STRUCTURAL PERFORMANCE OF REDUNDANT STRUCTURES UNDER LOCAL FIRES

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ABSTRACT

Most rules and investigations of the strength of structural members under fire assume that the member acts alone as an isolated structure. This matches the testing of individual members in a standard furnace test. The concept may seem appropriate where fire in a compartment effectively attacks only the individual structural members nearby. However, no account is taken of the interactions which inevitably occur with the surrounding structure. Where the complete structure is large and redundant, these interactions can completely change the structural response and effectively invalidate the design assumptions.

This paper discusses the response of a structural element under fire within a highly redundant structure, such as a large building. The behaviour of the element under fire is strongly affected by the restraint provided by the surrounding parts which are not subjected to heating. A number of responses in quite simple structures are shown, to illustrate the roles of expansion, loss of material strength, the relative stiffness of adjacent parts of the structure, development of large deflections, buckling and temperature gradients. These aspects are illustrated with simple examples, and it is shown that there are several counter-intuitive phenomena in structures of this kind. The significance of these findings for the design of large buildings is explored briefly.

INTRODUCTION

The fire resistance assessment of structures is currently based on fire tests on single elements, evaluated in terms of the time to failure. The conditions in these fire tests are not well related to the conditions prevailing in real fires. Not only is the fire test scenario artificial, but the structural idealisation is also unrealistic.

A determinate structure is one in which the pattern of internal forces and stresses can be determined using equilibrium considerations alone. Most tests on isolated members match this condition. A redundant structure is normally defined as one in which the pattern of forces and stresses within the structure cannot be determined by equilibrium alone, but depend instead on the relative stiffnesses of parts of the structure. Under collapse conditions, determinate and redundant structures are more sharply differentiated. The determinate structure collapses when the most highly stressed region reaches the local strength, appropriately reduced according to its current thermal state. By contrast, provided it has adequate ductility and does not suffer from instability, the redundant structure may find different load paths by which to support additional load when its local strength is reached at a single location. Where a structure is very redundant and there are many alternative load paths, large deformations can develop without a loss of its capacity to carry the imposed loads, and failure must be defined in a different way. This problem is faced in fields other than fire: researchers in pressure vessels and rectangular storage structures are also trying to find new failure definitions which can accommodate large displacement concepts.

Complex structural interactions take place during fires in highly redundant framed structures, which typically possess a high degree of static indeterminacy or redundancy (and hence offer several alternative load paths) leading to an extensive redistribution of loads. This phenomenon creates sufficient reserve capacity to allow most such structures to survive fires with little structural damage. As this behaviour results from interactions between most of the structural elements as an ensemble, it is necessary to consider such structures as an integrated whole when their fire resistance is being evaluated. Although this fact has been recognised for some time (as demonstrated by the fires in Broadgate Phase 8 in June 1990 [1], Mercantile Credit Bank Basingstoke and Minster Court), it is only recently that full-scale fire tests on realistic structural configurations have been carried out. The Cardington tests undertaken by British Steel (Swinden Technology Centre) and BRE represent a major landmark in the field.

The response of structures to fire depends upon the materials used in construction, the response of individual structural elements and the behaviour of the structural system used to connect the elements. In addition to the above the response of structures naturally depends upon the fire itself: its sequence, severity, spread and rate of development. The common assumption in structural fire resistance design is that fires will be localised through effective compartmentation. In the absence of this assumption, it becomes very difficult to predict the behaviour of the structure, mainly because the fire behaviour in such cases is quite uncertain, so the heating regime and sequence in the structure is unknown. A prime objective of the tests conducted at Cardington on the BRE 8-storey frame was to provide data to aid the development of understanding of the interactions between different structural mechanisms which determine the overall behaviour of composite steel frames in fire.

The data from the Cardington tests is currently being used to validate numerical models which are under development in a DETR funded project (through the PIT scheme) led by the University of Edinburgh and in collaboration with British Steel, Imperial College, SCI and BRE. The overall objective of the project is to develop analytical tools that adequately model the structural behaviour of composite steel frames in fire. One key observation from the tests is that composite steel-concrete frames have significantly more fire resistance than individual structural steel members, due to their ability to redistribute loads to relatively stiffer parts of the structure. Observations of real fires and fire experiments have long shown that such redistributions clearly exist. These redistributions are now being observed in large displacement numerical models of the Cardington tests.

If numerical models are to reveal the changing patterns of load transfer in the structure during the fire, they must include adequate representations of all the phenomena. A demonstration that a model predicts an observed deflection pattern against the known temperature history does not necessarily validate the model. It must also identify the key events during the fire, reveal the structural phenomena responsible for them and justify the conclusions using the fundamental principles of structural mechanics.

This paper gives a brief account of some of the underlying phenomena occurring in highly redundant framed structures exposed to fire which influence the complete response. The individual phenomena described here all influence the behaviour of the structure, leading to a complex total pattern which is not simple to interpret.

IMPORTANT PHENOMENA IN REDUNDANT COMPOSITE STRUCTURES

In this section, some key features of the response of a typical element of a redundant structure under fire are identified. The discussion is restricted to those which are, perhaps, less obvious. The structural elements in a building which experience the effects of fire most directly are the beams and floor slabs of the floor above the fire compartment. For this reason, the following discussion is very much focused on the behaviour of beams and slabs.

