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A simplified analysis of catenary action in steel beams in fire and implications on fire resistant design

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Abstract. This paper describes the results of a numerical investigation of the large deflection behaviour of steel beams under fire conditions, taking into consideration the effect of catenary action provided by the surrounding structures. The main focus is on the development, validation and application of a simplified calculation method that may be adopted in design calculations. Because no experimental result is available for validation of the simplified calculation method, the finite element program ABAQUS has been used to simulate the large deflection behaviour of a number of steel beams so as to provide alternative results for validation of the proposed method. Utilising catenary action has the potential of eliminating fire protection to all steel beams without causing structural failure in fire. However, practical application of catenary action will be restricted by concerns over large beam deflection causing integrity failure of the fire resistant compartment and additional cost of strengthening the connections and the surrounding structures to resist the catenary forces in the steel beams. This paper will provide a discussion on practical implications of utilising catenary action in steel beams as a means of eliminating fire protection. A number of examples will then be provided to illustrate the type of steel framed structure that could benefit the most from exploiting catenary action in fire resistant design.

Keywords: large deflection; fire resistant design; catenary action; steel beams; fire engineering; integrity of fire compartment.

1. Introduction

When assessing the performance of a steel beam under fire, the assessment is often based on the beam's flexural bending behaviour at elevated temperatures. In the presence of axial restraint at ends to the beam, the behaviour of the beam at large deflections is much different from that in flexural bending. The main feature of the behaviour of a steel beam at large deflections is that the effect of end axial restraints cannot be ignored because these end axial restraints can make the beam behave in catenary action which can become the dominant load carrying mechanism, instead of flexural bending. When acting in catenary, the beam will mainly be in tension and its resistance to the applied vertical load comes from the catenary force in the beam acting on the beam's vertical deflection. It is this mode of behaviour that will be the main focus of this paper.

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Compared with fire resistant design based on flexural bending behaviour of a beam (BSI 1990, CEN 2001), exploring catenary action has two important design implications. (1) Because the beam's resistance in catenary action is not limited by its material strength but is related to the beam's ability to deflect, the beam can still resist the applied load even at much reduced material strength at high temperatures, simply by deflecting more. Thus, it is theoretically possible that the steel beam would not collapse until near melting of the steel. If large deflection is acceptable, there is no need to provide any fire protection to the steel beam. It is commonly accepted that eliminating fire protection to steel structures has been one of the main drivers of recent steel structural fire research. Whilst the findings of the Cardington research programme have led to major advances in this area, through utilising tensile membrane action in the composite floor slabs (Bailey and Moore 2000a, b, Bailey *et al*. 2000, Newman *et al*. 2000), there are a number of restrictions to limit the scope of application of this method. For example, this method cannot be used in composite construction with precast floor slabs because these floor slabs are one-way spanning and would not be able to generate the necessary tensile membrane action. Furthermore, a substantial number of steel beams would still have to be fire protected to provide the necessary vertical support to the floor slabs to generate tensile membrane action. Utilising catenary action has the potential to eliminate these problems so as to enable wider use of unprotected steelwork, thereby having significant impact on construction cost. (2) An axially restrained beam will always develop catenary action at temperatures above the limiting temperature calculated on the basis of pure flexural bending behaviour. Therefore, if for any reason, the beam's temperature cannot be guaranteed to remain below this limiting temperature, e.g. due to damaged fire protection, the beam will impose a catenary force on the adjacent structure. If this catenary force is not recognised and the adjacent structure is not properly designed to resist this catenary force, there is a danger of the adjacent structure collapsing. The World Trade Center investigation report (FEMA 2002) indicates that this might have contributed to the collapse of the WTC buildings on September 11, 2001. Thus, proper allowance for catenary action in steel beams in fire resistant design of steel structures may bring about the benefit of safety and economy at the same time.

Up to now, catenary behaviour of steel beams has not been considered in fire resistant design. Even though the US fire test standard ASTM E119 (ASTM 2000) includes standard fire tests on restrained beams, the restraint is intended to prevent thermal expansion of the beam, not pull-in of the beam during catenary action. Also the acceptance criteria in the US fire test standard are based on temperature limits, not structural behaviour. This makes it difficult to collate sufficient data to form a database of fire test results for the development of a simple design method based on experimental results. Fortunately, with advances in finite element technology, it is almost trivial to be able to predict catenary action in steel beams using existing finite element analysis packages (e.g. Allam *et al*. 2002, Elghazouli and Izzuddin 2000, Moss *et al.* 2003, 2004, Yin and Wang 2004, Yu and Liew 2002). However, to enable wide consideration of catenary action in routine fire resistant design of steel structures, it is desirable to have a much less complicated calculation approach. The objective of this paper is to report the development and validation of such a simplified calculation method. When considering catenary action, the designer is interested to know two important quantities: the deflection of the beam so as to determine whether or not large beam deflections would be acceptable, and the maximum catenary force in the beam that the adjacent structure would have to resist to prevent structural collapse. These two quantities are the main outputs of the simplified calculation method.

Fire resistant design utilising catenary action in steel beams is new and there will be practical concerns. This paper will use examples to provide insight into how the following concerns would influence practical utilisation of catenary action in fire resistant design of steel beams.

(1) Would the large beam deflections cause integrity failure of the fire resistant compartment of which the beam is part?
(2) Would the connections need substantial strengthening to resist the catenary force in the beam?
(3) Would utilising catenary action to eliminate fire protection to steel beams be cost effective?

2. Simplified calculation method

2.1. Equilibrium equation

Fig. 1 shows a general steel beam with axial and rotational restraints at ends and non-uniform temperature distribution in the cross-section. Assuming that the beam is symmetrically loaded, the equilibrium equation is:

$$F_T\delta_{\max} + M_T + M_R + M_P = 0 \tag{1}$$

where F_T is the axial load in the beam; M_T the internal bending moment in the mid-span; M_R the restraint bending moment at the beam ends and M_P the maximum free bending moment in the beam under the externally applied load. $\delta_{\max}$ is the maximum total deflection of the beam.

Assume that the total lateral deflection of the beam, as a function of its position, is

$$z(x) = z_{mec}(x) + z_{th}(x) \tag{2}$$

where z_{mec} is the mechanical deflection and z_{th} the thermal bowing deflection of the beam.

The total axial shortening of the beam is

$$\Delta L = \int_0^L \left[1 + \left(\frac{dz}{dx}\right)^2\right]^{1/2} dx - L \tag{3}$$

