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Numerical Simulation of a Full Scale Fire Test on a Loaded Steel Framework

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ABSTRACT

This paper presents the results of a number of numerical simulations of the behaviour in a real fire of a full-size, loaded, two-dimensional, mainly unprotected steel frame. Data from the fire test, reported in Steel Construction Today 1987, provides the benchmark. The application of one, two and three-dimensional heat flow models is discussed, and the basis of the structural model used is described. The influence of lateral restraint, frame continuity and thermal expansion is quantified using the computer model. In contrast the simple method in draft Eurocode 3 is used to calculate the frame stability assuming that the temperatures of the beam and columns are uniform across their sections, and good agreement with the test result is shown. It is suggested that a rigorous computer program, like that described in the paper, could be usefully employed to identify those types of structure which might be analysed safely by the simplified method.

1 INTRODUCTION

It has been known for many years that, in common with all other materials, a steel structure may suffer loss of load bearing capacity and even collapse when submitted to the action of a severe fire.

The problem of steel in fire is not so much the loss of strength and stiffness with increasing temperature (comparable with the behaviour of other building materials) as the fact that the temperature in steel tends to increase rapidly due to the action of the fire and the high thermal conductivity of steel. One way to solve, or rather to avoid, the problem is to protect steel structures with insulating materials, thus delaying the temperature increase in the steel.

However, it is also known that a structure may reach higher temperatures while remaining stable provided that the load factor, i.e. the ratio between the actual load and the ultimate load at ambient temperature, is reduced.[1] Continuity (hyperstaticity) can therefore be a good solution in order to reduce the amount of insulating material used. At ambient temperatures, but even more at elevated temperatures, a continuous structure behaves better than the separate members and it is thus desirable to design as a complete structure.

Historically, the first fire safety design method relied entirely on full scale tests. Tests are expensive, time consuming and difficult to perform, especially on complete structures. Yet they may still be necessary to investigate new building systems or to validate theoretical models. British Steel plc (BS) and the Department of Environment (DoE) sponsored such a fire test which was performed in the Fire Research Station's Cardington Laboratory on a full size, fully loaded, two-dimensional steel frame[2] to generate data for use in the complementary development of analytical techniques to simulate the structural stability of steelwork in natural fire.

As elastic analysis leads to conservative results, theoretical design methods for steel structures exposed to fire mainly rely on the theory of plasticity, and this is widely used and accepted for simple structures such as continuous beams.

For other structures, where large displacements and the effect of restraint affect stability, the complexity of the problem makes it amenable to solution by numerical models which, based on acknowledged principles of the theory of structural mechanics, are able to consider, amongst other things, the visco-elasto-plastic behaviour of steel, the effects of thermal gradients, large displacements, restraint forces, and residual stresses. The first author has been active in the development of such a computer code at the University of Liège[3,4] and the other two authors are deeply involved in UK fire modelling work involving FRS, BS, Sheffield University,[5] City University,[6] and others. Much work on the subject has been undertaken elsewhere.[7-14]

Recent recommendations have been presented in Luxembourg within the Eurocode context[15,16] for the thermal and mechanical properties of steel at elevated temperatures for use in numerical models. The recommendations are under discussion and may be modified but they indicate what could be utilised in Europe for the foreseeable future.

The aim of this paper is to show how the recommendations can be applied in a numerical model of frame behaviour and how the results provided by the numerical model compare with the fire test results. Results are reported for sensitivity analyses which explored the effect of varying some important parameters, for example, yield stress of steel.

2 THE CARDINGTON FIRE TEST ON A LOADED STEEL FRAMEWORK

A natural fire test on a fully loaded, two dimensional steel framework was carried out by British Steel in collaboration with the Fire Research Station of the Department of the Environment and is described in detail elsewhere.[2] The experiment was carried out in a purpose-built compartment with a floor area of $50\,m^2$ and a ceiling height of 3·9 m, a size typical of office accommodation. The front elevation of the compartment is shown in Fig. 1. Ventilation was controlled by means of shutters placed within the long walls to obtain as symmetrical a heating exposure as possible. The fuel load comprised timber cribs which, together with the selection of ventilation openings, achieved an Equivalent Fire Duration of 32·5 min in the test. This was considered sufficient to ensure that the loaded beam reached its limiting temperature during the fire.

2.1 Test frame

The steel framework selected for testing under load was typical of that used in a building of two or three storeys in height. It comprised a