Thermal expansion

Beams are designed to carry their loads by bending and shear. In fire, significant axial forces develop if the beam is fully or partially restrained against axial expansion (or contraction during cooling). Depending on the surrounding structure, these forces can be either beneficial or deleterious to the performance of the structure. Floors are also designed to carry loads in bending and shear. When they expand, floor slabs can exert enormous forces on the surrounding structure.

Where high levels of restraint exist, the thermal expansion forces can become very large. For a fully restrained steel element compressive yield under thermal expansion occurs at a temperature of only

$$\Delta T_y = \frac{\sigma_y}{E\alpha} \tag{1}$$

in which ΔT_y is the temperature rise to cause yield, α is the thermal expansion coefficient and E is the elastic modulus of steel. This relationship shows that a temperature change of 102 degrees C for 250 grade steel and 142 degrees for 350 grade steel (ignoring any material degradation) is needed to achieve yield. These temperatures are so low that it is clear that there is plenty of scope for high stress development in real fires even when the restraint is only partial.

A heated structure can respond to thermal expansion with a variety of responses. Because structural engineers are trained from the outset to relate stresses to deflections and deflections to material properties, the more complex responses with thermal expansions sometimes give surprises. Indeed, because the literature on fire responses is mostly concerned with determinate structures in which these connections continue to exist, the importance of thermal expansion strains are often lost.

The key relationships which are needed to understand redundant structural behaviour under fire are

$$\varepsilon_{\text{total}} = \varepsilon_{\text{thermal}} + \varepsilon_{\text{mechanical}}$$

$$\text{with} \qquad \varepsilon_{\text{mechanical}} \to \sigma \qquad \text{and} \qquad \varepsilon_{\text{total}} \to \delta \tag{2}$$

The total strains govern the deformed shape of the structure δ, through kinematic or compatibility considerations. By contrast, the stress state in the structure σ (elastic or plastic) depends only on the mechanical strains.

Where the thermal strains are free to develop in an unrestricted manner and there are no external loads, axial expansion or thermal bowing results from

$$\varepsilon_{\text{total}} = \varepsilon_{\text{thermal}} \qquad \text{and} \qquad \varepsilon_{\text{total}} \to \delta \tag{3}$$

By contrast, where the thermal strains are fully restrained without external loads, thermal stresses and plastification result from

$$0 = \varepsilon_{\text{thermal}} + \varepsilon_{\text{mechanical}} \qquad \text{with} \qquad \varepsilon_{\text{mechanical}} \to \sigma \tag{4}$$

Most situations in real structures under fire have a complex mix of mechanical strains due to applied loading and mechanical strains due to restrained thermal expansion. These lead to combined mechanical strains which often far exceed the yield values, resulting in extensive plastification. The deflections of the structure, by contrast, depend only on the total strains, so these may be quite small where high restraint exists, but they are associated with extensive plastic straining. Alternatively, where less restraint exists, larger deflections may develop, but with a lesser demand for plastic straining and so less destruction of the stiffness properties of the materials.

These relationships, which indicate that larger deflections may reduce material damage and correspond to higher stiffnesses, or that restraint may lead to smaller deflections with lower stiffnesses, can produce example problems and structural situations which appear to be quite counter-intuitive for most structural engineers.

Thermal buckling

When an elastic beam with rigid axial restraint at its ends is heated, compressive stresses develop according to Equation 2. These cause it to reach a bifurcation state when the thermal thrust attains the classical buckling load

$$EA\alpha\Delta T = \frac{\pi^2 EI}{l^2} = \pi^2 EA\left(\frac{r}{l}\right)^2 \tag{5}$$

where l is the effective length of the beam and depends on the restraint conditions. The critical buckling temperature rise ΔT_{cr} in an elastic structure with unchanging elastic modulus E is thus,

$$\Delta T_{\text{cr}} = \frac{\pi^2}{\alpha}\left(\frac{r}{l}\right)^2 \tag{6}$$

For structural elements of the slenderness commonly found in slabs and beams, this critical temperature can easily be as low as 100 or 200 degrees C. The phenomenon is thus also likely to occur in most fires.

Where the elastic modulus is temperature dependent, the relationship cannot be so simply defined, since the thrust is a nonlinear integral of the thermal expansion and elastic modulus, whilst the stability is governed by a tangent modulus condition

$$\int E_T(T,\sigma)\alpha(T)dT = \pi^2\left(\frac{r}{l}\right)^2 E_T(T,\sigma) \tag{7}$$

in which $E_T(T,\sigma)$ is the tangent modulus which varies with the temperature and stress state.