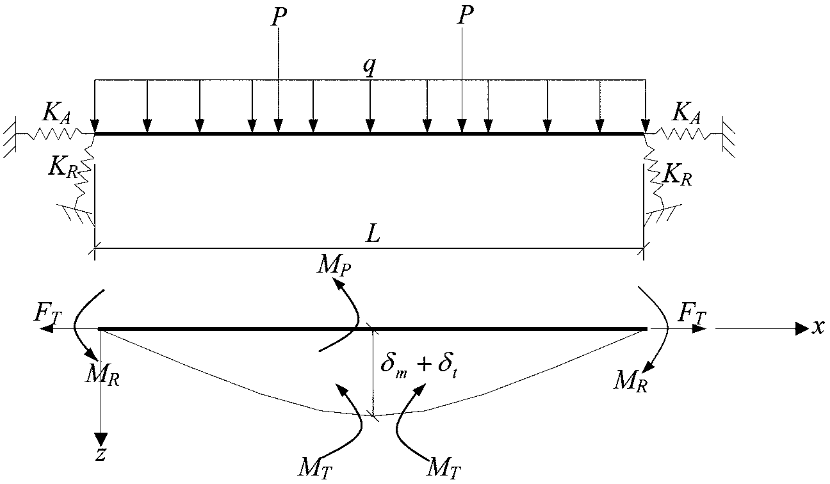


Fig. 1 Beam deflection and equilibrium diagrams

The beam's thermal expansion is

$$\Delta L_{th} = \alpha T L \tag{4}$$

where α is the coefficient of thermal expansion of steel and T the average temperature of the beam.
The total change in the beam's length as a result of mechanical strain is

$$\Delta L_{mec} = \Delta L - \Delta L_{th} \tag{5}$$

Assuming elastic behaviour for the time being, the beam's axial force (F_T), internal bending moment at mid-span (M_T) and internal bending moment at supports (M_R) may be expressed as:

$$F_T = K'_A\,\varepsilon_{mec} = K'_A\frac{\Delta L_{mec}}{L} = \frac{1}{\dfrac{1}{K_A}+\dfrac{L}{E_T A}+\dfrac{1}{K_A}}\frac{\Delta L_{mec}}{L} \tag{6}$$

$$M_T = E_T I_y \varphi_{mec}\Big|_{x=\frac{L}{2}} = E_T I_y \frac{d^2 z_{mec}}{dx^2}\bigg|_{x=\frac{L}{2}} \tag{7}$$

$$M_R = E_T I_y \varphi_{mec}\Big|_{x=0} = E_T I_y \frac{d^2 z_{mec}}{dx^2}\bigg|_{x=0} \tag{8}$$

in which, K_A is the axial restraint stiffness of the surrounding structure at the beam ends. E_T is the average Young's modulus of the cross-section. $E_T I_y$ is the temperature dependent flexural rigidity of the cross-section.

To consider inelastic material behaviour, it is necessary to impose limits on various terms of beam strength in Eqs. (6-8). The limit to the beam's axial force (F_T) is the lesser of the end axial restraint strength or the yield strength of the beam. For the mid-span bending moment M_T, it is limited by material yield taking into consideration the interaction between bending moment and axial load F_T, given in Eq. (9). For the end restraint bending moment M_R, the limit is the lesser of that considering interaction between bending moment and axial load or the bending moment resistance of the end rotational restraint.

For an I-beam at ambient temperature, considering interaction between the axial load and bending moment, the beam's axial force and bending moment should satisfy (ESDEP 1994):

$$\frac{M_{T(/R)}}{M_{Tp}} + \frac{(1+\alpha)^2}{\alpha[2(1+\beta)+\alpha]}\left(\frac{\mathrm{abs}(F_T)}{F_{Tp}}\right)^2 = 1 \qquad \frac{\mathrm{abs}(F_T)}{F_{Tp}} \le \gamma \tag{9a}$$

$$\frac{1-\gamma}{1-\dfrac{(1+\alpha)^2\gamma^2}{\alpha[2(1+\beta)+\alpha]}}\frac{M_{T(/R)}}{M_{Tp}} + \frac{\mathrm{abs}(F_T)}{F_{Tp}} = 1 \qquad \frac{\mathrm{abs}(F_T)}{F_{Tp}} > \gamma \tag{9b}$$

in which $\alpha = \dfrac{A_w}{2A_f}$, $\beta = \dfrac{t}{h_0}$ and $\gamma = \dfrac{A_w}{2A_f + A_w}$

In the above equations, F_{Tp} and M_{Tp} are the beam's axial and bending moment capacity respectively. These equations are also used, without validation, at high temperatures.

2.2. Deflection profiles

2.2.1. Beams with uniform temperature distribution

There is no thermal bowing deflection so that $z_{th} = 0$. The mechanical deflection profile is the same as the total deflection profile and will depend on the load type and the end rotational restraint.

Without any rotational restraint at the beam ends, the deflection profile of a beam under distributed load can be assumed to be a polynomial that satisfies the geometric (zero deflection at $x = 0$ and $x = L$, zero rotation at $x = L/2$) and bending moment (zero curvature at $x = 0$ and $x = L$) boundary conditions. The deflection profile of a beam under point loads should take the shape of the beam's free bending moment diagram. For example, the mechanical deflection profiles of a beam under uniformly distributed load or mid-span point load are:

$$z_{mec} = \frac{16\delta_{\max,\,mec}}{5L}\left(\frac{x^4}{L^3} - \frac{2x^3}{L^2} + x\right) \qquad \text{Under uniformly distributed load} \tag{10}$$

$$z_{mec} = \begin{cases} \dfrac{2\delta_{\max,\,mec}}{L}x & 0 \le x \le L/2 \\ \dfrac{2\delta_{\max,\,mec}}{L}(L-x) & L/2 \le x \le L \end{cases} \qquad \text{Under mid-span point load} \tag{11}$$

where $\delta_{\mathrm{mec,max}}$ is the beam's maximum mechanical deflection. It should be pointed out that the above different deflection profiles are used to calculate the axial force in a beam under the different associated loading conditions via Eqs. (3)-(6). For calculation of the beam's bending moments using Eqs. (7) and (8), since Eq. (11) would give infinite curvature at the position of point load, Eq. (10) is used for all loading cases. This introduces some error in calculating bending moments for beams with point loads. However, since the emphasis of this research is on catenary action in a beam and at this stage the effect of bending moments is small, this inaccuracy is considered to be small.

With complete end rotational restraint, the total deflection profile of a beam regardless of its loading condition may be assumed to be a 4th polynomial that satisfies the beam's geometric boundary conditions (zero deflection and rotation at $x = 0$ and $x = L$, zero rotation at $x = L/2$), giving:

$$z = \frac{16\delta_{\max}}{L^2}\left(\frac{x^4}{L^2} - \frac{2x^3}{L} + x^2\right) \tag{12}$$

With a flexible end rotational restraint, interpolation between zero restraint and total restraint should be used. Then the beam's deflection profile can be written as

$$z = c_s z_s + c_f z_f \tag{13}$$

in which

z_s is the deflection profile of a beam with zero end rotational restraint;
z_f is the deflection profile of a beam with complete end rotational restraint;

c_s, c_f are the degree of end rotational restraint, given by

$$c_f = \frac{K'_R}{K_{BR}} \le 1, \quad c_s = 1 - c_f \quad \text{where } \frac{1}{K'_R} = \frac{1}{K_R} + \frac{1}{K_{BR}} + \frac{1}{K_R} \tag{14}$$

in which K_R is the rotational restraint stiffness of the surrounding structure at the beam ends and K_{BR} the beam's flexural stiffness, given by $E_T I_y / L$.

2.2.2. Beams with non-uniform temperature distribution in the cross-section

This applies to steel beams that support a concrete slab on top and are exposed to fire on all sides. Assume a constant thermal curvature of

$$\varphi_{th} = -\frac{\alpha \Delta T}{h} \tag{15}$$

where ΔT is the temperature difference between the top and bottom of the beam cross-section and h the height of the cross-section. According to geometric boundary conditions (zero deflection at $x = 0$ and $x = L$, zero rotation at $x = L/2$), Eq. (15) gives:

$$z_{th} = -\frac{\alpha \Delta T}{2h}(x^2 - Lx) \tag{16}$$

The thermal deflection in Eq. (16) should be added to the mechanical deflection of Eq. (10) or (11) to give the total deflection of the beam if the beam's end rotational restraint is zero.

