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Application of the SAFIR Computer Program for Evaluating Fire Resistance

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ABSTRACT

In this paper, the application of the computer program SAFIR to study the behaviour of a steel framed structure exposed to fire is illustrated. The salient features of the computer program SAFIR are described. Using the computer program, a case study is carried out on a four storey steel framed structure under various fire protection configurations. Results from the analysis are used to compare the behaviour of individual structural elements with that of a structural element, acting as part of the overall frame. It is shown that the fire resistance is enhanced when the overall structural behaviour concept is considered, rather than the present concept that is based on a single member.

INTRODUCTION

When exposed to fire, a steel structure gradually loses its stiffness and strength as a result of deterioration in the properties of steel. In practice, structures are generally designed for ambient (room) temperature; and fire protection is then added to satisfy the fire-resistance requirements specified in the building codes[1].

Fire resistance is determined on the basis of a standard fire test on single members, such as columns and beams. Since the standard fire tests are based on idealized loading and boundary conditions, single members do not represent the actual behaviour that would be expected if the member were an integral part of a structure. In other words, the behaviour of a single member in fire differs significantly from its behaviour when acting as part of a complete structure. Studies[2,3] have shown that higher fire resistance can be obtained by considering the overall response of the structure rather than that of a single member.

The traditional approach of applying fire protection to structural members, aimed at preventing structural collapse during fire, has typically been safe. With the cost of fire protection representing a substantial portion of the cost of the structural frame, engineers are seeking opportunities for a more cost-effective fire resistance design.

To develop such a cost-effective fire resistance design method, the current approach needs to be re-examined[4,5]. Such an examination will help to determine whether the current emphasis on the performance of an isolated member can be shifted to that of an overall structure, with

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the possibility of reducing or eliminating fire protection for some members while maintaining the required level of safety.

To understand the behaviour of structures under fire conditions, a number of analytical methods have been developed[6,7,8]. These methods range from a sectional analysis approach for single members to more complex methods where the overall behaviour of the structure is considered.

Sectional analysis approaches are less complex to use, however, their applicability is limited for an analysis based on a single element behaviour. To fully utilize the inherent strength present in the structure (building), a detailed analysis would have to be carried out based on the overall structure behaviour.

With recent advancements in computers and analytical methods, tools to model the overall response of the structure are being developed[5,9-13]. Generally; the finite element method is used (FEM) in developing approaches for evaluating the fire resistance of the overall structure. This method is versatile and has an advantage over the sectional analysis approach, since various factors affecting the behaviour of structures in fire (e.g., material nonlinearly, geometric nonlinearly, nonuniform temperature distribution and thermal strains) can be incorporated into the analysis. In addition, it enables the behaviour of complicated structures to be studied.

SAFIR is one such program that has been developed for tracing the behaviour of structures under fire conditions. In this paper, some of the main features of the program are described. The applicability of the program to the analysis of a steel frame is illustrated through a case study.

COMPUTER PROGRAM SAFIR

SAFIR is the second generation of the structural fire models developed in the 90's at the University of Liege, Belgium and is a general purpose computer program. An earlier program called CEFICOSS[14,15] (Computer Engineering of the Fire resistance of Composite and Steel Structures) had been developed previously in the 80's at the University of Liege. Full details of the SAFIR computer program are documented in Reference[16].

Description of Numerical Code

The computer program SAFIR is a finite element (FE) based program specifically developed for the analysis of structures under fire conditions. The program is a non-linear FE program used for studying the behaviour of structures exposed to fire with a step-by-step simulation. Although the program was originally dedicated to structures exposed to fire, it can also be used for an analysis of structures under ambient temperature conditions simply by modifying the contents of the temperature output file obtained from the thermal analysis stage.

The computer program SAFIR is made up of two components; namely, thermal analysis and structural analysis.

In the thermal analysis part of the program, plane sections, as well as a three-dimensional (3D) structure can be analyzed. Plane sections are discretized by triangular or quadrilateral elements, while 3D structures are discretized with solid elements. Varying material properties in elements can be considered in the analysis. The fire temperature, which is defined as a function of time, can either follow standard curves[17,18] or realistic fire growth curves that can be provided as input to the program. Heat transfer in the plane section or solid is by conduction and between the fire and the surface of the structure it is by convection and radiation. Temperature-dependent material properties, as well as the evaporation of moisture in a material can be considered in the analysis. Radiation in internal cavities of the section can be taken into account.