Rigid axial restraint is generally impossible to achieve, and thus represents only a limit: real structures offer only finite restraint. Assuming that the restraint to axial expansion can be represented by a linear translational spring of stiffness k_t, (Figure 1(b)) again for an elastic beam with unchanging modulus, the compressive axial stress developed by thermal expansion becomes

$$\sigma = \frac{E\alpha\Delta T}{\left(1+\frac{EA}{k_t L}\right)} \tag{8}$$

The critical buckling temperature increment is modified from Equation 6 to become

$$\Delta T_{\text{cr}} = \frac{\pi^2}{\alpha}\left(\frac{r}{l}\right)^2\left(1+\frac{EA}{k_t L}\right) \tag{9}$$

From this relationship it can be seen that buckling and post-buckling phenomena should be observable at moderate fire temperatures (say 300 degrees C) in structures with translational restraint stiffnesses (k_t) which are quite comparable with the axial stiffness of the member ($\frac{EA}{L}$). This axial stiffness itself is reduced by heating through the reduction in E, so these post-buckling phenomena are very likely to be observed in beams in typical fires.

The effects of heating a beam between rigid axial end restraints (Figure 1(a)) whilst it is carrying a constant uniformly distributed load, are shown in Figures 2 & 3. A constant modulus elastic behaviour is adopted for clarity. The growth of the midspan deflection with temperature is shown in Figure 2(a). The distributed load smoothes the bifurcation phenomenon slightly, but the critical temperature can be clearly identified, and the post-buckling response involving rapid growth of deflections into a large deformation state can be seen. This growth of post-buckling deflections is rather different from traditional post-buckling under static loads. Under ambient temperature conditions, axial forces applied to a beam-column are largely unaffected by the member's response. Here, the axial forces develop only because the axial displacements are restrained, and the increasing deflections in the post-buckling state allow axial shortening through member curvature, and thus do not correspond to an unstable condition. The axial force developing in the beam under increasing temperature is shown in Figure 3(a). This force is close to constant in the post-buckling zone and additional thermal expansion is all absorbed in additional deflection, instead of causing increased stresses (see Equations 2-4 again). For local fires in real structures,

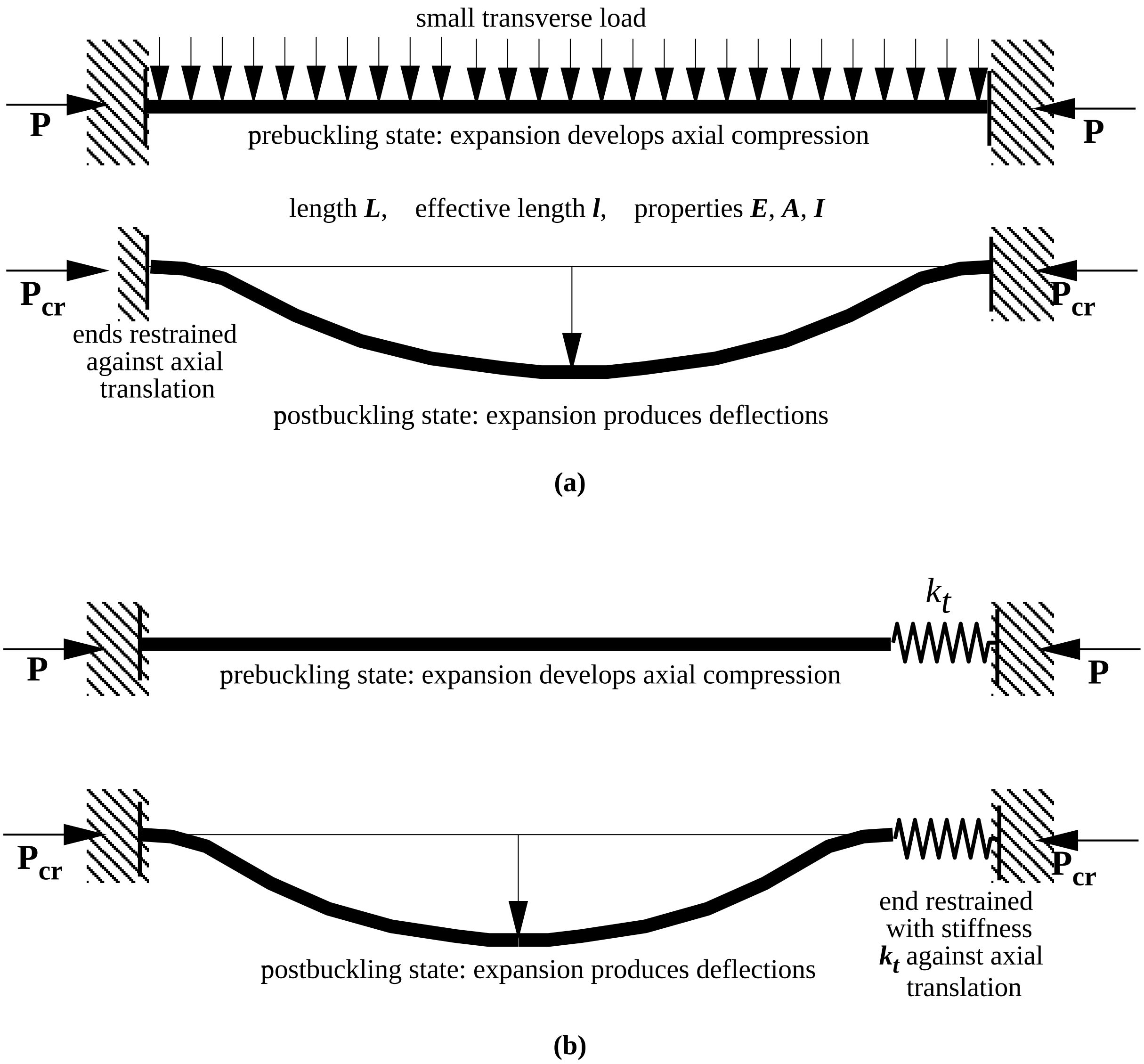
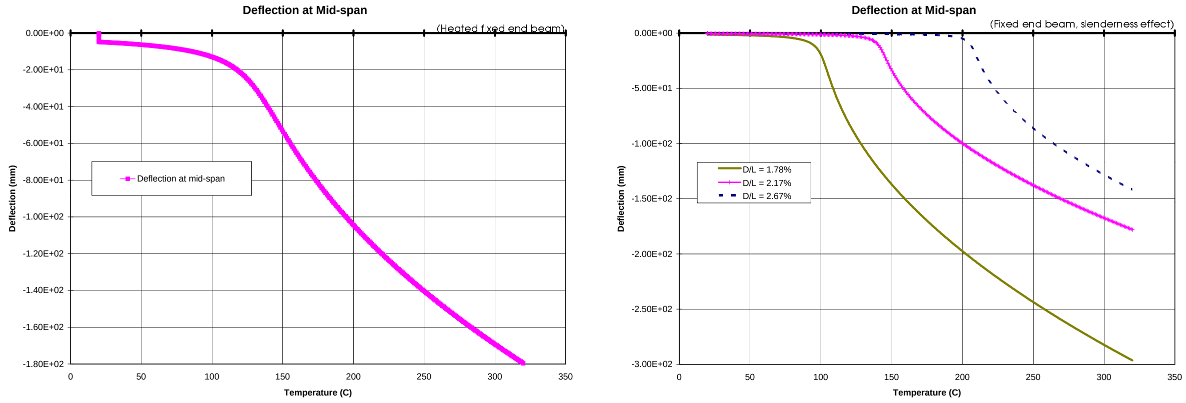


