steel beams, with connections designed to act simply. In the event of fire, it has been shown in the recent tests on the Cardington (England) full-scale test frame (Armer 1994) and in observations from real fires (Thomas *et al*. 1992) that connections which were assumed to be pinned at ambient temperature could provide considerable levels of both strength and stiffness at elevated temperatures and hence they had a beneficial effect on the survival time of the structure. In a previous recommendation (Lawson 1990) it has been suggested that the limiting temperature of a “simply” supported beam can be enhanced by more than 100°C depending on the details of the supporting connections.

Although most beams act essentially as independent elements at ambient temperature, in fire conditions their behaviour is highly affected by the presence of the cooler adjacent parts of the structure. Observed effects from tests such as those at Cardington include local buckling near to the connections when a beam tries to expand during heating and connection fracture when it tries to contract during cooling. Some tests have been carried out on isolated connections (Leston-Jones *et al*. 1997). However, it was believed that these tests, though providing valuable information about the behaviour of the connection in fire, do not represent the actual behaviour of connections that were found in the Cardington frame tests because they have been conducted without the axial forces generated by the existence of adjacent structure in fire.

Current design codes such as BS5950 Part 8 (1990) and the drafted Eurocode 3 (1994) allow designers to take advantage of the most recent developments in the field by treating fire related loading as another Limit State. The advances in understanding of steel structure behaviour in fire achieved in the last few years have been considerable. The concept of performance-based design comes a long way to change from one which only considers individual elements to one which includes the consideration of fire resistance of the overall structural totality. For instant, the limiting temperature of an unprotected steel beam is only related to the applied load ratio it is subjected at fire limit state. No consideration has been made relating to the conditions of its supports and adjacent structures.

Because of the restricted cost of carryingout real fire tests on full-scale structural system on the one hand, and computational advances in structural analysis on the other, analytical methods are now accepted as alternatives for determining the behaviour of structures in fire. These analytical methods should reflect an accurate prediction as possible by taking into consideration the factors governing the behaviour of the steel element in fire.

In theory, these advances make it possible for designers to treat the design for fire in an integrated manner with the design of a structure for all other types of loading by using the numerical modelling tools that have been instrumental in developing this understanding.

Theoretical modelling has increasingly been used to simulate the behaviour of continuous structures at elevated temperature. There are a number of numerical models (Burgress 1990, Saab and Nethercot 1991), which can predict the behaviour of steel structures with semi-rigid connections at elevated temperatures. These are useful for the study of overall frame behaviour during a fire. The moment re-distribution within the frame depends largely on the connection characteristics. Recent experimental tests on small scale specimens Leston-Jones *et al*. (1997) showed that it is possible to derive accurately the moment-rotation relationships at elevated temperatures for sub-assemblages. The development of the sophisticated tool by Liu (1999) also extended it to a full scale temperature-moment-rotation characteristic of connection which would be readily applicable to the overall frame analysis. Nevertheless, similar to the case of the experimental advances, the behaviour of connection in fire is not fully understood without the inclusion of the axial effect generated by the expansion of the connecting beams (Bailey 1996 and Liu 1996). In particular, the study of the interaction between connections and the connecting members has very little been mentioned.

The experimental program to be reported in this paper on the restrained beam tests provides almost for the very first time a controlled fire test on a quasi-full-scale structural system, for the study of the interaction between structural elements, connections, columns and beams. It also provides an excellent opportunity to model theoretically such interactions by a detail mathematical model.

2. Effect of axial restraints

Among the various aspects relating to the structural totality, the restraint condition may be the most important and immediate factors affecting the behaviour of a beam. One of the purposes of this research project was to investigate the effect of catenary action, which relates largely to the axial response of the heated beam. Over the majority of the period of time in the fire, the beam have been expanding under the restraint of the pair of columns they were connected to, as shown in Fig. 1(a), and deflecting downward. This axial force increases gradually as a result of the thermal expansion. There comes a point when the temperature is so high and the material has deteriorated so much that the deflection of the beam starts to run-away. The response of the restraints is to hold it back by reversing the axial force

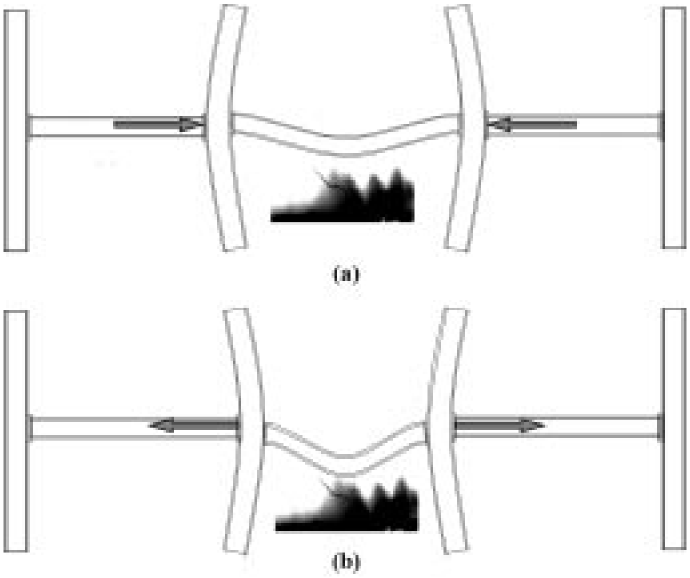


Fig. 1 (a) Heated beam under compression, (b) Heated beam under tension from compression to tension, and thus the catenary action as shown in Fig. 1(b). Effectively, the beam is hanged under its own member tension force. The onset and the rate of such reversal depend on the loading level. In most of the previous fire tests on isolated steel beams, effects of axial compression force and catenary action could not be observed and investigated in details.