For a beam with complete end rotational restraint and non-uniform temperature distribution in the cross-section, Eq. (12) gives the total deflection because it is derived purely on the basis of satisfying the geometrical boundary conditions. Therefore, when using Eqs. (7) and (8) to calculate the beam's bending moments, the mechanical deflection should be obtained by deducting Eq. (16) from Eq. (12).

A modification to the deflection profile of a beam with concentrated point load (CPL) should be made. Previously under uniform temperature distribution, $z_{mec} = z_{CPL}$ where z_{CPL} is the free bending moment profile (e.g. Eq. (11)). This is the deflection profile under pure catenary action, which is a reasonable assumption under uniform temperature distribution where numerical simulation results using ABAQUS indicate that the beam is yielded in almost pure tension with very little flexural bending deflection. Under non-uniform temperature distribution in the cross-section of a beam, the material is not fully yield in tension in the whole cross-section. Therefore, the beam's deflection profile should consist of partly catenary action profile, which is similar to the beam's bending moment diagram (e.g. Eq. (11), represented by z_{CPL} in Eq. (17)), and partly flexural bending deflection profile, which is similar to the beam's deflection profile under uniformly distributed load (e.g. Eq. (11), represented by z_{UDL} in Eq. (17)), obtained from consideration of the beam's geometric and bending moment boundary conditions. It is difficult to find the exact proportion of these two contributions, but an equal share has been found to be an acceptable approximation.

Therefore for a beam under CPL, the total deflection profile may be assumed to be

$$z = \frac{z_{UDL} + z_{CPL}}{2} \tag{17}$$

Under partial rotational restraint at the beam's ends, Eq. (13) should apply as under uniform temperature distribution.

Table 1 Summary of steel beam deflection profiles for catenary analysis

Situation		Total deflection	Thermal bowing deflection
Uniform temperature distribution	Simply support	UDL: $z_{mec} = \frac{16\delta_{\max,\,mec}}{5L}\left(\frac{x^4}{L^3} - \frac{2x^3}{L^2} + x\right)$ CPL: z_{mec} = free bending moment diagram	$z_{th} = 0$
	Fixed end	$z = \frac{16\delta_{\max}}{L^2}\left(\frac{x^4}{L^2} - \frac{2x^3}{L} + x^2\right)$	$z_{th} = 0$
	Flexible rotation	$z = z_{mec} + z_{th} = c_s z_s + c_f z_f$	
Non-uniform temperature distribution	Simply support	UDL: $z_{mec} = \frac{16\delta_{\max,\,mec}}{5L}\left(\frac{x^4}{L^3} - \frac{2x^3}{L^2} + x\right)$ CPL: $z = \frac{z_{UDL} + z_{CPL}}{2}$	$z_{th} = -\frac{\alpha\Delta T}{2h}(x^2 - Lx)$
	Fixed end	$z = \frac{16\delta_{\max}}{L^2}\left(\frac{x^4}{L^2} - \frac{2x^3}{L} + x^2\right)$	
	Flexible rotation	$z = z_{mec} + z_{th} = c_s z_s + c_f z_f$	

2.2.3. Summary

Table 1 provides a summary of the deflection profiles of beams with different end rotational restraints, loading conditions and temperature distributions. Fig. 2 provides a general flow chart to implement the simplified calculation method.

3. Validation against ABAQUS simulations

In order to validate the accuracy of the proposed simplified calculation method, this section will present a number of comparisons against numerical simulations using the general finite element package ABAQUS (2001). Detailed results of ABAQUS simulations have been presented in a previous paper (Yin and Wang 2003) and more extensive comparison between predictions using ABAQUS and the simplified method can be found in the second author's PhD thesis (Yin 2004).

Figs. 3-8 compare the predicted temperature – deflection and temperature – axial reaction relationships between the proposed method and the results by ABAQUS of an 8m UB457×152×60 beam with different boundary conditions, which show good agreement between the two sets of results.

Under uniform temperature distribution (Figs. 3, 5, 7), the proposed method predicts catenary forces that will become equal to the beam's tensile capacity at elevated temperatures, but numerical simulations will always predict slightly lower values. This comes about because these two methods use different ways of interaction between axial force and bending moment. In numerical simulations, both the axial force and bending moment are calculated from strain and stress distributions that satisfy structural equilibrium, deflection compatibility and material yield criteria. Therefore, there is always some contribution of bending moment resistance and the beam's catenary force would never reach its material yield strength. In the proposed method, the beam's catenary force is determined from its geometry without

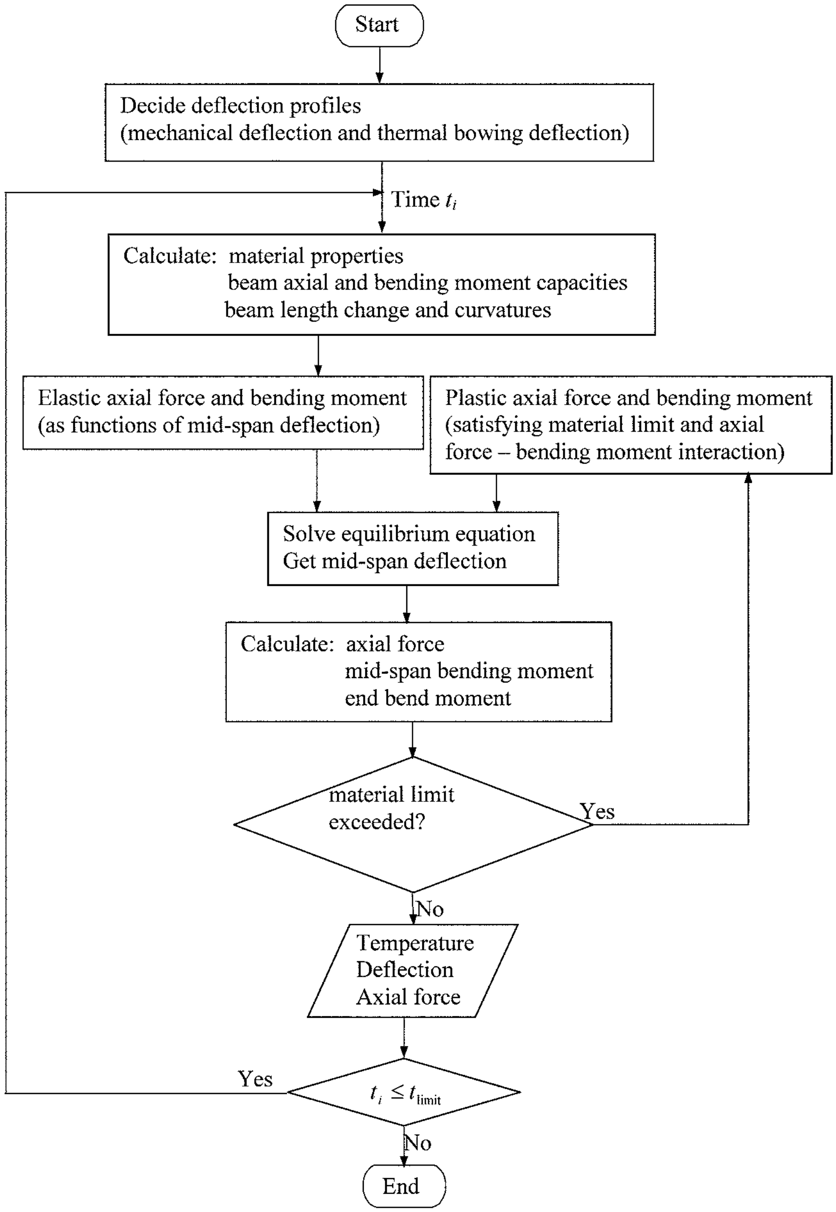


Fig. 2 General flow chart of the simplified calculation method for catenary action analysis

considering its stress distribution and bending moment resistance. This force will increase at increasing deflection until it reaches the beam's yield strength in tension, after which the beam's bending moment resistance becomes zero. Nevertheless, since the beam's bending moment resistance becomes negligible at the catenary action stage, the proposed calculation method produces results that are close to numerical predictions. In all cases, the proposed method gives slightly higher catenary forces.