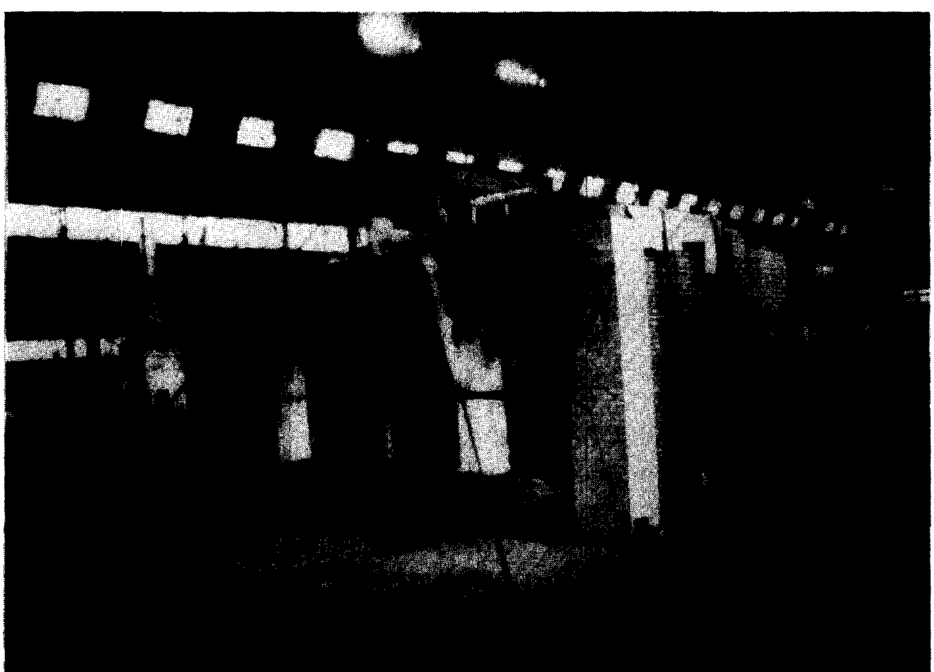


Fig. 1. Compartment with fire test in progress.

4550 mm length of 406 × 178 mm × 54 kg/m BS 4360:1979 Grade 43A universal beam section bolted to two 3530 mm lengths of 203 × 203 mm × 52 kg/m Grade 43A universal column section.

The test beam which spanned the compartment at ceiling height remained unprotected, but four 1200 × 5550 × 150 mm precast concrete slabs, which formed part of the compartment roof, were attached to the top flange by welded 12 mm diameter threaded bars. The slabs were separated by a gap of 25 mm to prevent composite action with the beam, and the gap was filled with ceramic fibre blanket. Each column, which extend above the beam, was pin jointed at the base. The webs were protected by autoclaved aerated concrete blocks with a density of 677 kg/m^3 (3·8% water content by weight) built between the flanges using an ordinary mortar mix. This system had been shown to be a relatively cheap method of raising the fire resistance of lighter freestanding columns to 30 min in the ISO 834 fire test. The beam/column connections utilised M20 Grade 8.8 bolts to provide improved resistance to loss in strength at high temperatures.

2.2 The loads

The complete assembly is shown schematically in Fig. 2. The test frame was centrally positioned inside the compartment, parallel to the short walls. It was surrounded by load reaction frames which gave a closed loading system so that only dead loads were transmitted to the floor. A

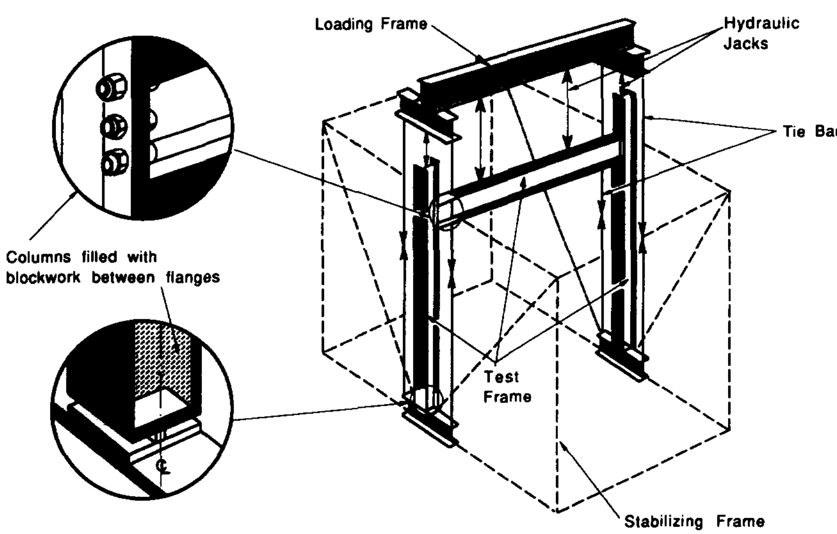


Fig. 2. Schematic layout of the loaded frame used in the Cardington tests.

subsidiary framework was designed to prevent lateral and sway instability in the test frame. A maximum axial compressive load of 552 kN was applied to each test column by an hydraulic ram and load cell placed between the top bearing plate and the load reaction frame. The test beam was loaded to 39·6 kN at each of four equal positions along the span. The loads were maintained constant throughout the fire test.

With the exception of the test frame, the remainder of the structure was fire protected.

2.3 Steel temperatures

During the fire, thermocouples fixed into the steel framework measured the changes in temperature in the flanges and the webs. The heating rate was fastest at the centre of the unprotected beam. Maximum temperatures of 775, 777 and 577°C were measured in this locality in the lower flange, centre of the web and upper flange, respectively, after 20 min. The corresponding temperatures in the lower flange and web close to the connections were 671 and 720°C, the web heating up more rapidly since it was thinner than the flange.

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

The load was removed from the structure after 22 min into the natural fire test. At this time the temperatures reached by the thread beneath the bolt heads were 397°C for the upper and 441°C for the lower bolt. The reduction in temperature along the thread, which extended into the blockwork, was approximately 100°C.

2.4 Deflection behaviour

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

At failure the beam exhibited considerable twisting as well as vertical deformation, together with tilting of the concrete slabs attached to the upper flange. Subsequent examination revealed the presence of a plastic hinge approximately 600 mm from each end of the beam and some plastic distortion of the welded end plates at the top of the connection.