3. Experimental detail

The work was based largely on two-dimensional studies, and will complement those already conducted on the full-scale frame at Cardington (Armer 1994), which have provided valuable data for steel frames with composite deck floors. The basic layout of the furnace is shown in Fig. 2. The furnace box was constructed of light rectangular hollow steel sections supporting thin steel plates with a ceramic fibre lining of thickness 200 mm.

The testing arrangement is in the form of the complete "Rugby goalpost" frame shown in Fig. 2. Basic restraints were provided by a pair of fire-protected test columns. Additional horizontal restraint could be provided by struts spanning between the column of the test frame and the column of the reaction frame at the level of the beam. The test beam, of 178×102×19UB (S275) section, was mainly unprotected. In order to simulate the heat-sink effect due to the concrete slab, the top flanges were wrapped with 15 mm thick ceramic fibre blanket, following a few trial and error in order to obtain similar temperature differences as in other beam tests. The columns, of 152×152×30UC (S275) section, together with the connections were generally fire-protected by the use of a 50 mm thickness of ceramic fibre blanket, and were therefore reusable for a series of tests. The columns were secured in position at top and bottom by four pin load-cells. Vertical point loads were applied to the beam using two independent hydraulic jacks connected to the top member of the reaction frame surrounding the furnace

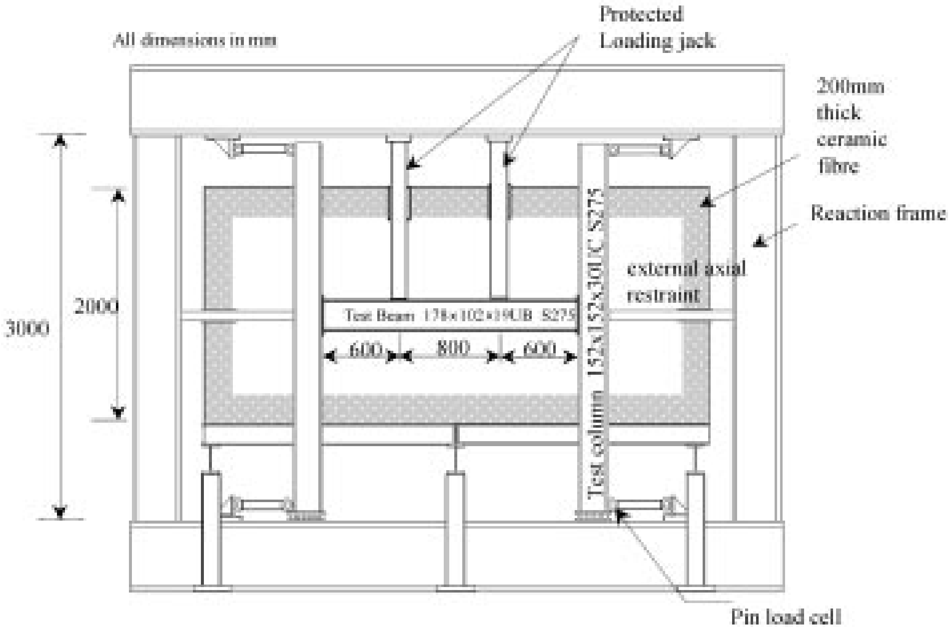


Fig. 2 Testing assembly (elevation)

to form a self-equilibrating system. Flush end-plate connections were used. The connections with the columns were fire-protected.

The behaviour of the sub-frames was assessed in terms of the moment and thrust resisted by the connections, the rotation of connections, and the mid-span deflection of the beam. The moments and thrusts transmitted by the connections to the columns were determined by measuring the horizontal reaction forces at the top and bottom of the columns using calibrated-pin load cells. A 200 mm displacement transducer located at the mid-span of the beam was used to measure the maximum deflection during temperature elevation. All displacement transducer measurements were made outside the furnace via ceramic rods which were pre-calibrated for the compensation of thermal expansion. The loading was applied manually at room temperature. It was then maintained at the same level throughout the fire test.