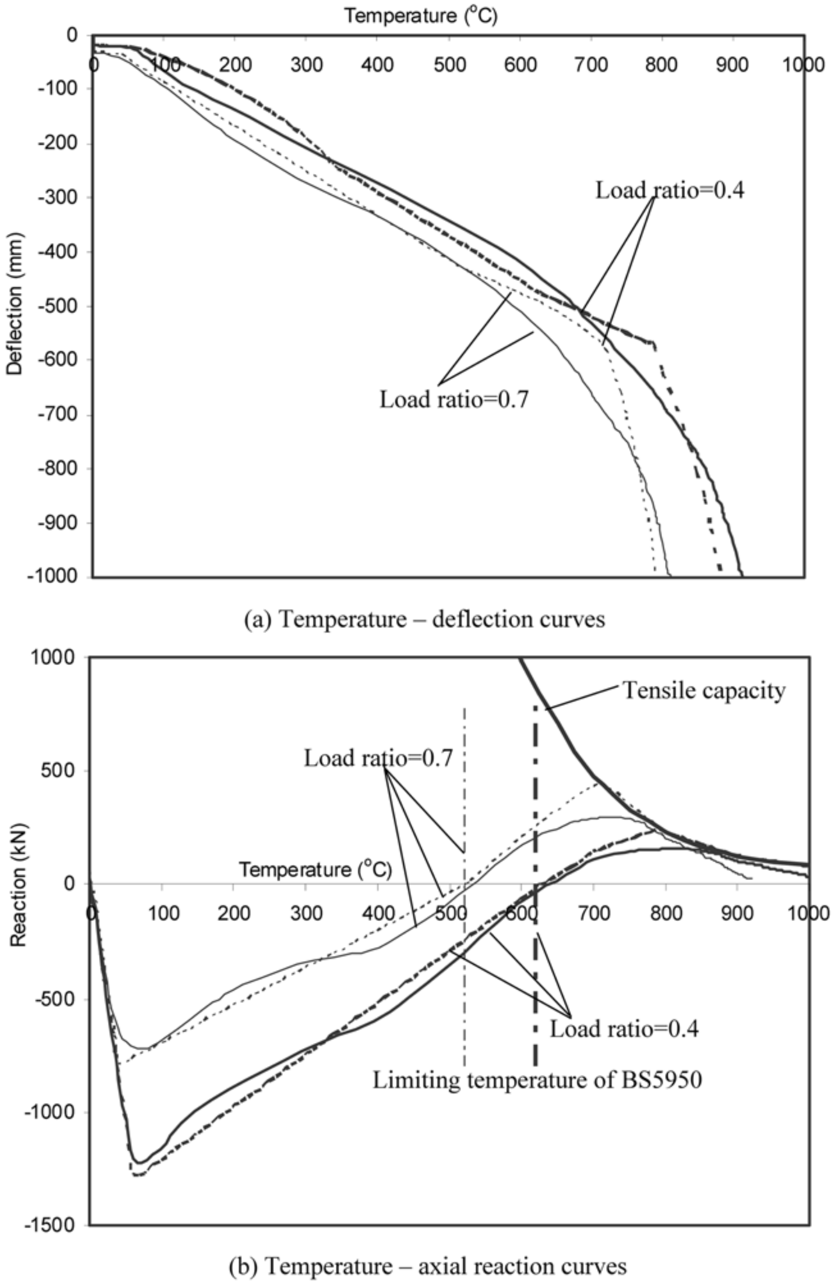
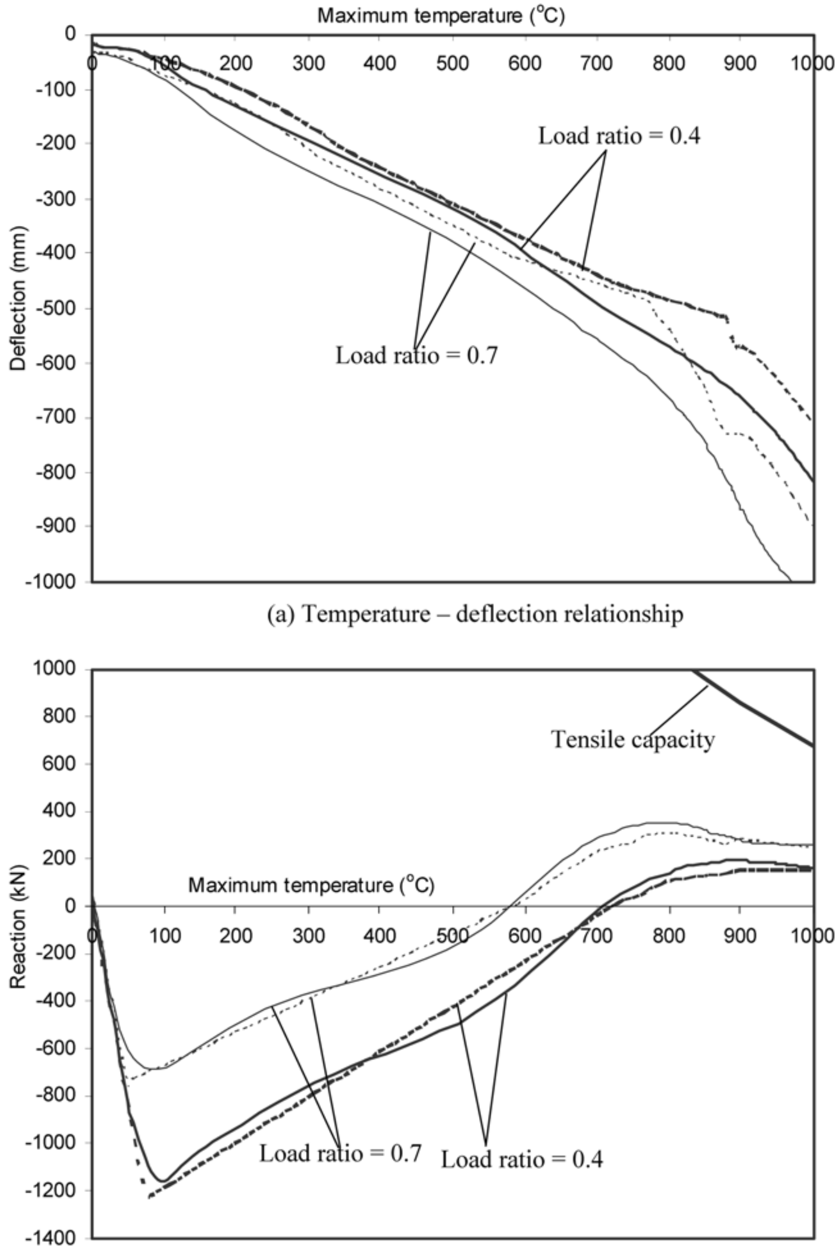


Fig. 3 Comparison between results of ABAQUS (broken lines) and HCM (solid lines). Uniform temperature distribution, UDL, fully axial restrained and free rotation at ends

Under non-uniform heating (Figs. 4, 6, 8, with the temperature of the top flange being half that of bottom flange and web), the maximum catenary force will not reach the beam's tensile capacity in general. This comes about because: (1) Non-uniform temperature distribution produces a thermal bowing deflection that is additional to the mechanical deflection. Therefore, at high deflections, the



(a) Temperature – deflection relationship

(b) Temperature – axial reaction relationship

Fig. 4 Comparison between results of ABAQUS (broken lines) and HCM (solid lines). Non-uniform temperature distribution, CPL, fully axial restrained and free rotation at ends

required catenary force to balance the applied load is smaller. (2) At the same maximum beam temperature, the beam with non-uniform temperature distribution will have a higher tensile strength than that with uniform temperature distribution.