For the structural analysis part of the program, plane or 3D structures can be examined. The discretization can be accomplished by means of: (i) truss elements, made of one single material, having one uniform temperature per element; (ii) beam elements, either steel, reinforced concrete or composite; and (iii) solid (shell) elements. Large displacements can be considered in the analysis. Thermal strain effects (thermal restraints) and temperature-dependent non-linear material properties can be accounted for in the analysis. The program allows the introduction of imposed displacements, and residual stresses can be applied by means of initial strains. External supports may or may not be parallel to the global axes and nodal co-ordinates can be defined in cartesian or cylindrical systems of axes.

Although the computer program allows the simulation of three-dimensional (3D) structures, as highlighted above, only a plane frame structure is analyzed in this paper to keep the analysis simple. Hence, further details on the formulation and the assumption adopted for the beam element is outlined in the following sections.

Beam Element

The beam element is straight in its unreformed geometry and the displacement of the node line is described by the displacements of three-nodes. Two end nodes with three degrees of freedom per node-two translations and one rotation - and one node at mid-length supporting the non-linear part of the longitudinal displacement constitute a beam element. The longitudinal displacement of the node line is a second order power function of the longitudinal coordinate, while a third order power function of the longitudinal coordinate describes the transverse displacement of the node line.

The discretization of the cross-section in each beam element is made according to the fibre model by means of quadrilateral and/or triangular shaped elements. At every longitudinal point of integration–two or more per element – all variables such as temperature, strain and stress, etc., are uniform in each fibre. Each fibre can have its own material, allowing composite sections made of different materials to be analyzed.

Assumptions

The following assumptions are used in the development of the program:

- A plane section remains plain under bending; shear energy is not considered as per Bernoulli hypothesis.
- In the case of strain unloading, material behaviour is elastic with the modulus of elasticity equal to the Yung's modulus at the origin of the stress-strain curve.
- The plastic strain is not affected by an increase in temperature[19].
- Residual stresses are considered by means of initial and constant strains[13].

- The non-linear part of the strain is averaged on the length of the elements to avoid locking.

Analysis Procedure

For the analysis of a structure under fire conditions, two stages of analysis are required to trace the behaviour of the structure. The first stage involves a thermal analysis in which the temperature distribution in a member cross-section is determined. The second stage of analysis involves structural analysis. The temperatures due to fire are supplied as input data or through the use of standard time-temperature curves. Thermal analysis is carried out independently of the structural analysis and needs to be performed and results stored in a file.

For the structural analysis, the behaviour of the structure is simulated as a function of time using the temperature distribution evaluated during the thermal analysis. At each time step, an iterative technique is used to find the equilibrium between the external load and the internal stress. The tangent stiffness matrix is evaluated at each iteration and the system of equations are solved using the Newton-Raphson technique. For each time step, the iterations are repeated until the convergence is achieved. When the convergence is achieved, the following values are computed: (a) displacements of the structure at each node, (b) axial and bending moments at each integration point in each element, and (c) stresses, strains and tangent modulus of each element in each fiber and each longitudinal integration points.

The procedure repeats successive time steps until the specified final time step is reached or the failure of the structure occurs (whichever occurs first).

CASE STUDY

To demonstrate the applicability of the computer program SAFIR, a case study was carried out on a steel framed structure exposed to fire. The analysis was carried out for various fire protection configurations of a steel frame.