Three main levels of loading (Load Ratios of 0.3, 0.5 and 0.7) and various degrees of axial restraints were used. The Load Ratio is here defined, as in other literatures, as

$$L.R. = \frac{\text{applied load at fire lim it state}}{\text{load-carrying capacity at room temperature}} \tag{1}$$

The design moment and shear capacities of the 178×102×19UB are respectively 48 kNm and 156 kN, based on a yield strength of 275 N/mm^2. This gives an expected load-carrying capacity of 80 kN jack load at room temperature, the limit being due to flexural failure. Following the recommendation in BS476: Part 21, the beam is said to have failed in the fire limit state when the mid-span deflection exceeds span/20 (i.e., 100 mm in this series of tests). However, the temperature rise was allowed to continue in most tests and deflections continued to be measured in order to investigate the effect of catenary action.

The 3 m 152×152×30UC (S275) fire-protected test column alone in this sub-frame provided an axial restraint equivalent to a stiffness of 8 kN/mm (about 2% of the axial stiffness of the beam at room temperature) to the test beam. The possible in-plane restraint imposed by its neighbouring sub-frames, if any, has been estimated to have a value up to about 25% of the beam's axial stiffness. With the use of additional struts spanning between the column of the test frame and the column of the reaction frame, two other overall stiffnesses of 35 kN/mm and 62 kN/mm (respectively 9% and 16% of beam's stiffness) were achieved. These stiffnesses were obtained by direct calibration. The axial thrusts in the test beams were measured by strain gauges on the strut, together with the horizontal reactions at the top and bottom of the columns. The rotational restraints imposed by the restraint system, which is about 14000 kNm/rad, remained unaltered for all the cases.

4. Numerical modelling

The theoretical thermal-structural model consists of two modules: thermal response and structural response. Although more than 50 thermocouples were installed in the specimens to measure the temperature distribution, it is not possible to use them directly in the structural response model as a continuous function distributed over the whole structure. In particular, those confined regions with partial fire-protection, the temperature distribution is more complicated than those can be measured. Besides, the temperature distribution varies from test to tests owing to the non-uniformity of the fire inside the furnace, causing difficulty in direct comparison. In order to obtain a consistent comparison between various cases, a thermal responses analysis has to be carried out. The result could therefore be applied to the structural response model.

4.1. Thermal response analysis

The temperature distribution in the specimens was confirmed by a thermal analysis using a computer program FIRE-T3. It is based on a three-dimensional finite element analysis, in which the whole beam-and-column assembly was descretised into 8-node brick elements for thermal conduction. 4-noded surface elements were used for thermal radiation and convection purpose. The F.E. model includes the connection and all the stiffeners. The thermal properties of steel, such as thermal capacity, conductivity and density, were based according to the recommended values in Eurocode 3: Part 1:2, which may not reflect the actual properties of the materials used in the tests. Thermal convective and radiation coefficients were based on previous literatures and experience. The ceramic blankets were not included physically in the numerical model. Instead the surfaces which were covered by the ceramic blankets were given exceptionally small values of radiation absorptivity and thermal convective coefficients.

Fig. 3(a) shows the complete temperature-time curves at various locations measured in typical fire tests and compared with those obtained in the F.E. simulation. Fig. 3(b) shows the temperature contour plot after 12 minutes in the fire. Although the top flange of the beam was also fire protected, the heat energy from the hot web had no way to go, but stayed in the top flange. Its temperature, therefore, increased gradually, though at a much lower rate than other parts of the beam. The connection linked up the heated beam and the cold column. As the column is fully protected, it acted as a large heat sink. Heat energy was easily conducted away from the connection zone into the column and the temperature near to the connection can be kept lower. There was a zone of about 100 mm near to the connection, where the temperatures were much lower than the main part of the beam.

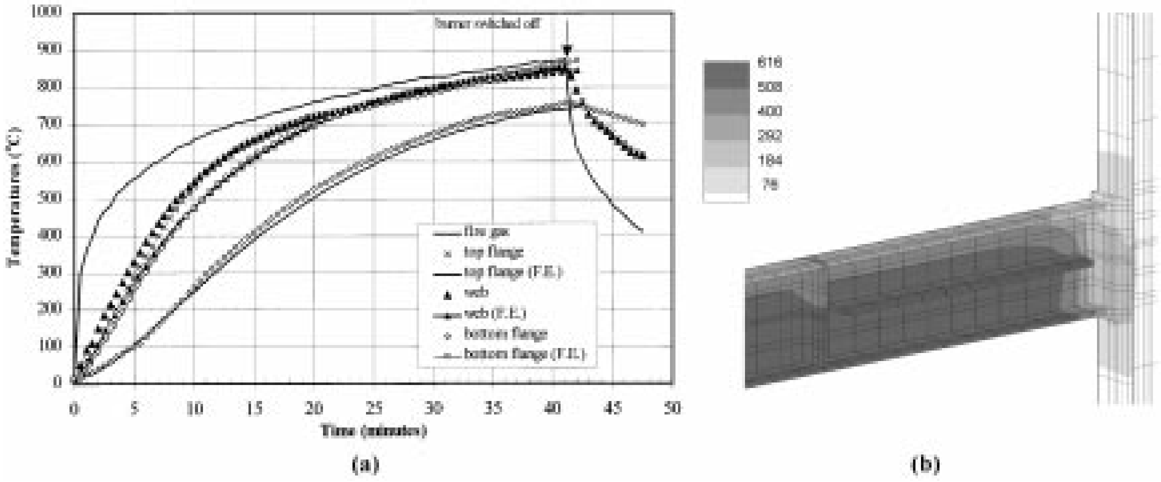


Fig. 3 (a) Temperature distribution, (b) simulated temperature distribution (FIRE-T3)

4.2. Structural response analysis

The finite element program FEAST, developed in-house, has been proved to be capable of simulating the behaviour of steel beam-to-column connections and of predicting their moment-rotation characteristics at elevated temperatures (Liu 1996, 1999). It is capable of generating the non-linear characteristics of a connection, and allows for the inclusion of arbitrary temperature distributions over the cross-section and along the members.