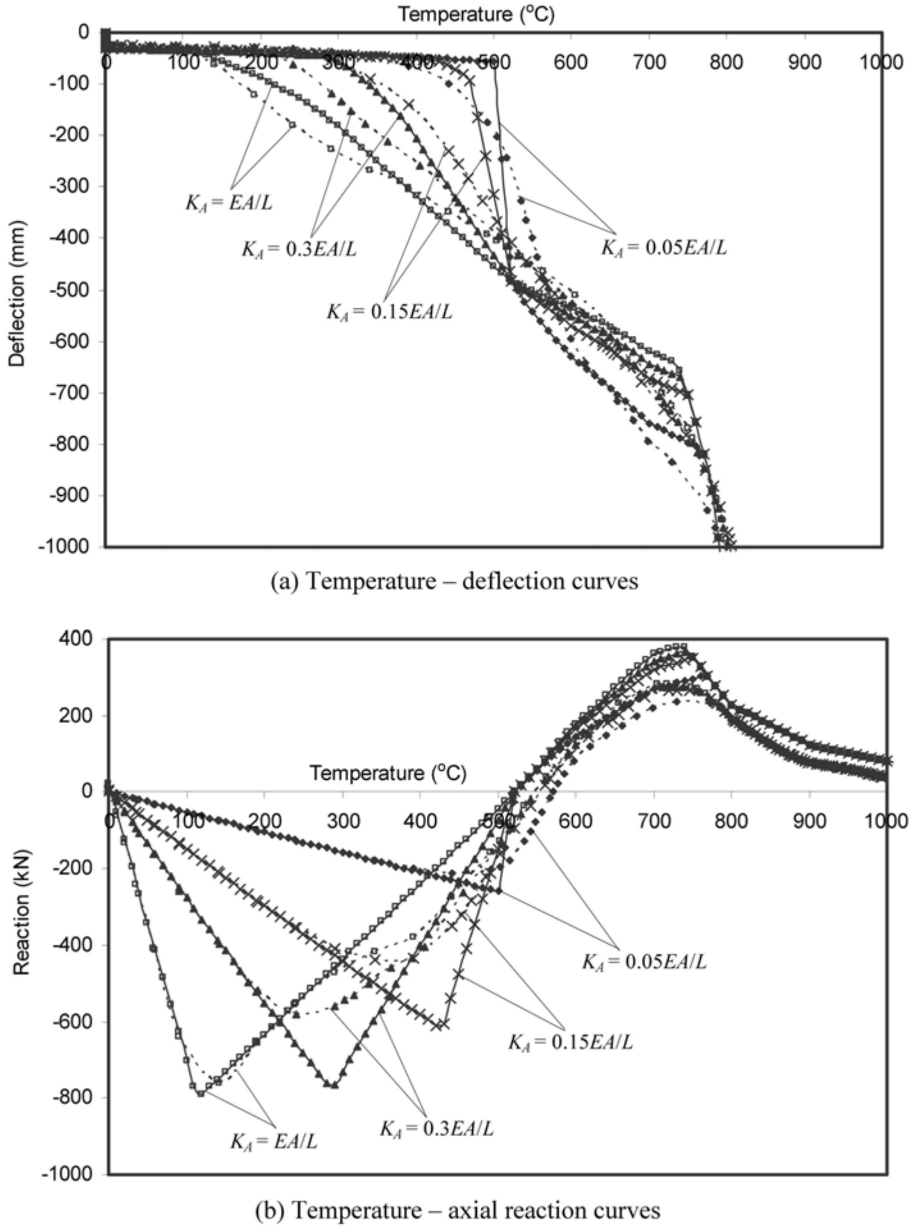


Fig. 5 Comparison between results of ABAQUS (broken lines) and HCM (solid lines) for beams with different axial restraint stiffness. Uniform temperature distribution, CPL, free end rotation, load ratio=0.7

Because the beam's tensile resistance is higher than the catenary force, this will avoid using the approximate axial force – bending moment plastic interaction equation, enabling the proposed simplified method to give better agreement with numerical simulations under non-uniform temperature distribution than under uniform temperature distribution, as demonstrated by comparing the results in Figs. 3, 5 and 7 to those in Figs. 4, 6 and 8 respectively.