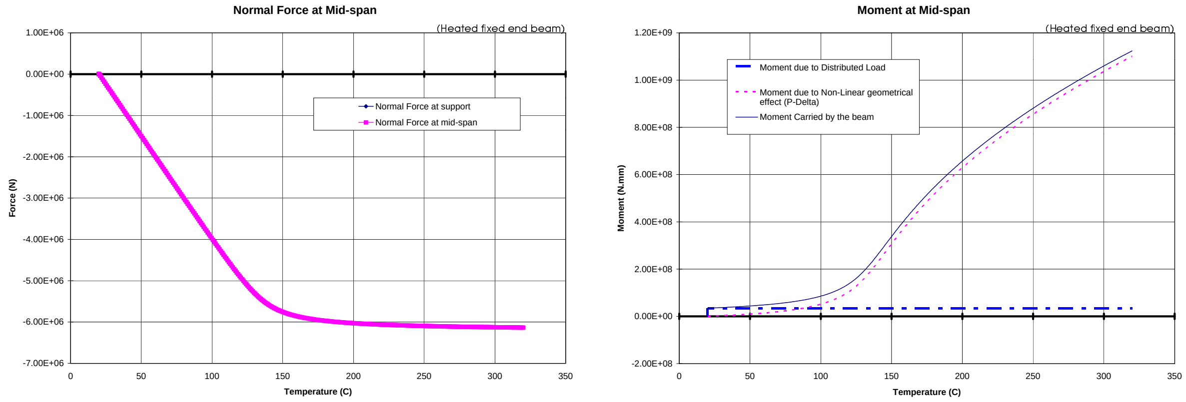
Figure 1: Axially restrained beams subjected to increasing temperature: (a) Rigid restraint, (b) Finite restraint



(a) **(b)**

Figure 2: Deflection of axially restrained elastic beams subjected to heating: (a) Single beam, (b) Three beams of varying slenderness

this is a helpful effect as it limits the additional forces generated by the restrained thermal expansion and thus does not damage adjacent parts of the structure so much. Thus, it can be concluded that *buckling is good for the structure*! It should be remembered that the material is assumed to be elastic and the stable axial force is associated with this. The moments developing in the beam as the temperature rises are



(a) **(b)**

Figure 3: Forces in an axially restrained elastic beam subjected to heating: (a) Axial Forces, (b) Moments

shown in Figure 3(b). The transverse loading moments are overwhelmed by $P-\Delta$ moments as the axial force and deflection rise: the stability of the post-buckling axial force means that this figure follows the deflection pattern The growth of deflections in three beams with different slenderness ratios is shown in Figure 2(b). The same response is seen in all three, with buckling temperatures at relatively low values. Space here does not permit an exposition of the relevant theory, but the transverse deflections due to thermal effects δ_y in this large displacement post-bucking regime can be very closely modelled by

$$\delta_y = 2L\sqrt{\frac{\alpha\Delta T}{\pi^2} - \left(\frac{r}{l}\right)^2} \tag{10}$$

Thermal gradients

As noted above, most real fires heat the floor and beams from below, leading to a regime in which temperature differentials develop between the upper and lower surfaces. These differentials lead to thermally induced bending or thermal bowing, which can increase deflections. A high temperature gradient through the depth of a floor slab (typically concrete) will induce either bending moments or additional deflections or both in the slab (see Equations 2-4 again).