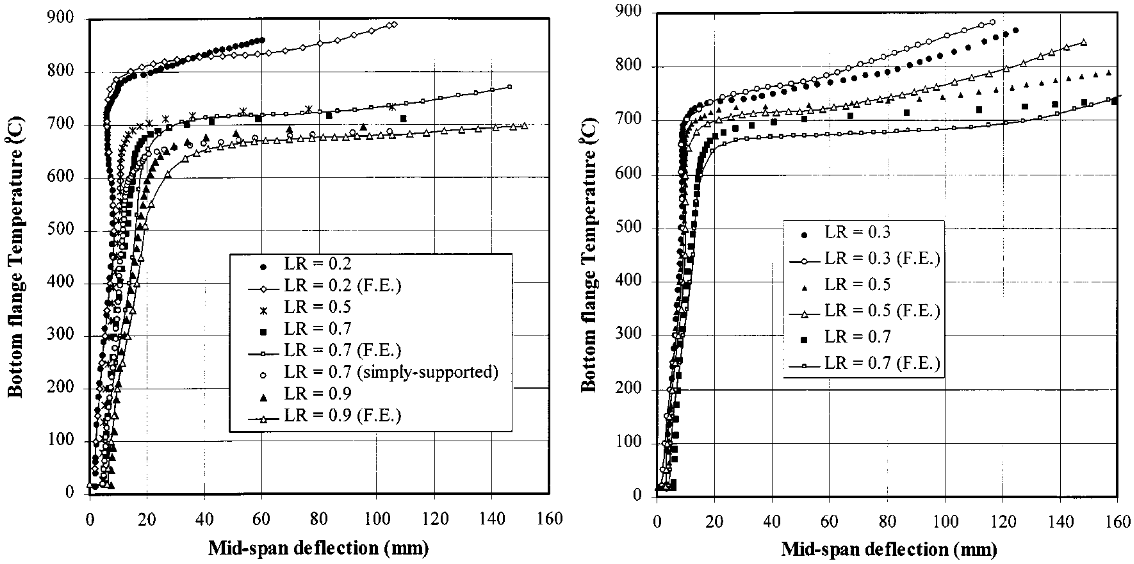
In short, the three-dimensional mathematical model is based an 8-noded isoparametric shell element, with tangential stiffness approach, allows sophisticated simulations to be executed economically. The model includes the consideration of the material plasticity and its deterioration with temperature. The stress-strain-temperature relation in Eurocode 3 (1994) was adopted. The yield strength and Modulus of Elasticity were obtained from coupon tests at room temperature. Von-Mises yield criterion is assumed valid even at very high temperature. Non-uniform thermal expansion across section and geometric non-linearity are included so that large deformations at very high temperatures can be modelled. Special elements were used for the modelling of the bolts and contact between end-plates and column flange (Liu 1999). The procedure allows the simulation of the structural behaviour of the connections be integrated with the behaviour of the members as opposed to most of the other global frame behaviour model. The interactions include the axial stiffness and the rotational stiffness, the re-distribution of loading in accordance with the instantaneous distribution of stiffnesses and temperatures.

The temperature distributions used are obtained from FIRE-T3, and are identical for all analyses. The nodal temperatures in the brick elements in the thermal analysis are interpolated into the nodal temperatures in the shell elements in FEAST. Any temperature gradient in the element thickness is ignored. The analyses start with applying loading to the required level at room temperature. The temperatures are then raised step-by-step in the numerical model. Maximum temperature steps of 10°C are used initially. When the deformation rate increases, the step sizes are controlled in order to obtain uniform deflection steps of 1 mm at the mid-span of the beam. During the initial modelling exercise, the finite element mesh sensitivity is examined and a typical optimal mesh for the specimen consists of more than 1200 nodes.