The blocked-in columns expanded axially to reach a maximum extension of 20 mm after 15 min. The distance between the columns increased during the test due to the axial expansion of the beam and rotation of the ends of the beam due to thermal bowing. The average lateral displacements measured on the columns at different heights with time are shown in Fig. 3.

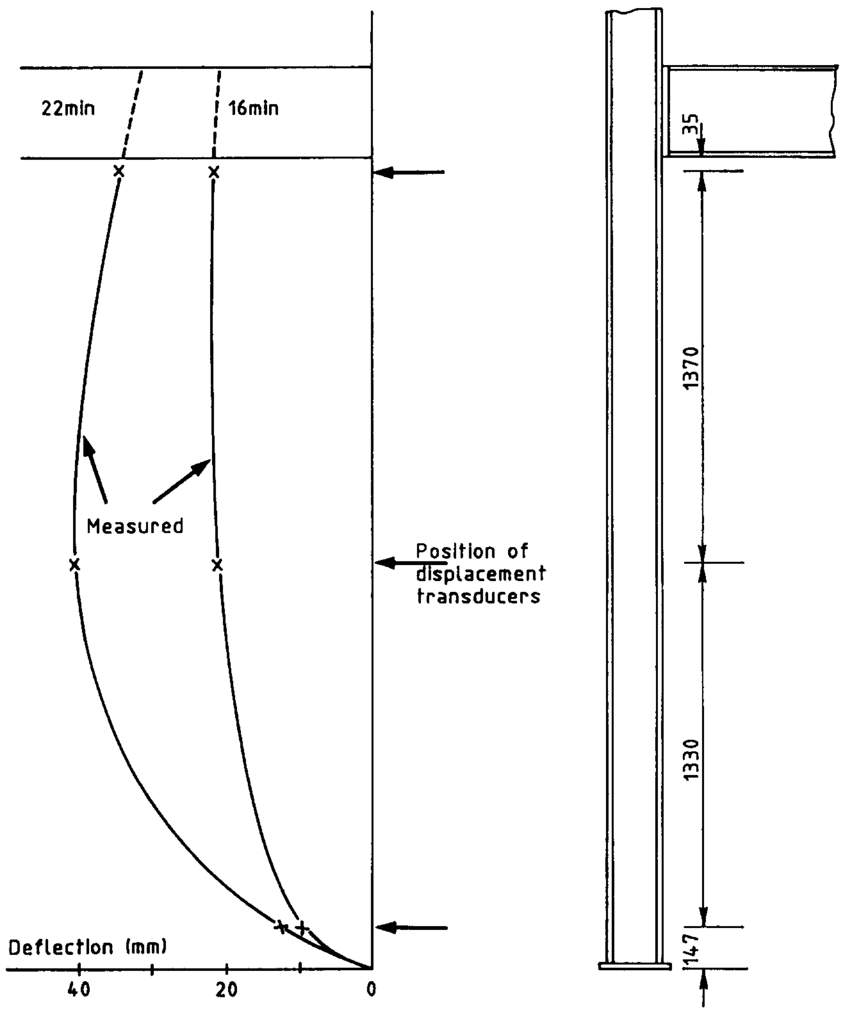


Fig. 3. Average lateral deflection of column in the fire test.

3 THE NUMERICAL MODEL

Before a calculation of load bearing capacity can be made it is necessary to know the temperature distribution in the structure. The temperature data may come from a fire test or be predicted from a knowledge of the fire environment. Except at the moment of complete collapse, the rates of deformation are low enough to neglect the heat that is caused by plastic straining. Hence the static state of the structure does not influence the temperatures of the structure. This generally accepted assumption allows the calculation of temperatures in the structure to be separated from the calculation of deformations, strains and stresses (structural calculation).

It should be noted that it is not at present possible to calculate the effect of excessive deformation of the supporting structure on the adhesion/cohesion of applied insulation material or the effect of cracks developing in concrete. Both effects could influence the temperatures attained by the structure.

Whatever method is used to calculate the temperatures within the structure, it is usual to regard the temperature of the environment (combustion gas) as the main parameter affecting the heat exchange between the environment (the fire or the furnace) and the structural member. Indeed, specifications for fire tests very often deal only with the evolution of combustion gas temperature in the furnace, without reference to the radiation from the furnace linings or burner flames which can markedly affect the heat transfer to the specimen as a test proceeds.

3.1 One-dimensional temperature distribution

An often-used approximation is the assumption of a uniform temperature distribution within the steel section, justified by the high thermal conductivity of steel. The section is then characterised at any point in time by one temperature (from which comes the concept of a critical or limiting temperature) and the equation describing this temperature increase is of the one-dimensional type. The section is characterised by its section factor (i.e. heat-exposed perimeter divided by cross-section area) and the heat flow equation can be adapted to consider the effect of an insulation protection.[17,18]

This method does not consider the temperature gradient that may arise over the depth of the cross-section. This can considerably influence the bowing of beams or the buckling of columns. Furthermore, if a steel beam is in direct contact with a concrete slab (as in the Cardington test) the heat transferred from the steel section to the concrete slab cannot be considered so that the uniform temperature calculated by the one-dimensional approach differs from the average value of the two-dimensional temperature field.

This approximate method is therefore unsatisfactory if the deflection response is to be reliably modelled, but is commonly used together with the simple plastic design methods mentioned in the Introduction.