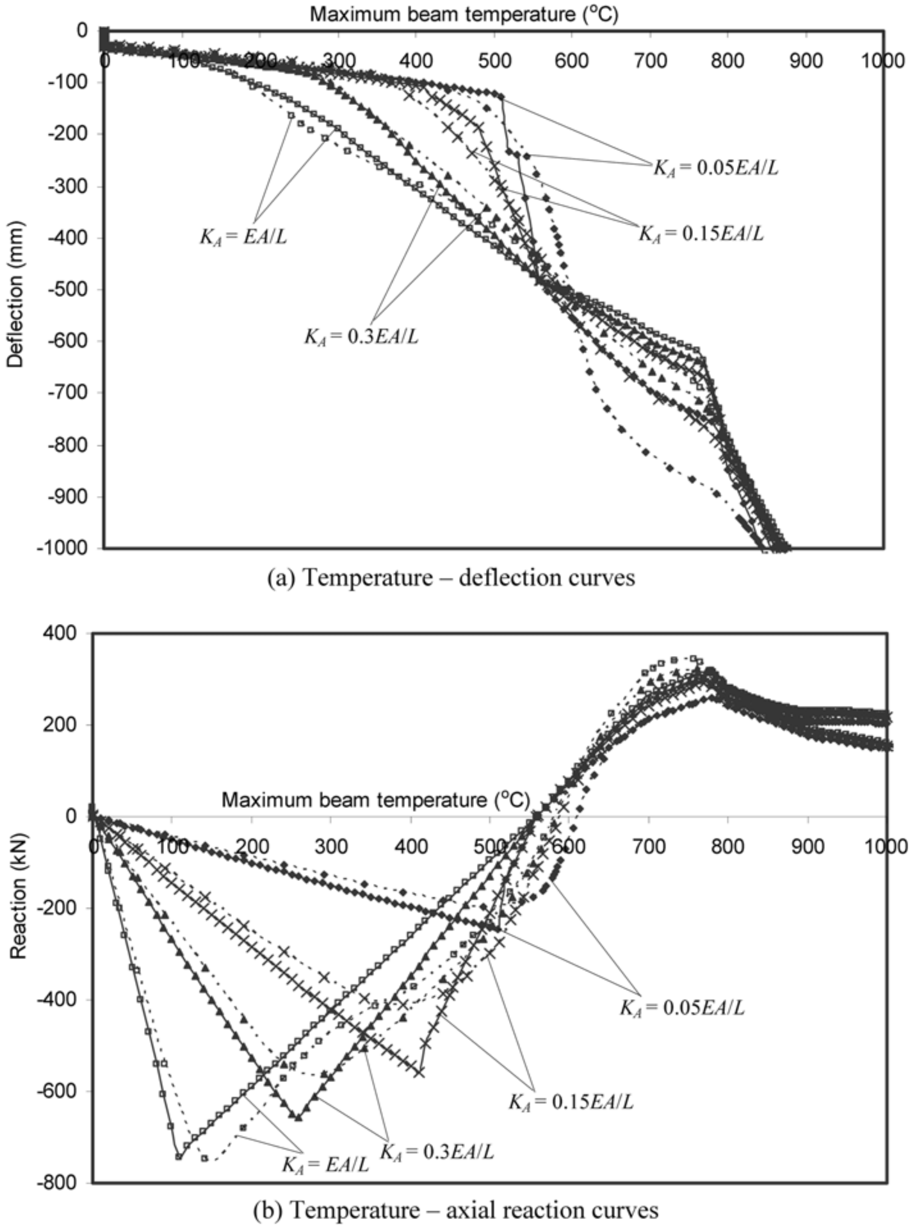


Fig. 6 Comparison between results of ABAQUS (broken lines) and HCM (solid lines) for beams with different axial restraint stiffness. Non-uniform temperature distribution, CPL, free end rotation, load ratio=0.7

4. Practical concerns

The results in Figs. 3-8 suggest that an axially restrained steel beam in catenary action would have no practical limit to its resistance to high temperatures in fire. This makes it possible to eliminate fire protection to the steel beam without causing a structural collapse. However, practical exploitation of

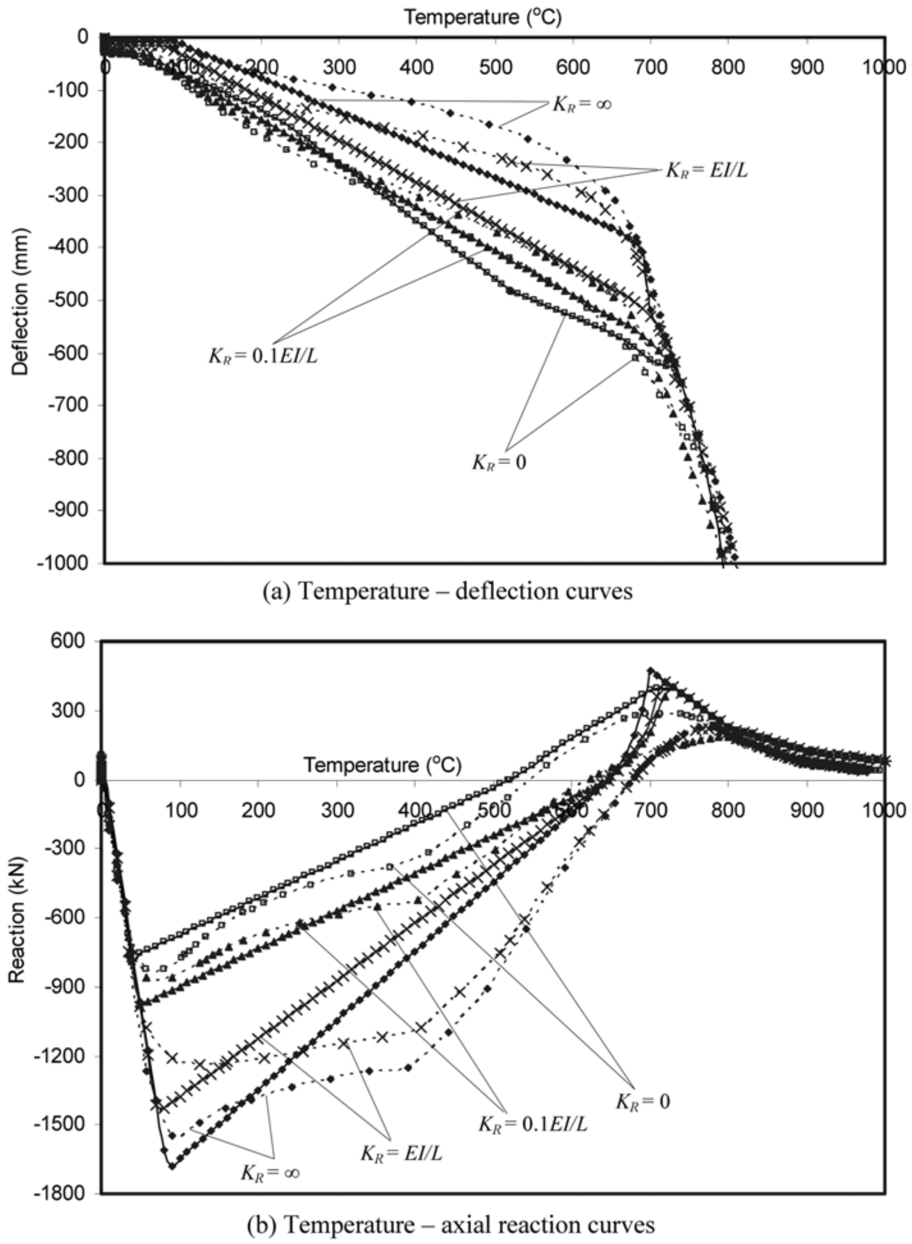
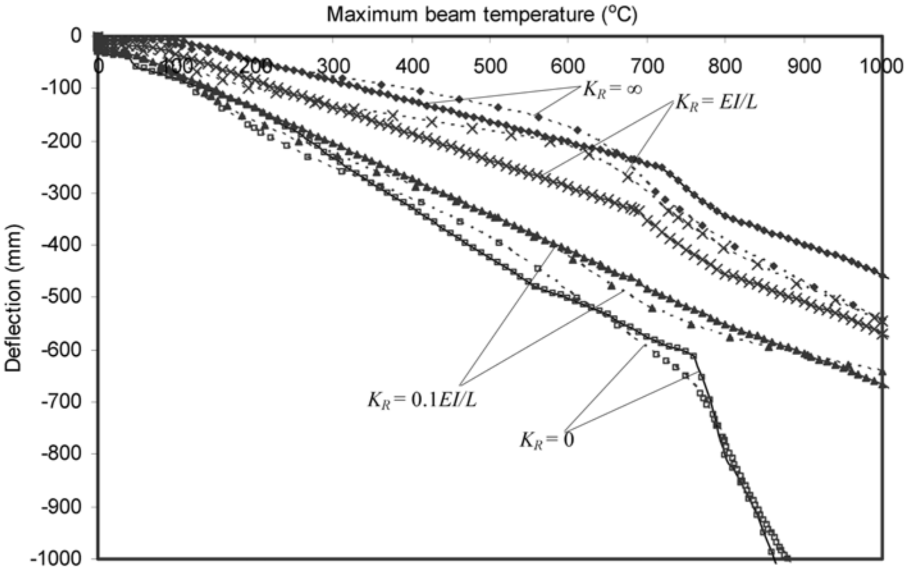
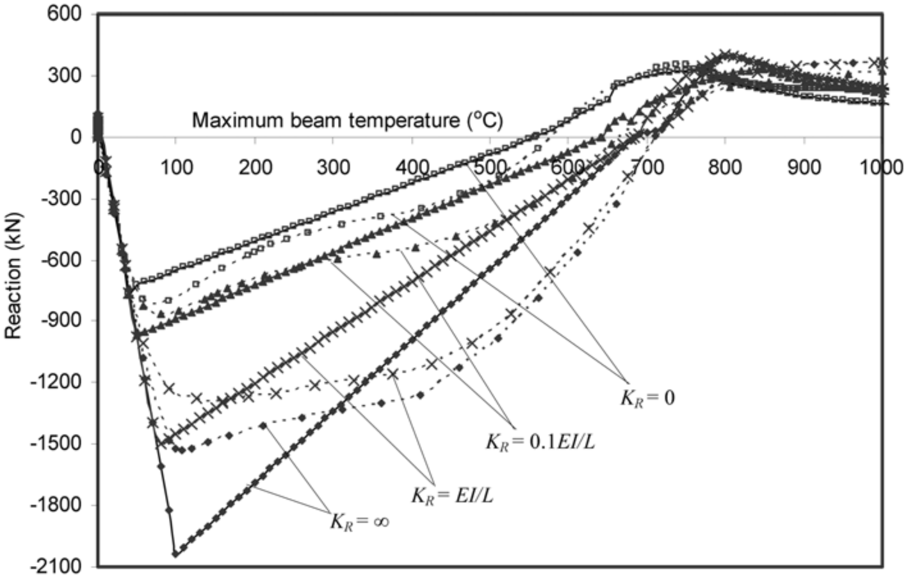