4.3. Temperature-deflection curves

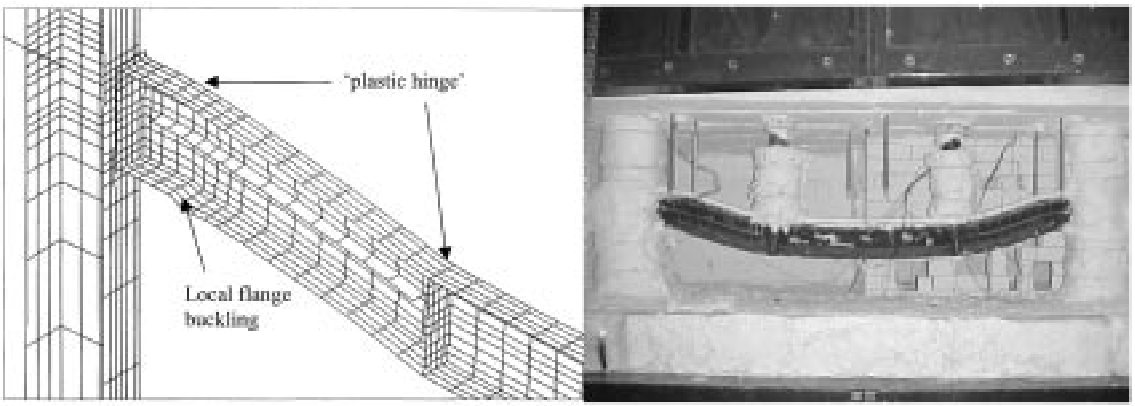
Figs. 4(a) and 4(b) show the temperature-deflection curves at various loading level with restraint provided by the columns only and by restraint of 35 kN/mm respectively. They also show their counter-parts obtained by the F.E. simulations. It should be noted that except the loading levels, the scenarios used in the numerical modelling are identical within the same restraint group of tests, whereas in the fire tests, these scenarios could not be possibly maintained identical mainly as a consequence of variations in the actual temperature distributions. Other factors such as unavailable material properties and the generalised stress-strain relationships used also contributed to the discrepancy between F.E. models and tests. Despite all these unfavourable conditions, the simulated temperature-deflection curves are within acceptable tolerance from the measured experimental curves.

Most of the tested beams were able to sustain the loads without excessive deflection up to a bottom flange temperature of about 650°C. They deflect initially very slowly as a result of the thermal gradient. The low rate is also due to the induced hogging moment at the connection. Beyond 550°C, there was reversal in the rise of temperature gradient (as seen in Fig. 3(a)), i.e., the top flange temperature then rose faster than the web and the bottom flange. The thermal bowing was thus reversed, leading to a



(a) K_R=8 kN/mm (0.02K_B) (b) K_R=35 kN/mm (0.09K_B)

Fig. 4 Temperature deflection curves at various loading levels



(a) F.E. simulation (b) After fire test

Fig. 5 Deformed beam

slight reduction of deflection in those beams with smaller loadings. As a result of material deterioration and the loss of hogging moment at the connection, the deflection increases very quickly at about 670°C to 750°C. This deflection runs away rapidly thereafter but is subsequently slowed down gradually when deflection becomes larger as a result of catenary action supported by the axial restraint. Fig. 5(a) shows a typical deformed beam after the fire. In general, two plastic hinges formed under the loading points and two others formed near the support connections. Near the connections there was sign of local buckling at the bottom flange. This resembles very closely to the deformed shape obtained by the F.E. simulation as shown in Fig. 5(b).

Figs. 6(a) and 6(b) compare the temperature-deflection curves obtained by the F.E. simulations for K_R

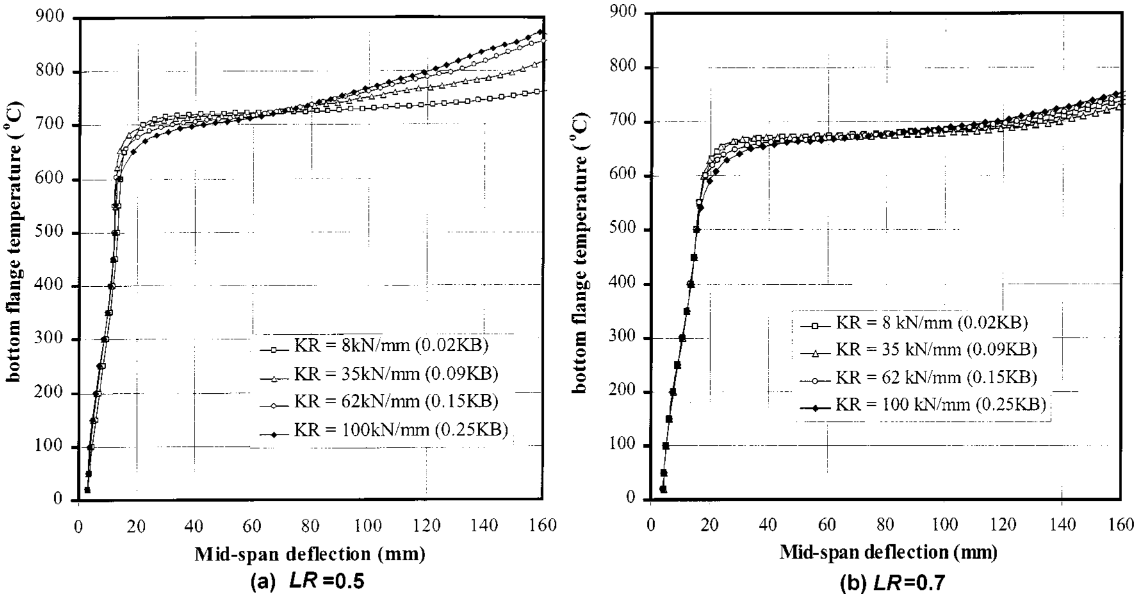


Fig. 6 Temperature deflection curves ar various restraint levels (F.E. simulations)