The first example given here is of a beam which is axially and rotationally unrestrained at its ends and subject to a linear thermal gradient which is constant along the length. It produces a uniform curvature given by $\phi = \alpha T_{,y}$, where $T_{,y}$ is the gradient of temperature through the beam depth. No stresses are induced and a hot lower surface leads to downward bowing. If instead, the beam is rigidly restrained against end rotations (but axially free to translate), no deflections develop at all in the beam! It remains perfectly straight. Instead, a constant bending moment is induced throughout the beam (see Equations 2-4 again), given by $EI\alpha T_{,y}$. The stresses associated with a hot lower surface are the bottom fibre in compression, and in concrete first cracking occurs on the top unheated surface (this is a counter-intuitive result for most structural engineers).

It is difficult to achieve rigid restraint against rotation, except in the case of perfect symmetry and small rotations. In fires, large rotations in the slab occur and cracking occurs early in the concrete over the supports, reducing the rotational restraint. If the adjacent structure is represented by a rotational spring stiffness k_r at one end (whilst the other end is assumed to fixed), a moment is induced in the rotational spring of,

$$M_k = \frac{EI\alpha T_{,y}}{\left(1 + \frac{4EI}{k_r L}\right)} \tag{11}$$

where L is the length of the beam. Thus for substantial bending moments to be induced by thermal gradients, the rotational restraint stiffness must be comparable with the rotational stiffness of the beam itself when its far end is fixed (a perfect analogy with the axial case above). If the unheated adjacent spans are identical and have effectively fixed ends and the system is elastic, this restraint can be achieved. Where the hot surface is on the bottom, it should be noted that the thermal bowing deflection is still downward, but that tensile stresses develop on the top surface, and in concrete first cracking occurs on the top, producing an image which surprises most engineers.

When the beam is rotationally unrestrained, the thermal curvature due to a uniform gradient (with no net temperature rise), given by $\alpha T_{,y}$, causes a deflection δ_y in an axially free beam of

$$\delta_y = \frac{1}{\alpha T_{,y}}\left(1 - \cos\frac{\alpha T_{,y} L}{2}\right) \tag{12}$$

and, in a large displacement evaluation, this causes the distance between the supports to reduce by

$$\delta_x = L - \frac{2}{\alpha T_{,y}}\left(\sin\frac{\alpha T_{,y} L}{2}\right) \tag{13}$$

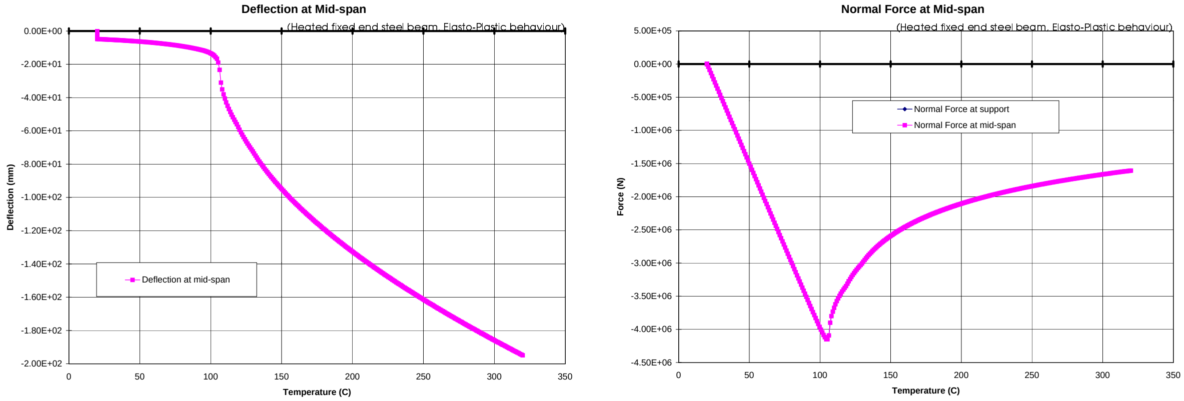
If the beam ends are now axially restrained, the loss of length in arc shortening δ_x must be replaced by a stress-related extension, which requires a uniform axial tension. Thus, for axially restrained but rotationally free beams (close to real conditions), thermal gradients will produce axial tension. By contrast, a uniform temperature rise produces axial compression.

Thus, the deformed shape gives little indication of whether part of the structure is in tension or compression, and a mixture of thermal gradients and centroidal temperature increases can lead to axial tension or axial compression with quite similar deformations. Some of these forces can participate in load-carrying mechanisms (under large displacement regimes), whilst others are purely self-stressing in character.

Degradation of strength and stiffness

The strength of a beam can be quantified from the geometry of its cross-section and its material strength. High temperatures will result in loss of strength (both yield and ultimate strengths) and stiffness (moduli of elasticity). Under inelastic conditions, the value of E in the above equations depends on other parameters. In the case of fire E depends on the temperature: most materials lose stiffness with heating.