3.2 Two-dimensional temperature distribution

For slender structural members such as beams, columns or bars, a less restrictive assumption is that the temperature distribution does not vary along the length of the member. The main equations of the problem are two-dimensional with respect to the Cartesian coordinates y and z that are perpendicular to the longitudinal axis x.

Except for very simple cases, these equations must generally be solved numerically using finite element or finite difference techniques. In Liège, the first author has been using a two-dimensional finite difference program (thermal part of CEFICOSS see Refs 3 and 4), specifically written for the calculation of temperatures in composite steel-concrete building members exposed to fire. The cross section is discretized by a rectangular mesh, Fig. 4. The temperature and the type of material (steel, concrete or insulating material) are assumed to be uniform within each rectangle. The equation of heat transfer is transformed into a finite difference equation which can be written for every rectangle of the cross-section. The heat flow from the environment to the section is assumed to be convective and radiative.

The equations are integrated with respect to time by a totally explicit scheme which provides an equation in which all thermal properties are evaluated at the beginning of every time step. As the equation is written for each rectangle, there is no need to form and solve a large system of equations: for one time step it is only necessary to solve as many equations with one unknown as there are rectangles in the cross-section. The time step has to be chosen to ensure stability and convergence of the solution. The maximum allowable time step can be automatically computed[3] and, because of the temperature-dependent thermal properties of building materials, it usually increases as the member heats up. The limited size of most two-dimensional problems means that they can be easily solved with commonly-available desktop computers.

An advantage of the rectangular discretization is that the temperatures can be presented in a rectangular array which directly gives a good idea of the temperature distribution in the cross section, Table 1.

The main disadvantages of this code arise from its inability to deal with curved surfaces (as for circular columns or for the root radius of a hot

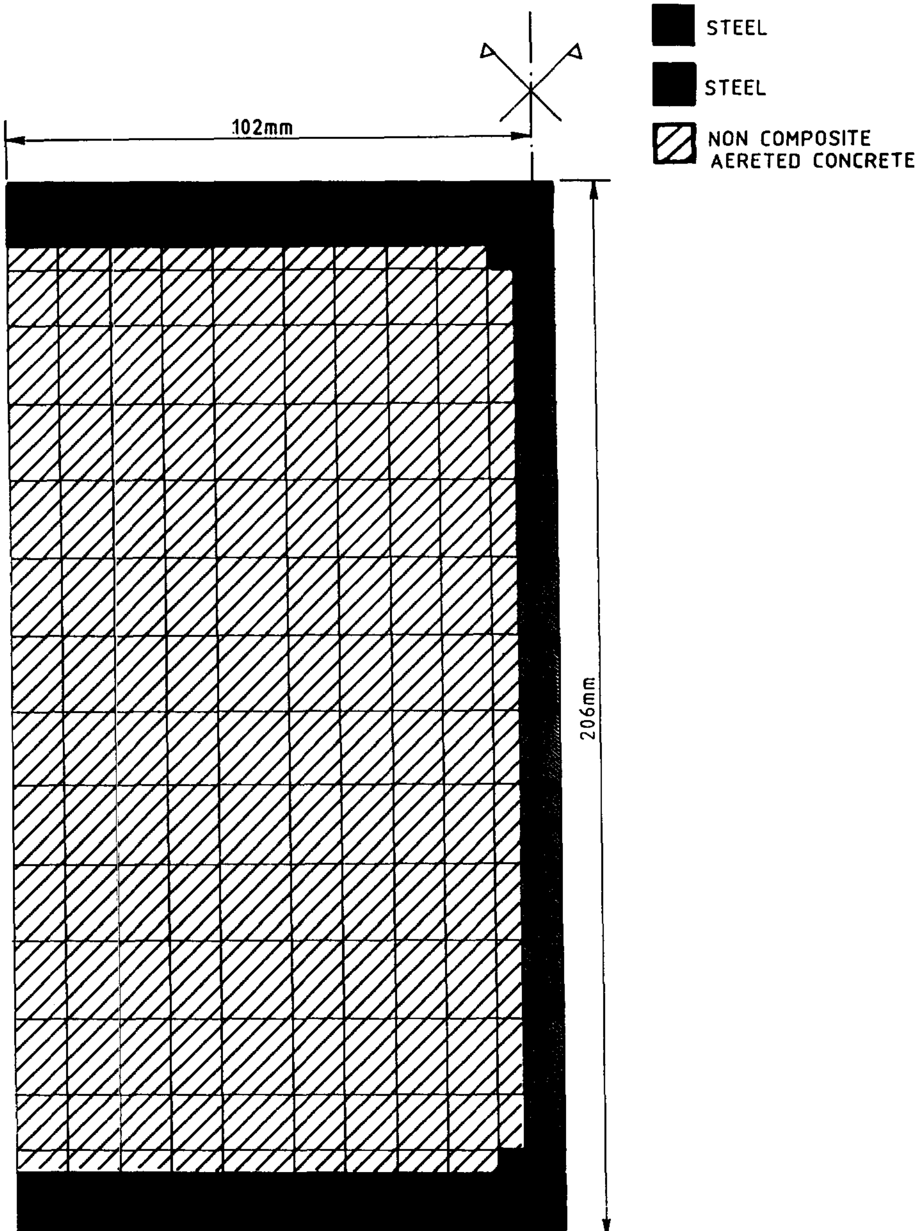


Fig. 4. Discretization of the column cross-section by $11 \times 16 = 176$ rectangles.

rolled steel profile) and with radiation between surfaces of internal voids (in hollow core concrete slabs or a steel profile insulated by a box casing). That is why a new thermal code based on the finite element technique has been written in Liège (THERMIN, see Ref. 19). However, as some

TABLE 1
Typical Presentation of Calculated Temperatures

Section type: 1 after 600 Seconds of Fire

The time step of the thermal calculation is 0·86 sec.