Problem Definition

The structure considered is a four-storey, three-bay steel frame with the fire compartment located at the central bay of the ground floor as shown in Figure 1 (a). The dimensions and loading for each member are also shown in the figure. The sectional properties of the frame members (columns and beams) are taken from the Handbook of Steel Construction[12] and are given in Table 1. Four cases were considered in the analysis with each case characterized with respect to the three members in the fire compartment - Beam B21, Columns C12 and C13 - as follows:

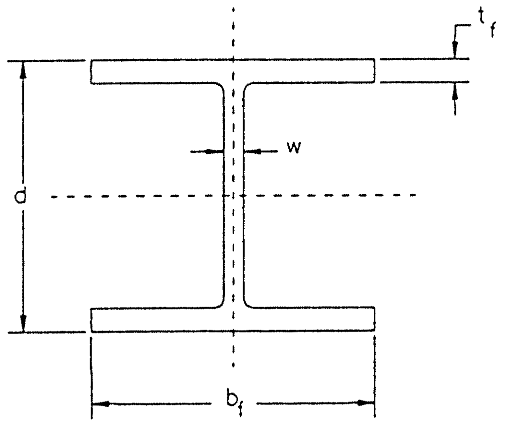
- Case 1 - Beam B21, Columns C12 and C13 are unprotected.
- Case 2 - Beam B21 protected and Columns C12 and C13 unprotected.
- Case 3 - Columns C12 and C13 protected and Beam B21 unprotected.
- Case 4 - Beam B21, Columns C12 and C13 protected.

All other members in the frame remain cold (room temperature) throughout the analysis of each case study.

Table 1. Cross-sectional properties and material properties used in the analysis.
(See cross section below)

Designation		Total Area	Grade	Depth	Flange Width b_f	Flange Thickness t_f	Web Thickness W
		mm²		mm	mm	mm	mm
B11, B21, B31, B12, B22, B32, B13, B23, B33	W410x60	7580	300W	407	178	12.8	7.7
B14, B24, B34	W360x45	5730	300W	352	171	9.8	6.9
C11, C12, C13 C14, C22, C23, C32, C33, C42, C43	W310x118	15000	300W	314	307	18.7	11.9
C21, C24, C31, C34, C41, C44	W310x97	12300	300W	308	305	15.4	9.9

Properties of steel at room temperature: modulus of elasticity = 200 000 MPa; Yield stress = 300 MPa; Poisson's ratio = 0.3



The insulation thickness applied to the members is determined in accordance with the code[1] requirements. The insulation was asbestos cement and was of 20 mm thickness on both steel beams and columns, determined based on the ratio M/D, in which M is the weight of the beam or column per metre length and D is the developed heated perimeter in metres. The columns were profile (contour) protected on all four sides, while the beams were similarly protected, but only on three sides, to simulate the effect of the floor slab on the top flange of the beam.

In addition to the four-case studies carried out, an analysis was also conducted to determine the response of Beam B21 when acting as an isolated member, Beam S21 (see Figure 1 (b)). This was done to compare the overall structural behaviour to that of a single member behaviour under fire exposure. The single beam was simply supported and the analysis performed under two fire protection conditions; beam protected and beams unprotected. For the protected case, contour protection was assumed. In the analysis, all the properties were kept the same as in Beam B21 (acting as an integral part of the frame).

Idealization for Analysis

Two types of idealization (one for thermal analysis and the other for structural analysis) were used. For the thermal analysis, the cross-sections of the members (beams and columns) were discretized with triangular and rectangular elements as shown in Figure 2 (a). These sections are exposed to heating, simulated in accordance with ISO 834[17] standard temperature-time relation which yields fire temperatures very similar to that from ASTM E119 curve. The expression that describes the ISO curve is:

$$T = T_0 + 345\log_{10}(8t + 1) \qquad (1)$$

where t is the time in minutes, T is the fire temperature in °C and T_0 is the initial temperature in °C. An initial temperature of 20°C is assumed in the analysis.

For the structural analysis, the members of the frame were discretized with beam elements along their lengths, as shown in Figure 2(b). The beams were subjected to a uniformly distributed load computed on the basis of the plastic analysis theory.

The material properties used in the analysis are given in Table 1. The relationships used for thermal and mechanical properties at elevated temperatures are taken from the Eurocode and these relationships are given in Reference [16].