The same beam as that of Figures 2 and 3, spanning between rigid end restraints, was given elastic-plastic properties. The changed behaviour is shown in Figures 4. The key difference is that the axial force rapidly drops shortly after buckling, as the midspan and end cross-sections reach the plasticity yield surface and are constrained to follow it. Increasing deflections then mean that the axial force must drop even if the moment were to remain constant, but the yield surface permits an increasing moment with falling axial force (Figure 5). The corresponding effects in a beam which remains elastic but whose properties degrade with temperature (as for steel) are shown in Figures 6. The same effects can be seen, but the consequences are less dramatic. In real structures, a mixture of thermal degradation of stiffness and yielding caused by thermal expansion occurs, and these effects are more difficult to separate out.



(a) **(b)**

Figure 4: Deflections (a), & Axial forces (b), in an axially restrained elastic-plastic beam

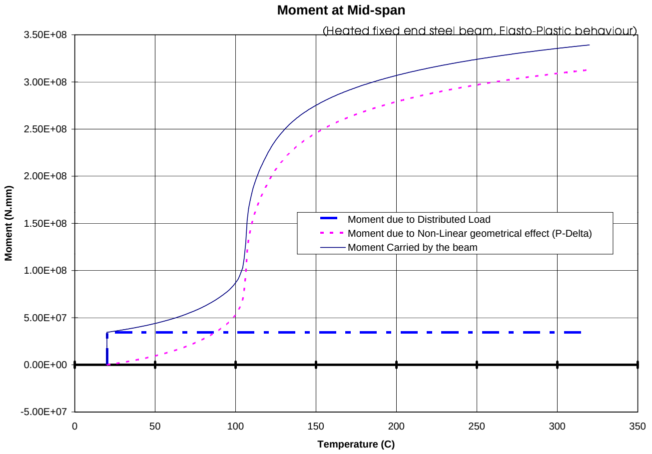
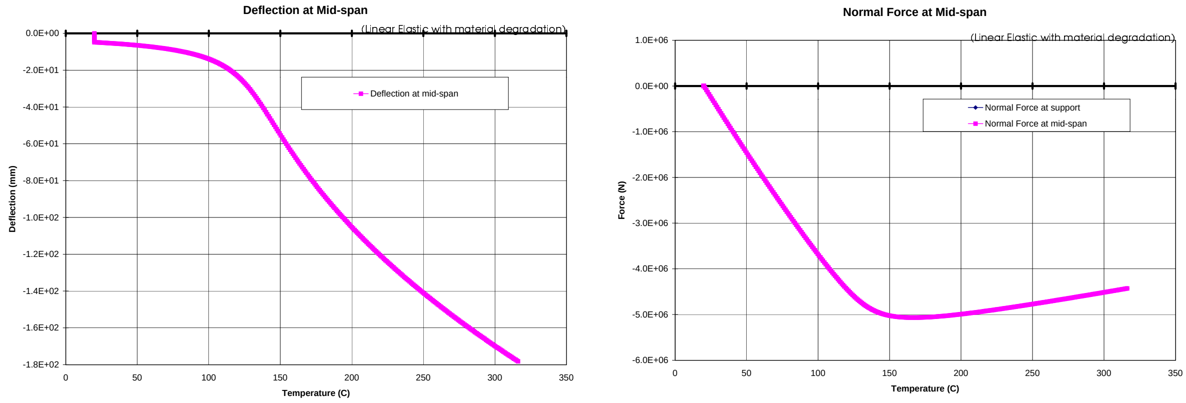


Figure 5: Moments in an axially restrained elastic-plastic beam

Large deflections



(a) **(b)**

Figure 6: Deflections (a), & Axial forces (b), in a restrained beam with reducing elastic stiffness

Under fire conditions, axially restrained beams develop large deflections for several reasons. Chief among these is thermal expansion, which induces enormous thrusts leading to post-buckling states (depending upon the restraint available) which force the beam down. In composite steel-concrete frames, this action begins in the rolled steel joist, until some cross-section(s) reaches the ultimate capacity. The principal source of restraint is the extensive and massive concrete slab, which has a high thermal inertia, and provides an extensive cold surround to the fire event. The action continues in the heated part of the concrete slab, which heats more slowly, with higher gradients, but to a much lower mean (middle surface) temperature. However, the forces the slab can exert are very high because a large cross-sectional area is involved. Since the slab expansion occurs in its own plane, and this is the plane in which the cold surrounding slab provides restraint, large thrusts can develop in two directions. Slab deflections can increase further through cracking leading to significant reductions in stiffness and end rotational restraint.

Paradoxically, reductions in the stiffnesses of the system may reduce deflections, because deflections are driven by the stiff restraint of strong thermally expanding structural elements. A reduction in axial stiffness reduces the thrusts that cause deflection.

Sources of restraint

The issue of how much restraint is available in a composite framed structure is important. A structural member which is far from the boundaries of the structure can be reasonably thought of as stiffly restrained. The restraint to interior elements comes from the surrounding cold composite slab and steel frame. If the fire compartment is sufficiently distant from the boundary (perhaps by one bay), the restraint this provides can be considered as almost rigid.