up to 0.25 K_B subjected to identical temperature regime for LR = 0.5 and 0.7. They have effectively captured all the deflection characteristics throughout the temperature rise. There is little difference in the initial stages within the same loading group. However, the deflections of those with higher restraints start to runaway earlier at a slightly lower temperature. With a higher loading level, such as LR = 0.7 as shown, the subsequent behaviour is dominated by the extensive loss of material strength and stiffness. For those with smaller loading level, the strength in the member, especially that preserved in the top flange where the temperature is lower, is able to work with the external restraint. The deflection run-away can be slowed down. It is more obvious in those with higher restraint. As a result, the commencement of catenary action comes earlier for higher restraint beams.

4.5. Axial force in the beam

The principal action to support the loading at high temperature is the member axial force. Axial compression force develops when the thermal expansion in the heated steel beam is resisted by the column and restraints from adjacent structure. Fig. 7 plot the growth of the measured axial compressive force against the bottom flange temperatures in the beams with flush-end-plate connections in the three restraint groups. They are also compared with the corresponding curves obtained from the F.E. simulations.

All curves within the same restraint group have similar gradients up to a fairly similar maximum compression force, confirming that they have similar rates of thermal expansion irrespective of the applied load ratio. The minor variation is mainly because of the temperature distribution. The initial rise of this axial compression force can easily be evaluated by

$$\Delta P = \frac{1}{1 + \dfrac{K_R}{K_B}} K_R L \alpha \Delta T \tag{2}$$

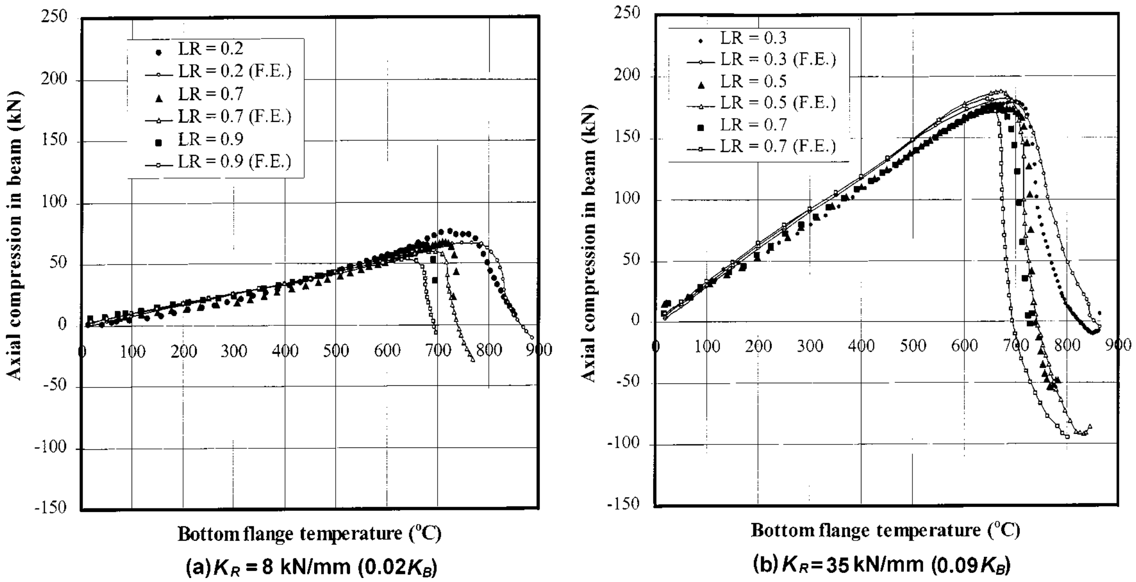


Fig. 7 Axial force vs temperature at various loading levels

where K_B (400 kN/mm) is the axial stiffness of the beam at room temperature, K_R is the stiffness of the restraint (including the column) due to an axial force in the beam, α is the coefficient of thermal expansion and ΔT is the change in the mean temperature of the beam and L is the length of the beam. Since the restraints are fire-protected, the stiffness K_R is largely unchanged. However, K_B, which relates to the temperature depended Young's modulus and the extent of 'yielding', change with the mean temperature of the beam.

With the column restraint alone, K_R = 8 kN/mm. This gives a thermal compression force of about 9.5 kN per 100°C rise of mean temperature or about 8 kN per 100°C in bottom flange temperature initially. In another case, an additional restraint was provided so that the overall axial restraint to the heated beam became 35 kN/mm. The corresponding restraint force becomes 160 kN per 100°C rise in bottom flange temperature. They are similar to those measured in the tests, as well as those obtained from the F.E. simulations. The comparisons are very good apart from some discrepancy due mainly to the variation in the temperature distributions.