Rigid connections and non-sway conditions were assumed in the analysis. In order to simulate the action of braces in preventing side-sway movement, the midpoints in all internal beams were considered restrained in the horizontal direction. The fire is assumed to be restricted to one compartment (the central ground floor bay). It is assumed that the top surface of Beam B21 is protected by a floor slab and the other three surfaces of the beam are exposed to fire. Further, it is assumed that appropriate compartmentation exists in the zone surrounding the fire so that fire affects only the members (Beam B21, Columns C12 and C13) located in the compartment. The rest of the frame remains at room temperature.

Analysis

The analysis was carried out with two minutes time increments, with a final time step of 120 minutes. The analysis is terminated when failure occurs in any of the members due to instability or the iteration fails to reach convergence at any time step, which indicates that the equilibrium is not satisfied.

RESULTS AND DISCUSSION

Figures 3 and 4 show temperature distribution as a function of time at various locations in the beam and column cross section, respectively. The temperature from fire exposure (ISO 834) is also shown in these figures. The predicted temperatures follow the expected trend with the unprotected sections, at any given instant, attaining higher temperatures than the protected section. Also, the temperature rise in the insulated column (beam) is slower in the initial stages as compared to the unprotected column (beam). Further, as expected, the temperatures in the top flange are lower than the web in both column and beam cross-sections, in both protected and unprotected cases.

At the later stages of fire exposure, temperatures in both unprotected columns and beams reach the fire exposure temperatures. At approximately 40 min, the temperature values of the protected and unprotected section at the web location are 742 and 878°C, respectively. The corresponding fire exposure temperatures is 885°C, about 1% greater than that for unprotected section, 16% more than that for the protected section in the web. Overall, these predictions indicate that the computer program SAFIR is capable of predicting temperature distribution in a cross-section composed of steel and different protection configurations.

Figure 5 shows the variation of mid-span deflection over time for Beam B21 (as a member of the frame) under four cases corresponding to different fire protection scenarios. For Case 1, corresponding to the unprotected case, the deflection at the time of failure is about 670 mm and occurs in approximately 20 min. In Case 4, corresponding to the fully protected beams and columns, the failure occurred in approximately 85 min with the resulting deflection reaching to approximately 700 mm. In Case 4, the deflection rate is slower in the initial stages (up to 40 min); thereafter, the rate of increase in deflection is higher. This is as expected since the presence of insulation acts as a heat insulator until gradual deterioration in its properties occurs. The failure in both Case 1 and Case 4 is from the buckling in the columns, which occurs before the failure in the beams and, hence, the overall failure of the frame is governed by the columns.

For Case 2, when only the beam is protected, the deflections rise slowly. Figure 5 indicates that the failure does not occur in the beam until approximately 28 min. However, the failure in this case also results from the buckling of the column at a much earlier time and this is explained with reference to Figure 6. For Case 3, when the columns are protected and the beam is unprotected, the failure occurs in the beam but the failure and deflection trend is very much similar to Case 1. The resulting deflection is about 630 mm, as compared to 670 mm for Case 1.

The lower deflections and slightly higher fire resistance in Case 3 as compared to Case 1, is due to the structural interaction and load transfer mechanism that occur in the frame. Results from full scale tests at Cardington[20,21] have shown that unprotected beams can have a higher fire resistance and withstand large deflections due to slab-beam interaction and resulting tensile membrane action. However, in the present analysis no slab-beam interaction was accounted for in order to keep the analysis simple and hence only a marginal increase in fire resistance is noticed.

An examination of the results from SAFIR by comparing the deflection in Beam B21 to the rest of the beams in the frame show that the failure of Beam B21 to sustain further load after 20 min (Case 1) does not constitute a failure of the entire frame. The high deflection in Beam B21 (unprotected) results from the deterioration of steel properties, such as stiffness and strength, as a result of exposure to fire. Since the temperature in other Beams B11, B12, B22 (as well as columns) does not rise, their deflections tend to be negligible and no failure occurs in any other bay. Further, interaction exists between members in the frame, after the failure of Beam B21, leading to alternate load paths developing and transferring of the loads to the surrounding cooler and stronger members.