It is currently difficult to determine how large an edge zone must be if it is to provide a sufficiently stiff restraint and adequate anchorage for the slab forces required for alternative load carrying mechanisms. In the simplest global terms, the Cardington tests suggest that even edge panels can benefit enormously from transfer of loads from the weakening systems of bending and shear towards other mechanisms. When the fire compartment is at an edge or a corner, restraint may be provided by the in-plane shear stiffness of a protected composite edge beam. This may be assisted by a ”tension ring” force pattern in the edge composite beams, leading to a coupled flexure-tension behaviour. This effect can be seen clearly in Figure 7, which shows a plot of major principal stresses in the slab of a composite frame (modelled here using elastic shell elements). The internal heated zone is surrounded by a ring of membrane tension. An expanding heated steel beam which acts compositely with a slab is restrained by the relatively colder slab throughout a fire and can be severely plastified. The slab then becomes the major cause of thrust in

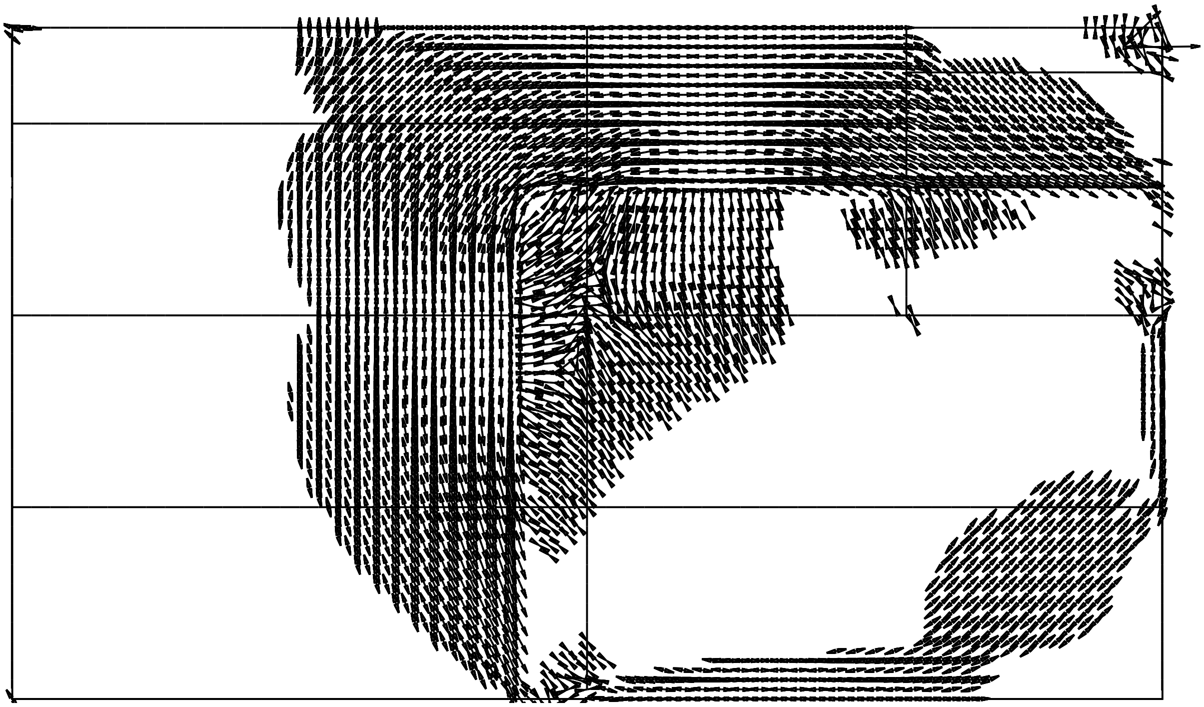


Figure 7: Tension ring restraint in the slab over a corner compartment under fire

the steel beam for most of the duration of the fire. This thrust leads to large plastic strains in the steel joist, causing significant shortening. On cooling, the reduction in length is not easily recovered, especially as the cooling steel gains stiffness and strength faster than the necessary tensile stresses are induced. Thus, plastification of the steel beam caused by slab restraint leads in due course to very high tensile stresses during the cooling phase. If rupture damage is to be avoided during this period, connections and weak linkages must be designed with tensile ductility in mind. It is normally far from a designer's mind to design beam to column or beam to beam connections in structural steelwork to be highly ductile when large tensions are applied to the joist (this connection is thought to be in compression as a result of negative bending!).

Membrane action

Two separate structural stress patterns in slabs are termed ”membrane action”. Both involve forces in the plane of the slab (membrane forces) acting at the middle plane of the slab plane. Both require the boundaries of the slab to be restrained to support forces in the plane of the slab (in the above discussion this was termed axial restraint). At small displacements, compressive membrane action occurs. When cracking occurs in concrete, the neutral axis or zero strain axis is displaced in the direction of the compression face. The middle plane of the slab is thus effectively subjected to an expansion. Such an effect can occur at both midspan in sagging bending and at supports in hogging bending, giving additive expansive displacements. Where these expansions are resisted by a stiff boundary, additional compressive forces develop, and where the slab is thick, the eccentricity of the compressive force transmission produces an arching action which can carry a greatly increased load. This mechanism is present in steel-concrete composite beams in highly redundant structures even under ambient conditions, due to the very large disparity between their hogging and sagging neutral axes. However, this action is more powerfully demonstrated in the thermally expanding slab of the composite structure, because the thermal expansions are very large and can cause major changes in the load-carrying mechanism.