737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0
737·0	269·0	261·0	253·0	246·0	237·0	229·0	222·0	214·0	206·0	200·0	197·0
737·0	337·0	247·0	214·0	206·0	199·0	191·0	185·0	180·0	177·0	189·0	182·0
737·0	436·0	222·0	100·0	100·0	100·0	95·0	93·0	97·0	100·0	143·0	162·0
737·0	469·0	217·0	100·0	62·0	40·0	36·0	36·0	42·0	68·0	100·0	133·0
737·0	475·0	215·0	96·0	46·0	27·0	23·0	23·0	29·0	48·0	91·0	110·0
737·0	475·0	215·0	94·0	44·0	25·0	21·0	21·0	25·0	41·0	76·0	94·0
737·0	475·0	215·0	94·0	44·0	25·0	21·0	21·0	24·0	37·0	69·0	85·0
737·0	475·0	215·0	94·0	44·0	25·0	21·0	21·0	24·0	36·0	65·0	81·0
737·0	475·0	215·0	94·0	44·0	25·0	21·0	21·0.	24·0	36·0	67·0	83·0
737·0	475·0	215·0	94·0	44·0	25·0	21·0	21·0	25·0	39·0	73·0	91·0
737·0	475·0	215·0	94·0	44·0	25·0	21·0	22·0	26·0	44·0	86·0	105·0
737·0	475·0	216·0	97·0	47·0	27·0	24·0	24·0	31·0	55·0	100·0	126·0
737·0	472·0	221·0	100·0	68·0	47·0	42·0	42·0	50·0	85·0	106·0	156·0
737·0	451·0	241·0	100·0	100·0	100·0	100·0	100·0	100·0	100·0	170·0	194·0
737·0	380·0	295·0	259·0	250·0	242·0	233·0	226·0	219·0	214·0	230·0	221·0
737·0	330·0	320·0	310·0	301·0	291·0	281·0	272·0	263·0	253·0	244·0	240·0
737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0	737·0

modifications have still to be introduced to make the results of THERMIN utilizable by the structural part of CEFICOSS and because the thermal part of CEFICOSS has been used for the simulations presented in this paper, this new program THERMIN will not be described here.

3.3 Three-dimensional temperature distribution

Structural members may have temperature variations along their length as well as across their section due to differences in heat input and heat loss (axial heat sink effects). Three-dimensional temperature distribution can occur in, for example:

- continuous beams (or columns) with some spans (or storey heights) exposed to fire and others not;
- continuous beams with every span exposed to fire but supported on masonry walls which provide shielding and act as local heat sinks;
- connections between members of different type (different size and shape).

There are several programs specifically dedicated to the calculation of three-dimensional temperature fields in building members exposed to fire. One is FIRES-3D[20] mainly used for reinforced concrete. Important endeavours have also been made in Germany concerning composite steel-concrete structures.[12]

To model three-dimensional temperature distributions it is usually sufficient to extend the two-dimensional discretization of the cross-section along the third dimension prismatically so that the discretization of the cross-section is the same at every location on the longitudinal axis. Such three-dimensional temperature distributions can be directly used in the structural computer code avoiding tedious manual input of temperatures if beam (i.e. prismatic) finite elements are used.

There are however some problems for which beam finite elements are unsuitable, and brick finite elements should then be used. An example is a connection detail for a beam-to-column joint. The temperatures resulting from such a calculation can be used in the structural computer code if the latter is also based on the brick model. The amount of data to introduce, the time taken to sort the results and analyse them, as well as the time taken in computing, make such three-dimensional brick models suitable mainly for the analysis of local details, and not the analysis of complete structures.

In the structural part of the program CEFICOSS from Liège, the effect of the third dimension in the temperature distribution can be introduced to some extent by an approximate method. If $T_2(y, z, t)$ is the two-dimensional temperature distribution that has been calculated, it is possible to consider in the structural calculation that the temperature increase at a particular point, $\Delta T_3(x, y, z, t)$, is only a fraction of ΔT_2, such that:

$$\Delta T_3(x, y, z, t) = f(x)\Delta T_2(y, z, t)$$

or

$$T_3(x, y, z, t) - T_0 = f(x)\,(T_2(y, z, t) - T_0)$$

where:

t = time
T_0 = initial temperature
f = reduction function obtained from experimental data or a more specific thermal analysis.

This approximate method makes it possible to get some qualitative information about the influence of the third dimension. It has also been used here when considering the temperature increase in the beam of the Cardington test.

3.4 Basis of structural model

Though finer discretization could be necessary to investigate local problems such as local buckling phenomena, the beam element discretization seems to be suitable for the analysis of complete steel frames.

The structural part of the CEFICOSS program developed in Liège[3] is based on a plane beam finite element. The element has two nodes with three degrees of freedom. It is accepted that it would probably be better to use an element with three nodes and seven degrees of freedom if highly unsymmetrical (with respect to the depth of the section) plastic zones are expected.[21] Shear energy is not considered and the expression of Jennings[22] is used for the axial strain.