Figure 5 also shows the comparison of deflections for the analysis; Beam B21 acting as an integral part of the frame and Beam S21 acting as a single member (simply supported beam) as shown in Figure 1b with without protection. As expected, when Beam S21 is protected, the fire resistance increases from about 10 min to 38 min. Further, while Beam S21, without protection, acts as a single member which failed in 10 min, Beam B21, when acting as part of the frame, failed in 20 min. It can been seen from the figure that Beam S21 (acting as a single member), had a deflection of 700 mm (at the failure time in 11 min), while the corresponding deflection of Beam B21 (as a part of the frame) was 67 mm (at 11 min). It can also be seen from this figure that the deflection rate in Beam S21 is slower in the initial stages and, thereafter, the deflection rate increased until failure. However, the deflection rate in Beam B21 is slower until about 10 min and thereafter increased rapidly to failure. This difference in behaviour can be explained as follows:

Beam S21 acting as a single member (simply supported beam) required only one plastic hinge to become a mechanism. Before the mechanism was formed, the deflection rate was slower, however, once the mechanism is formed, the support no longer offered resistance to rotation and the deflection increased rapidly. For Beam B21, acting as a member in the frame, three plastic hinges (two that form simultaneously at the supports and one at the mid-span) are required for the beam to become a mechanism. Due to this, the deflection rate was slower for a longer time in Beam B21 than for Beam S21. Therefore, there is an improvement in the fire resistance performance when the beam is an integral part of the frame.

Figure 6 shows the variation of the vertical displacement at the top of Column C12, for the four cases with different fire protection conditions. First, the column displacements increase due to thermal elongation, but as soon as the column buckles, the displacement quickly decreases. The results obtained from SAFIR follow a similar trend as observed in the fire tests. In Case 1 and Case 2, Column C12 buckled after 17 min of fire exposure, thus the failure is governed by this consideration. However, in Case 3 where Columns C12 and C13 were protected, and the beam is unprotected, buckling was not observed at the end of the analysis and the failure is governed by failure in the beams, which occurred in 20 min. Hence, compared to Case 1 the fire resistance from Case 3 is marginally higher. This analysis also demonstrates that the protection of the critical members, such as columns, is necessary in case of fire. For Case 4, with all members protected, the buckling in Column C12 or C13 occurs in 70 min and this governs the failure.

The above comparisons show that the computer program SAFIR is capable of predicting the thermal and structural response of steel framed structures exposed to fire. Since the program is based on the finite element formulation, the overall structural response can be assessed. However, to fully assess the inherent fire resistance, all the effects of slab-beam interaction should be accounted for. Further, results from the case study illustrate that the fire restricted to a compartment significantly affects the exposed members located in the compartment, but has little or no effect on the members that are outside the fire compartment.

CONCLUSIONS

Based on the information given in this paper, the following conclusions can drawn:

- Both thermal and structural analysis can be carried out using the computer program SAFIR.
- The computer program SAFIR is based on the finite element formulation and, hence, can be used to study the overall behaviour of structures under fire conditions.

- There is an improvement in the fire resistance performance of a beam when acting as a part of a structure than when acting alone as a single member.
- The effect of slab-beam interaction has to be fully accounted for in the analysis to assess the realistic fire resistance of a steel frame.

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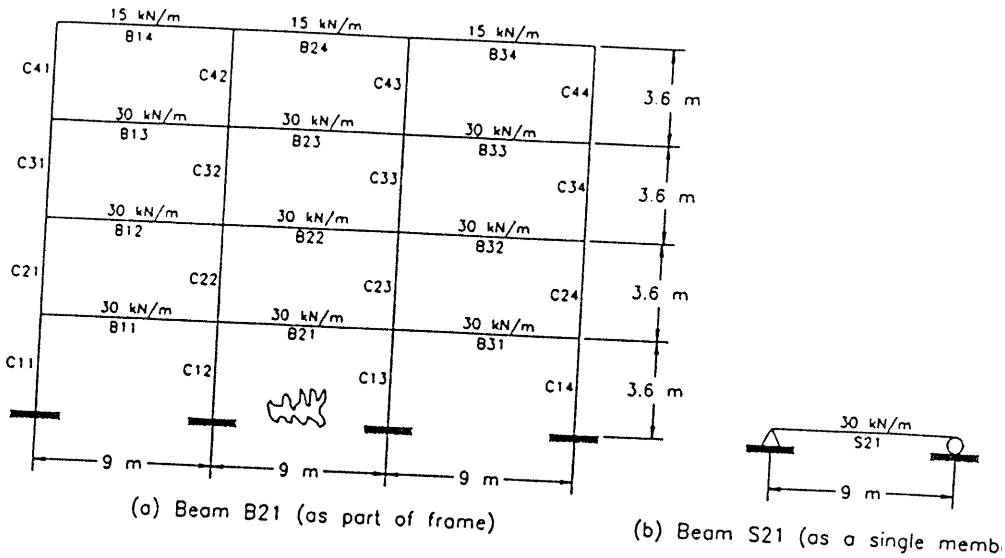