Fig. 7 Comparison between results of ABAQUS (broken lines) and HCM (solid lines) for beams with different rotational restraint stiffness. Uniform temperature distribution, CPL, fully axial restrained at ends, load ratio=0.7

catenary action should take into consideration other important aspects of fire resistant design of structures. Since there is very little quantitative information presently available to help translate these practical considerations into a design method, this section will only provide a discussion of these issues and suggest how further studies may be undertaken to help resolve these issues.



(a) Temperature – deflection curves



(b) Temperature – axial reaction curves

Fig. 8 Comparison between results of ABAQUS (broken lines) and HCM (solid lines) for beams with different rotational restraint stiffness. Non-uniform temperature distribution, CPL, fully axial restrained at ends, load ratio=0.7

4.1. Integrity of fire resistant compartment

The objective of fire resistant design is to prevent fire spread from the original fire compartment. Thus, not only should the structure be stable in fire, the integrity of the fire compartment should also be maintained. Therefore, if the steel beam forms part of a fire resistant compartment, the beam behaviour should not breach integrity of the fire resistant compartment. At present, it is not possible to quantify integrity of fire resistant compartments. A current attempt is to link integrity requirement of a fire resistant compartment to the maximum deflection of the beam. All standard fire resistance test methods (e.g. BSI 1987) specify a maximum beam deflection of span/20. However, this is thought to be related to the limit of the standard fire resistance test furnaces to safely accommodate large deflections of the standard test beams without causing damage to the furnace. This deflection limit should not be considered to have anything to do with integrity of the fire resistant compartment. In fact, the latest compartment fire test in the Cardington 8-storey steel framed structure suggests that the maximum steel beam deflection was over span/10, yet there was no indication of fire compartment failure (da Silva *et al.* 2004).

Following on from the much-publicised Cardington structural fire tests, the UK's Steel Construction Institute (SCI) published a new fire safety design guide (Newman *et al*. 2000) which gives some advice on beams in the wall plane and through walls. In the former case, the SCI guide suggests that "unprotected beams above and in the same plane as separating walls, which are heated from one side only, do not deflect to a degree that would compromise compartment integrity, and normal movement allowances are sufficient". In the latter case, the SCI guide recommends "that a deflection allowance of span/30 should be provided in walls crossing the middle half of an unprotected beam. For walls crossing the end quarters of the beam, this allowance may be reduced linearly to zero at end supports". The SCI guide does not give any advice on how much a steel beam is allowed to deform if the separating walls are at the ends of the steel beam.

Recently, the UK's Office of Deputy Prime Minister (ODPM) commissioned a study to investigate "Integrity of compartmentation in buildings during a fire (http://www.steelinfire.org.uk/downloads/BREfireconferenceTL1.pdf). However, the scope of this study was rather limited and it was not possible to resolve the issue of quantifying integrity of fire resistant compartments.

The issue of fire integrity is clearly emerging as one important fire resistant design consideration. Imposing a blanket cover of span/20 by following the standard fire test specification is likely to restrict practical applications of advanced structural behaviour such as catenary action. To develop rational performance based design methods, more research is needed to gain thorough understanding of this subject.

4.2. Connection behaviour

It should be pointed out that this study is limited to beam behaviour during the heating phase. Connections have been noted to fracture during the cooling phase. Clearly, this is an issue that should be properly resolved. Nevertheless, the main focus of this study is to determine whether existing simple connections would be adequate if catenary action during the heating phase of beam behaviour is to be utilised in fire resistant design. In this context, it is not the objective of this paper to research on more robust connections that will have sufficient strength to resist catenary forces. Also during the heating phase, the connection may be subjected to a compression force. However, this compression force is not dealt with in this paper because (1) this phase of beam behaviour occurs prior to the onset of catenary action, therefore the methodology of dealing with this compression force would be exactly the same irrespective whether or not catenary action is considered in design; (2) a compression force in the connection would still ensure that the beam is in contact with the surrounding structure so it is not considered to be as damaging as a tension force in the connection.

To check whether more substantial connections are necessary to resist the maximum catenary force in the beam, a common type of simple steel connection (flexible end plate connection) is considered. Figs. 4, 6 and 8 indicate that the maximum catenary force in the beam under analysis is about 400 kN. Using the common, simple flexible end plate connection with industrial standard M20 Grade 8.8 bolts, and assuming that the connection components in fire maintain 50% of their load carrying capacity at ambient temperature, 4 rows of bolts (total No. 8) would be required to carry the maximum catenary force in the beam. This can be easily accommodated. Weld resistance is more difficult to satisfy, with the example beam in Figs. 4, 6 and 8 requiring a weld throat thickness of about 7 mm (weld leg length about 10 mm). Nevertheless, this demand may not be particularly demanding, which implies that although connections should be suitably designed to accommodate the maximum catenary forces in the beam, it is unlikely to be necessary to require substantial strengthening of connections over and above the requirements of ambient temperature design.

4.3. Cost analysis

On the assumption that there is no technical obstacle to practical application of catenary action to eliminate fire protection to steel beams, one important decision would be whether or not the alternative design method would be cost-effective. The following examples briefly illustrate the application of the simplified calculation method to assess the possible savings of utilising catenary action in fire engineering design in different types of steel frame. Two methods are compared: the limiting temperature method (BSI 1990) and using catenary action. The first method will require fire protection to the beams. The second method will not require fire protection to the steel beams but will need larger external columns. These examples are based on a 2-D structure. Consideration for a 3-D structure would be similar, but will need to include the steel frame in the 3rd direction. It should be recognised that when utilising catenary action, since an internal column would be subjected to the same catenary force on both sides, there is no additional force in the internal column. Only the edge columns would need to be larger than those used in the first method. In these examples, the increased connection costs were not included as it has been demonstrated that this would not be significant.

Consider three plane steel frames, 2-bay by 2-storey, 20-bay by 5-storey and 5-bay by 20-storey. Each storey is 4 m high and each beam is 6 m in span. The frames are assumed to be simply constructed, with 30 kN/m dead load and 15 kN/m imposed load uniformly distributed on the beams. The beams are assumed to be fully laterally and torsionally restrained by the floor slabs so that there is no lateral torsional buckling. The frames are required to have 60 min standard fire resistance. Spray fire protection is used at a cost of £10.98/m^2 (DLE 1997). The fabricated steel structural cost is assumed to be £900/ton.