The cross-section is discretized using the rectangular mesh used for the thermal calculation so that the calculated temperatures can be directly used by the structural part of the program. All variables (type of material, temperature, strain, stress, tangent modulus, plastic strain...) change from one rectangle to another. The same advantage of a clear presentation of the results derives from the rectangular discretization, Table 2.

The effect of large displacements are taken into account by an updated Lagrangian description. The developments are classical if not for the fact of the numerical integration on the rectangles of the cross-section to compute the tangent stiffness matrix and the internal forces (fibre model). The longitudinal integration is by the Gauss method, using two points of integration.

It is possible to use the non-linear stress–strain relationships and thermal strains of materials recommended in Parts 10 of Eurocode 3[15] or Eurocode 4[16] which means that creep strain is implicitly introduced in the stress-related strain.

The program proceeds step by step. The load is first applied in several increments at ambient temperature. Stresses, strains and displacements are then calculated at a number of time steps during the fire up to failure.

4 MODELLING THE CARDINGTON FIRE TEST

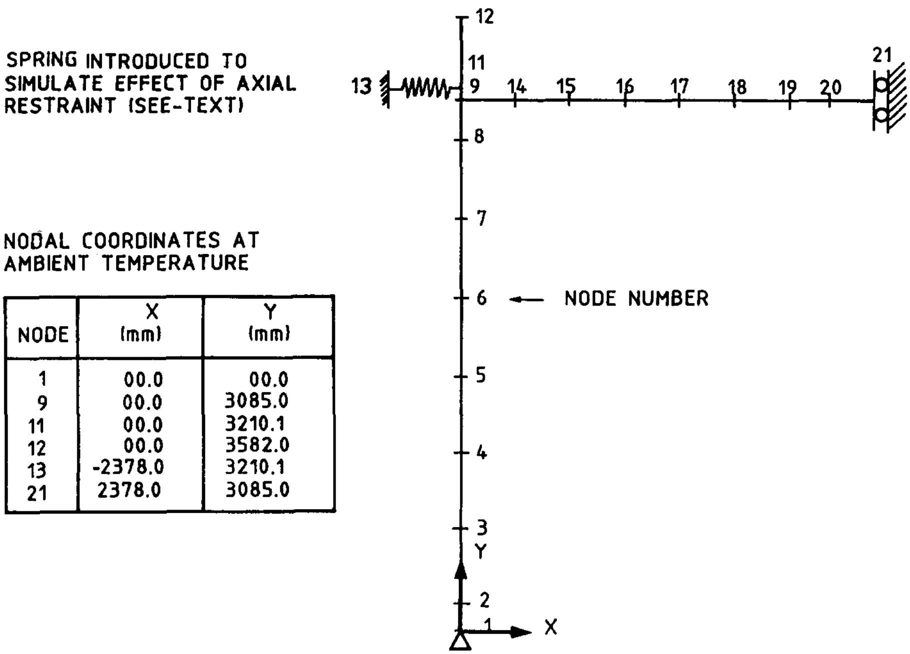
In the first simulation, no initial imperfection of the frame geometry was introduced and, for reasons of symmetry, the longitudinal discretization and node numbering is as shown in Fig. 5. The restraint offered by the secondary steelwork may be regarded as a spring at node 11. The

TABLE 2
Typical Presentation of Calculated Stresses

After 600·0 Seconds of fire

Stresses in element: 8, point of integration: 2

3·10	5·12	7·15	9·10	11·34	13·58	15·54	17·61	19·83	21·62	22·46
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	20·35	22·19
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	24·12
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	25·51
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	24·13
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	20·34
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	14·56
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	7·22
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−1·62
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−12·01
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−23·98
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−37·18
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−37·59
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−36·41
0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	−35·54	−35·77
−33·97	−34·09	−34·21	−34·34	−34·50	−34·67	−34·83	−35·01	−35·23	−35·41	−35·50



NODE	X (mm)	Y (mm)
1	00.0	00.0
9	00.0	3085.0
11	00.0	3210.1
12	00.0	3582.0
13	-2378.0	3210.1
21	2378.0	3085.0

Fig. 5. Longitudinal discretization of frame.

calculated values of the axial stiffness (67 kN/cm) and the axial plastic load (86 kN) represents the bending stiffness and resistance of the supporting framework which remained at room temperature due to the presence of thermal insulation. The influence of lateral restraint and the assumption of symmetry are discussed later.

Although residual stresses are known to have a significant influence on the fire behaviour of concentrically loaded steel columns,[23] they are ignored in the analysis because of the over-riding effects of thermal bowing and the presence of bending moments in the columns. The columns are pin-jointed at the base while the beam is rigidly fixed to the columns. Full rigidity was assumed in the fire condition because the connection detail was at a lower temperature than the rest of the structure due to a lower section factor and the loss of heat into the concrete filled column through this connection. This was justified from an examination of the beam-to-column connection after the fire.