Fig. 1. Frame and single beam dimension and loading (sections in Table 1).

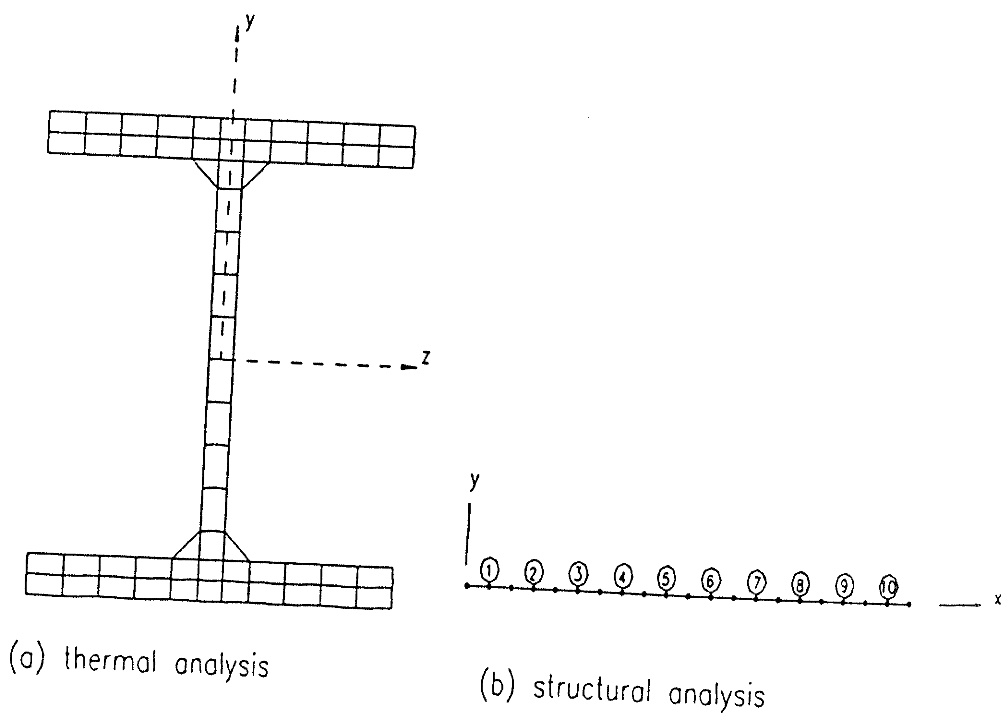

Fig. 2. Typical finite element idealization for analysis using SAFIR.