Structural sizing according to BS5950 Part 1 (BSI 2000) at ambient temperature:

The design beam load is $q = 1.4 \times 30 + 1.6 \times 15 = 66$ kN/m, giving

$$M = \frac{qL^2}{8} = \frac{66 \times 6^2}{8} = 297 \text{ kNm and } W_p = \frac{M}{f_y} = \frac{297 \times 10^6}{275} = 1080000 \text{ mm}^3$$

Select UB457×152×60, giving $W_p = 1287000$ mm^3 and $M_p = 354$ kNm

Limiting temperature according to BS 5950 Part 8 (BSI 1990) Under fire conditions:

$q = 1.0 \times 30 + 0.8 \times 15 = 42$ kN/m, giving $M = 189$ kNm

This gives a load ratio =189/354 = 0.53. For a three-side heated beam, this gives a limiting temperature of 641°C (BSI 1990).

Table 2 Indicative costs of steel frames without and with catenery action

Beam: UB457×152×60					
Steel cost (£900×L×den_L): £322.9			Fire protection cost (£10.89×L×H_p): £98.3		
2-Bay by 2-Storey (4 beams + 2 internal columns + 4 external columns)					
No catenary action (All members protected)			Catenary action design (Beam unprotected)		
IC	Type	UC152×152×30	IC	Type	UC152×152×30
	S cost	£108.0		S cost	£108.0
	FP cost	£39.8		FP cost	£39.8
EC	Type	UC152×152×30	EC	Type	UC305×305×158
	S cost	£108.0		S cost	£569.2
	FP cost	£39.8		FP cost	£81.3
Total cost		**£2571.6**	Total cost		**£4189.2**
Saving: 0%	Unprotected steel: 1435 kg		Protected steel: 2770 kg		
20-Bay by 5-Storey (100 beams + 95 internal columns + 10 external columns)					
No catenary action (All members protected)			Catenary action design (Beam unprotected)		
IC	Type	UC254×254×89	IC	Type	UC254×254×89
	S cost	£320.0		S cost	£320.0
	FP cost	£66.4		FP cost	£66.4
EC	Type	UC203×203×52	EC	Type	UC305×305×158
	S cost	£187.2		S cost	£569.2
	FP cost	£52.9		FP cost	£81.3
Total cost		**£81238.5**	Total cost		**£75512.5**
Saving: 7.1%	Unprotected steel: 35880 kg		Protected steel: 40106 kg		
5-Bay by 20-Storey (100 beams + 80 internal columns + 40 external column)					
No catenary action (All members protected)			Catenary action design (Beam unprotected)		
IC	Type	UC305×305×240	IC	Type	UC305×305×240
	S cost	£864.0		S cost	£864.0
	FP cost	£84.2		FP cost	£84.2
EC	Type	UC305×305×158	EC	Type	UC305×305×158
	S cost	£569.2		S cost	£569.2
	FP cost	£81.3		FP cost	£81.3
Total cost		**£143996**	Total cost		**£134166**
Saving: 6.8%	Unprotected steel: 35880 kg		Protected steel: 102096 kg		

*IC: internal column, EC: external column
*S cost: steel cost, FP cost: fire protection cost

For an UB457×152×60 beam requiring 60 min standard fire resistance, the maximum temperature is 936°C. Use the simplified calculation method to analyse the beam, assuming the beam is fully axially restrained and under a non-uniform temperature distribution with the top flange temperature being 0.7 that of the bottom flange and web. Up to 936°C, the maximum catenary force in the beam is 337 kN and the maximum deflection of the beam at 936°C is 920 mm. The maximum catenary force in the beam will produce a bending moment of 337×8/4 = 674 kNm in the external columns. Compared to design without considering catenary action, this will require larger external columns, but the internal columns will be the same. Table 2 lists the sizes and indicative costs of the three frames using both design methods.

It can be seen from Table 2 that for the small scale frame (2×2 bays), there is no advantage in using catenary action in design. But for the two large scale frames, the savings of utilizing catenary action to eliminate fire protection to the steel beams can be quite impressive. If more expensive fire protection systems, such as intumescent coating, are used in the first design method, the savings by utilising catenary action can be greater. In tall buildings where large columns of steel tubes filled with concrete are used, it may not be necessary to increase the external column size and more savings in fire protection cost could be had by using catenary action in design. Furthermore, if the required fire resistance is greater than 60 minutes that is used in this example, which would be the case for tall buildings, the above analysis may be repeated to demonstrate that it is more advantageous to utilise catneary action. This is because the fire protection cost to the steel beams will be higher but the maximum catenary force in the beam will be the same since the maximum catenary force will be reached long before the unprotected steel beam has been heated by the standard fire for 60 minutes.

5. Conclusions

This paper has presented the development and validation of a simplified calculation method for predicting beam large deflection behaviour under elevated temperatures. Validation examples have been provided for beams with different temperature distributions, under different loading conditions and with different end axial and rotational restraints. This paper also discussed some of the more important obstacles to practical application of catenary action to eliminate fire protection to steel beams. Some examples are provided to illustrate the type of steel framed structure that could benefit the most from exploiting catenary action. The main conclusions of this paper are:

- In the simplified analysis, the beam's deflection profile is an important assumption for calculating the beam's axial load. In the proposed model, the beam's deflection is a summation of its mechanical deflection and thermal bowing deflection, and it depends on the beam's loading condition, end rotational restraint and temperature distribution in the cross-section.
- In the proposed method, both the axial force and bending moment of the beam are calculated assuming elastic behaviour initially. When the beam has fully yielded under combined axial load and bending moment, the beam's axial force is calculated first. The beam's bending moment is then obtained from the beam's axial force - bending moment interaction equation. There is no check of stress condition or strain compatibility.
- Comparisons between analytical calculation results and ABAQUS numerical simulation results indicate that the proposed method provides a good approximation to the more complicated numerical analysis. A main feature is that the simplified method over-predicts the beam's catenary force. This is a direct result of the way in which the beam's axial forces and bending moments are calculated, as described in the previous paragraph. However, the proposed method will always predict high catenary forces, which would lead to stronger connections and supporting structures than necessary to resist the catenary forces, therefore being on the safe side. Under non-uniform distribution in the cross-section, due to the additional thermal bowing deflection and higher tensile capacity, the beam's catenary force will usually not reach the beam's tensile strength. Since this will avoid using the approximate axial force – plastic bending moment interaction equation, it will enable the proposed method to give better agreement with numerical simulations under non-uniform temperature distributions than under uniform temperature distribution.

- The proposed method can trace the deflection and catenary force variation as a function of the beam temperature and may be adopted as a design method to enable engineers to exploit catenary action in fire resistant design of beams, where the main design decisions are likely to be the maximum catenary force that has to be resisted by the adjacent structure and the maximum beam deflection to assess whether this can be tolerated without causing integrity failure of the fire resistant compartment.
- Connection behaviour is unlikely to restrict practical application of catenary action in fire resistant design. However, further studies are necessary to resolve whether or not large deflections of the steel beams would cause integrity failure of the fire resistant compartment.
- The benefit of using catenary action is most likely to be useful in high-rise and/or multi-span buildings. Particularly, using catenary action would benefit buildings with high fire resistance requirement.

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