The cross-section of the column is discretized as shown earlier in Fig. 4. All the thermal and mechanical properties of steel are assumed to vary according to Part 10 of Eurocode 4,[16] from which a relative emissivity of 0·5 is recommended; however, for the column flange facing the wall of the fire compartment it was considered appropriate to reduce the value arbitrarily to 0·3 to account for some degree of radiative shadowing. A

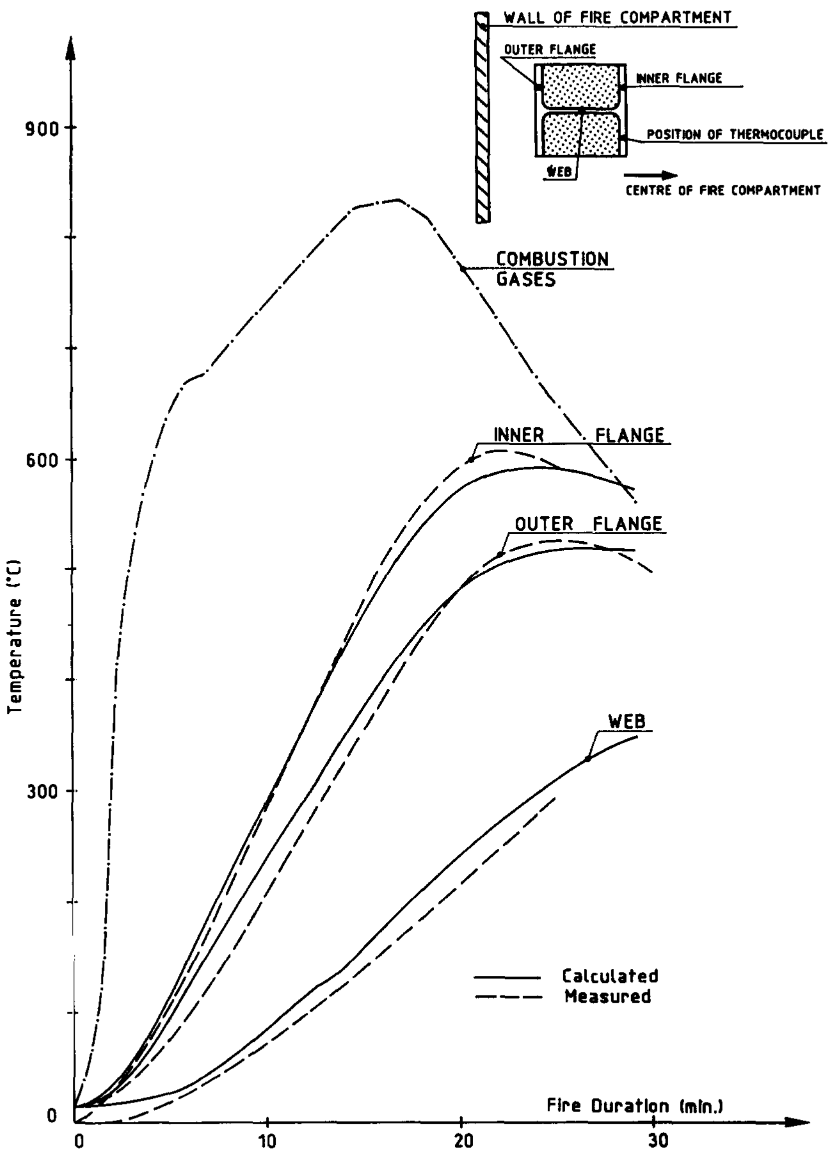


Fig. 6. Temperature in the column.

convective heat transfer coefficient of 25 W/m^{2}K was used as in the Eurocode. The lightweight (aerated) concrete blocks used to fill the column flange voids are considered to give only thermal insulation (they do not carry any load) and their thermal properties are assumed to be:

density = 677 (kg/m^3)
specific heat = 1050 (J/kgK)

thermal conductivity $= 0{\cdot}20 + 0{\cdot}0004 * T$ (W/mK)
moisture content $= 25{\cdot}7$ (kg/m^3).

Figure 6 shows the development of the average combustion gas temperature measured in the fire compartment and the development of calculated temperatures in the flanges and the middle of the web of the column. The computed temperatures agree well with the measured temperatures.

The discretization of the cross-section of the beam is shown in Fig. 7. The concrete cover slab is represented because of its influence on the temperature distribution in the beam. The concrete was given zero