At large displacements, tensile membrane action begins. In tensile membrane action, the large deformations of a slab with axial tensions (through thickness tensile forces), lead to a change in load-carrying by change of geometry: effectively a small component of the tension carries transverse load directly. Under ambient conditions, such large displacements mean that large mechanical strains have developed, and there is a danger of rupture due to loss of ductility. Under fire conditions, much of the necessary deformation (Equation 4) is derived from thermal expansion, placing a less onerous burden on mechanical strains, and consequent plastic straining or ductility. The post-buckling deformations described earlier in this paper assist in developing large displacements, and the three dimensional character of the slab, with the displacement field required to be continuous, permits tensile membrane action to develop even close to areas which are in a post-buckling compressive state. The restrictions which buckling places on the development of compression forces, together with the increased deformations, make the development of tensile membrane action easier to achieve.

These membrane mechanisms make the floor slabs the strongest structural elements in the building (in the sense that, under extreme conditions, they possess considerably greater strength than is required to carry the design loads). However, for these mechanisms to be mobilised, the floor must be adequately restrained along its edges by a surrounding structure that is relatively unaffected by fire.

WHOLE STRUCTURE RESPONSE AND IMPLICATIONS FOR FIRE DESIGN

Framed structures consisting of a grid of beams and columns supporting floor slabs represent the most common structural system. If adequate compartmentation is provided, this system is very effective in containing fires and redistributing loads away from the fire affected structural members to those less affected. In steel and concrete structures this is achieved principally through the slab, which possesses extraordinary reserves of strength, and bridges the loads over weakened beams and columns to the stronger members. The beam-column structural grid is also very effective in redistributing loads. These redistribution mechanisms allow a framed structure to continue to carry loads even though some of its members reach their ultimate strength: other less heavily loaded members take additional loads.

The single most important issue that determines a redundant structure's response to heating is the manner in which it deals with the inevitable thermal strains induced in its members through heating. If the structure lacks restraint to thermal expansion, the considerable strains are taken up in expansive displacements, producing a displacement-dominated response. This response occurs in structures whose behaviour is primarily determinate, and can have disastrous consequences, such as masonry walls being 'pushed over' by expanding steel members. This kind of event can be foreseen and designed against without compromising structural integrity.

In structures such as the composite steel frame at Cardington, the slab strongly restrains the thermal expansion strains and consequently develops large membrane compression and tension forces in the composite steel-concrete floor system. The membrane compressions can be limited by the large downward deflections which occur through thermo-mechanical post-buckling effects and thermal bowing (these are nonlinearly additive). The resulting behaviour is then a combination of displacement and force responses. The heated steel part of this composite system, if unprotected, rapidly reaches its axial capacity (through local buckling and strength degradation), and produces a beneficial effect by limiting and then reducing the total membrane compression, so allowing increased expansion of the steel through softening and ductility. This is clearly a desirable behaviour here, as it reduces the force imposed on the structure by the expansion forces and allows the damage to be localised.

It is clear that the worst scenario for a fire in a composite frame building structure is compartment breach. Structural fire design should define compartment breach as an 'ultimate limit state' and ensure that it is prevented. The only structural member in a composite frame that acts as a compartment boundary is the composite floor slab. As most of the slab near the fire is in membrane compression throughout the fire, a compartment breach of the slab is unlikely. However, local areas in membrane tension late in the fire could cause tensions cracks which are large enough to cause compartment breach. Appropriate reinforcement should be provided to prevent this, which will also aid load redistribution to cooler parts of the structure and enhance its intrinsic redundant behaviour.

Finally, it should be noted that where extensive plastic straining occurs (as in any steel joists which do not deflect greatly: see Equations 2-4 again), the plastic strains are difficult to recover without the development of very high tensile stresses. Shear connections at beam ends should be designed to be very ductile when subjected to a tensile force in the beam.

CONCLUSIONS

This paper has presented several phenomena which occur only in highly redundant building structures under localised fire conditions. They strongly influence the behaviour of the structure, and a lack of awareness of them can lead to serious misunderstandings about the structural phenomena occurring in fire tests.

Whilst material degradation is the key phenomenon in determinate structures under fire, for highly redundant structures the single most important factor is the effect of thermal expansion. Thermal expansion couples with large displacement effects to produce a number of effects which appear counter-intuitive to the conventionally trained structural engineer. The key roles of Equations 2-4 in decoupling the displacement and stress fields should be considered carefully wherever an understanding of behaviour is sought.

It is clear that large displacements in these structures are not always associated with degradation of the structure, but may indeed be beneficial. Large displacements are commonly associated with bending failures, but here they may occur with membrane thrusts, or with membrane tensions, depending on the thermal regime in the structure. The development of large deflections is helpful in that it permits the slab to transfer load to the alternative load carrying mechanism of tensile membrane action. This action can be regarded as more reliable under thermally induced deformations because the ductility requirement placed on mechanical straining is reduced.

These findings are of fundamental significance to the development of understanding of composite frames in fire. The implications for the development of design philosophies and procedures are considerable.

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