According to Eq. (2), the axial compression force starts to drop when the heated beam loses most of its axial stiffness relative to the restraint stiffness. This is followed by the onset of the vertical deflection run-away. With large value of K_R, this will take place earlier. The F.E. simulation captures the instance fairly accurately, though the onsets of run-away are consistently earlier. At temperature of such level, the behaviour would be very sensitive to the actual material behaviour and the test performance.

After the axial force changes into tension, the vertical load on the beam can partly be supported by the vertical component of the member tension force and the beam is hung like a cable. Note that the axial force depicted here is the horizontal component of the member force, which may be substantially larger at large deflection. The run-away is therefore slowed down and the rise of tensile force also slows down. Fig. 8 compares the theoretical axial forces for beams with various magnitudes of axial restraints and demonstrates the validity of Eq. (2). It shows clearly its correlation with the temperature-deflection

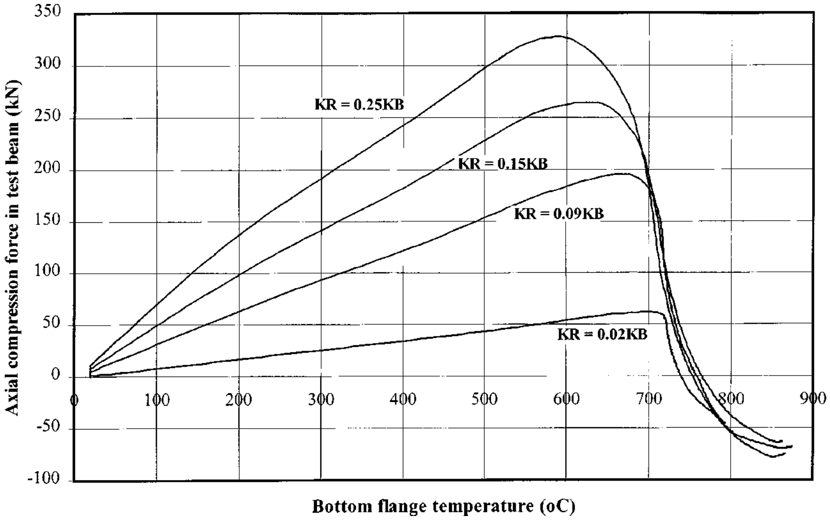


Fig. 8 Axial force vs temperature at various restraint levels (LR = 0.5) (F.E. simulations)

curves shown in Figs. 6(a) and 6(b). The compression force starts to drop at a slightly lower temperature for cases with higher axial restraints.

5. Catenary action

Catenary action in the beam can be defined as the ability of the beam to support itself by means of axial tension when the beam undergoes a large deflection and behaves as a mechanism in the form shown in Fig. 5(a). Catenary action usually takes place at large deflection, and it can effectively prevent the beam deflections from running away. In this particular set of beams, most of the catenary action became obvious only after the deflection had exceeded span/20 and temperature above 700°C. The condition of the beam can be diagrammatically represented in Fig. 9. The two roller supports represents the movement of the column controlled by stiffness represented by the springs. The beam between the two roller supports forms a mechanism with plastic hinges formed near to the column-beam connections with capacities $M_{c,\theta}$, reduced due to the co-exist axial force $\boldsymbol{T}$. Two other plastic hinges are formed under the point loads with capacity $M_{b,\theta}$. At large deflection, the applied load $\boldsymbol{P}$ is partly

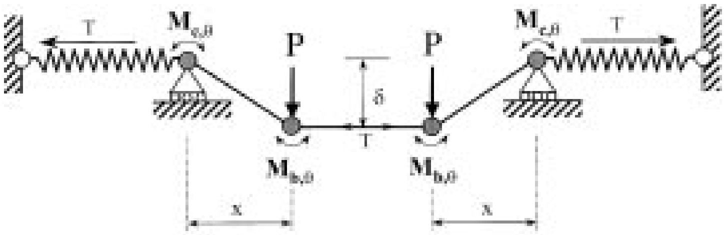


Fig. 9 Catenary mechanism at large deflection

supported by the remaining shear/moment capacity and partly by the vertical component of the tension.

$$\boldsymbol{P} = \frac{M_{c,\theta} + M_{b,\theta}}{x} + T\frac{\delta}{x} \tag{3}$$

where $\boldsymbol{T}$ is the horizontal component of the tension force in the beam and δ is the mid-span deflection of the beam.

At the time when the bottom temperature reaches 750°C, the temperature in the top flange is about 600°C. The strength in the web and the bottom flange is only about 10% of that at room temperature, while the top flange has about 40% left. The tensile capacity remained in the beam is about 110 kN (mainly from the top flange). Assuming there is no bending stiffness in the beam, the mechanism has to deflect by 110 mm to achieve equilibrium with a pair of point loads of 20 kN (*L.R.*= 0.5). The corresponding value for *L.R.* = 0.7 is 158 mm. In reality, the bending stiffness left in the beam at this temperature is sufficient to support the load with very little contribution from the tension effect. However, for further increase in temperature, the bending stiffness drops at a much higher rate than the tensile capacity. When the bottom temperature reaches 800°C, the tensile strength is reduced to about 75 kN while the bending stiffness has virtually vanished, i.e., $M_{c,\theta}$ and $M_{b,\theta} = 0$. The required deflections are 166 mm and 240 mm respectively for the two loading cases to achieve equilibrium. These agree with those obtained from the F.E. analyses.