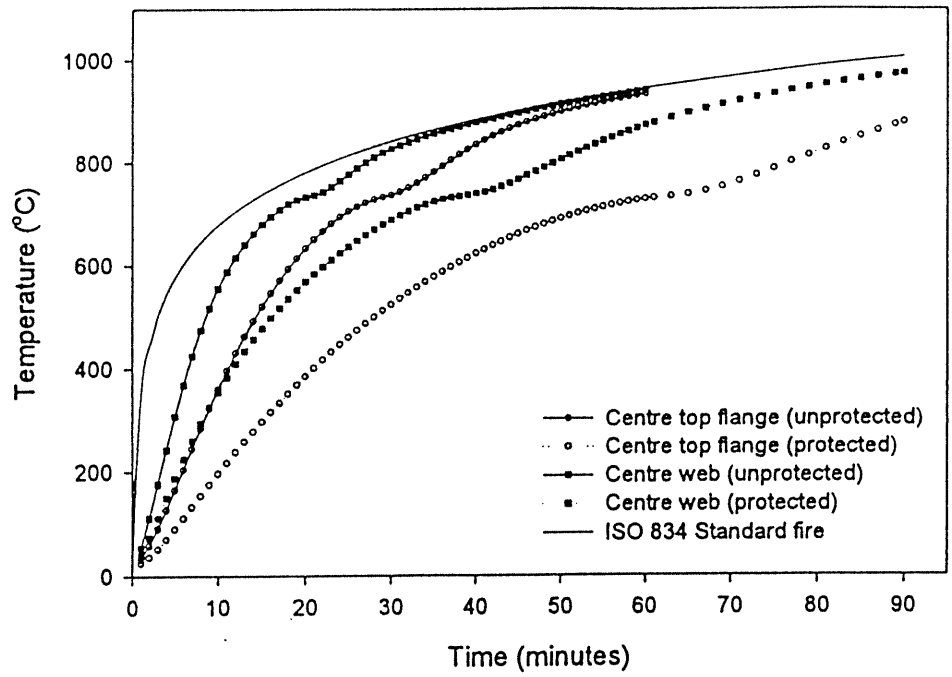

Figure 3: Variation of temperature as a function of time for beam

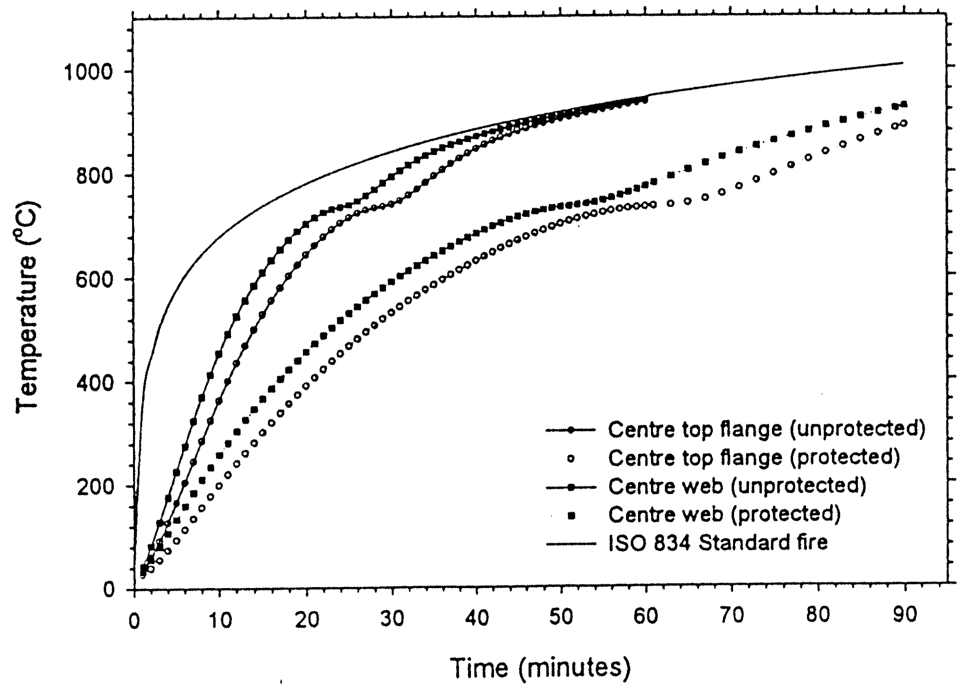

Figure 4: Variation of temperature as a function of time for column

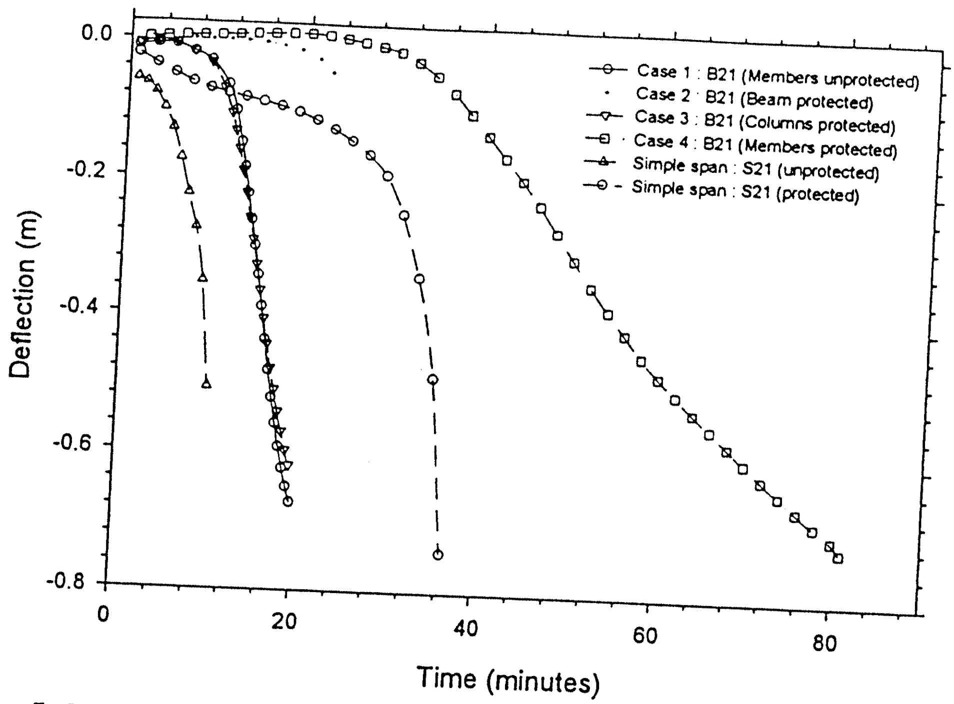

Figure 5: Midspan deflection of Beam B21 and Beam S21 as a function of time

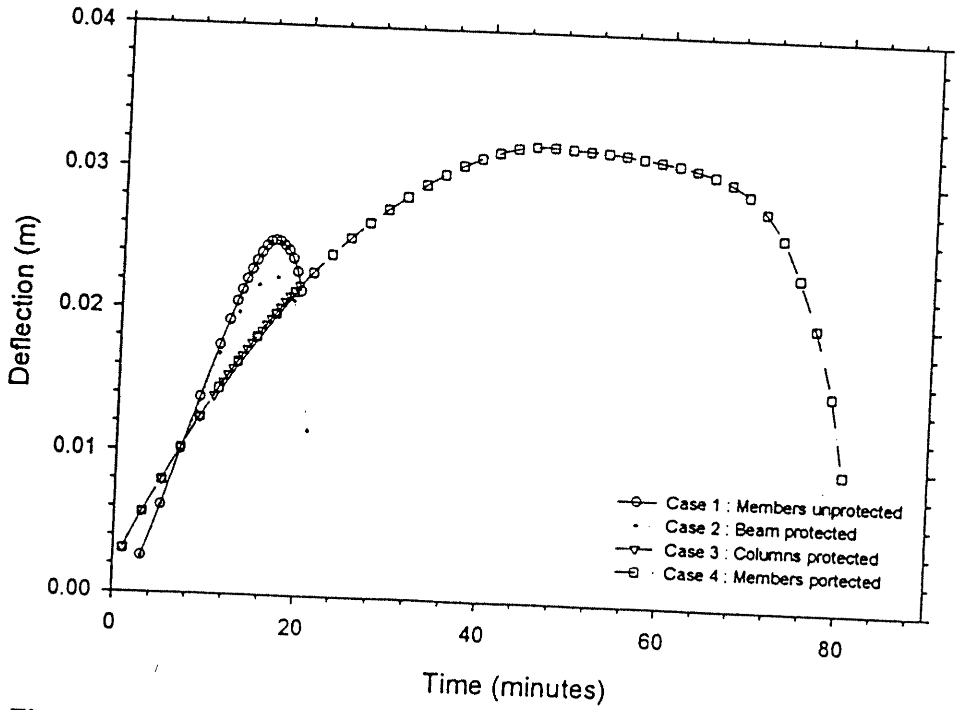

Figure 6: Axial deflection at top of Column C12 as a function of time