5.1. Limiting temperature for fire engineering design

The principle of limiting temperature is one of the design methods recommended in the British Steel design code (BS5950: Part 8:1990). It is the temperature in the bottom flange when a beam, subjected to a load ratio at fire limit state, fails in the event of a fire. One of the failure criteria associated with the structural stability of the beam in fire is a deflection limit of span/20 (BS476:Part 20). The limiting temperature recommendation is based on a large amount of fire tests on steel beams. However, no support condition and restraint condition is considered. From the tests and the F.E. simulation, it can be seen clearly that the beam does not become unstable but can sustain the applied load at a high temperature with the aid of the external restraint. It remains arguable whether a deflection limit is adequate to allow the advantage of catenary action to be considered in the design. If a high deflection limit is used, the limiting temperature will be increased accordingly. Fig. 10 shows the relationship between the limiting temperature and the axial restraint when different deflection limits are adopted. By using a deflection limit of 160 mm (span/12.5), the limiting temperature increases from 765°C to 860°C for the beam with $K_R = 0.25\,K_B$ and *L.R.*= 0.5. This corresponds to about 15 minutes in the temperature regime used in this study. The advantage is smaller with a higher load ratio and smaller restraint.

6. Conclusions

Traditionally, structural fire behaviour is assessed on the basis of experiments on isolated structural element, in which structural continuity and effect of restraint are neglected, so that conservative results are obtained. In the tests described in this paper, a part frame of the form of a complete “Rugby goalpost” was examined.

The behaviour of the beam under the influence of axial restraints now becomes clearer. When the temperature starts to rise, the top flange stays cooler than the rest of the section, causing a downward

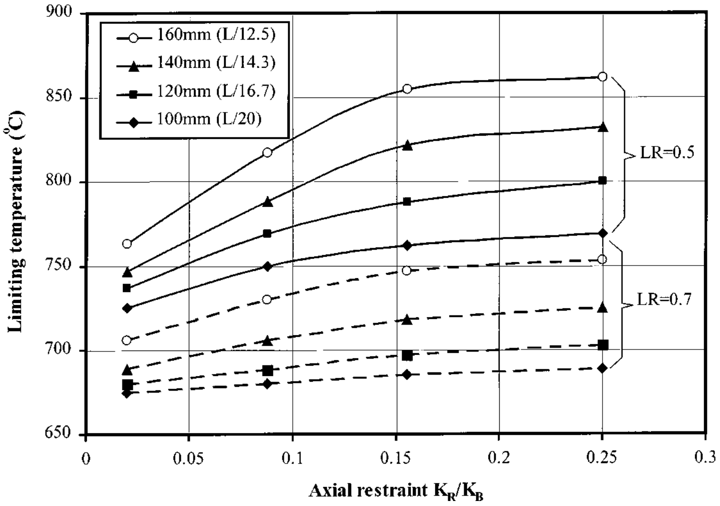


Fig. 10 Effect of axial restraints on limiting temperatures

thermal bowing. An axial compressive force is induced along the beam. As the temperature rises and material yields further, the loss of bending strength make it difficult to support the loading and run-away starts at about 700°C. The axial restraint then reacts quickly; the compression force drops rapidly and changes into a tension force. As catenary action takes place, slowing the rate of run-away. The connection then loses its moment of resistance and this leads to immediate loss of stiffness. The catenary action is more pronounced in cases with lower load levels and higher axial restraint. However, it becomes obvious only at large deflection. A new structural "failure" criterion may need to be formulated to define the fire limit state for beams, when there is no intrinsic need to limit deflections.

A finite element model FEAST, based on an 8-noded isoparametric element, has been adopted to model the behaviour of partially fire-protected steel beams in fire under an axial restraint. The input temperature is generated numerically by another program FIRE-T3. Various loading levels are applied in accordance to those in the fire tests. The temperature-deflection curves and the variation of axial forces in the heated beam were compared with the experimental results. While the input temperature distributions are kept identical for all models, the discrepancies between the tests and the numerical models are extremely limited.

The advantage of FEAST over many other numerical models is that the influence of each component and the interaction with one another can be generated within themselves and not artificially assigned externally. This is evidenced by the reproduction of some important features of the test results: the rotational behaviour of the connections; local flange buckling near to connection; the formation of plastic hinge at about 125 mm outside the connection; and the transmission of axial force and bending moment between the columns and the beams and so on. This software has also been effectively used to control the conduct of the testing procedure and to understand the behaviour during the course of the research programme.

The advantage of the catenary action has been clearly demonstrated. It is a result of the inclusion of the axial restraint provided by the cooler adjacent structures outside the fire compartment. However, it only happens at large deflection. The advantage can only be utilized when the deflection limit, normally adopted as one of the fire limit state criteria, is raised accordingly.

Acknowledgements

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