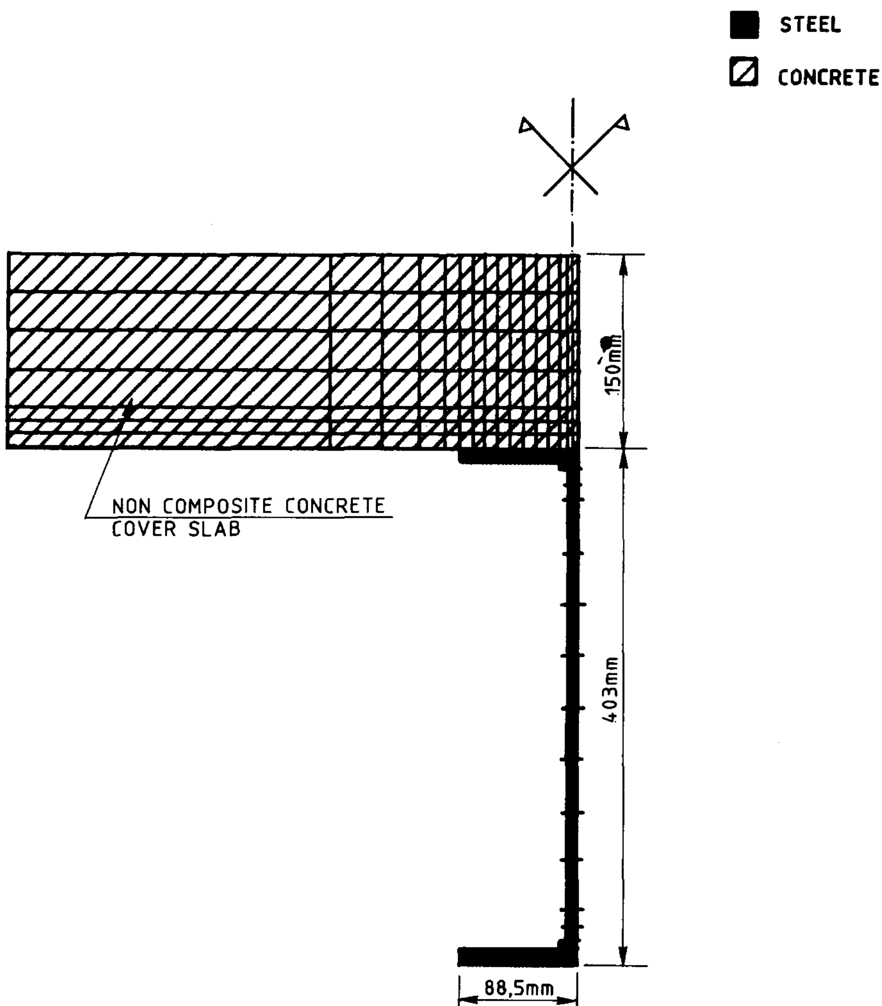


Fig. 7. Discretization of the beam cross-section and concrete cover slab by 141 rectangles.

strength because the slab makes no structural contribution due to its segmented form.

Figure 8 shows the development of the calculated temperatures in the steel beam compared with the measured data. The agreement is good, confirming that the transient temperature distribution in the profile can be accurately calculated by numerical methods using the recommendations of the Eurocodes for the thermal properties of materials.

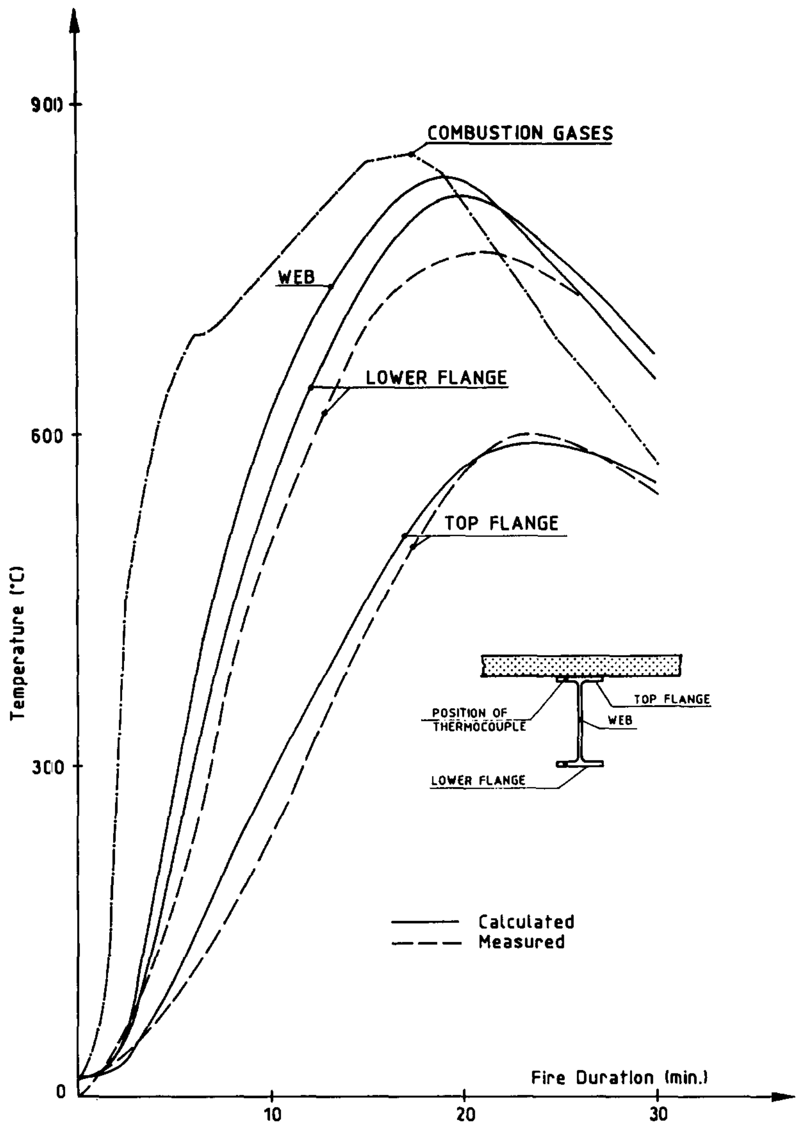


Fig. 8. Temperatures in the beam.

To represent the fact that the measured temperatures of the combustion gases were slightly lower in the vicinity of the beam-to-column connection, the increase of temperature in the beam has been multiplied by a reduction function ($f(x)$ given in 3.3) based on experimental results which has a sinusoidal shape along the beam axis with values of 0·90 at the beam/column interconnection and 1·00 at node 21, Fig. 5.

The vertical load on the columns, the two concentrated loads on the beam and the dead weight of the beam, columns and precast concrete slabs are all assumed to act on the frame.

The beam and column sections were made from BS 4360 Grade 43A steel. A hardness test confirmed that the steel satisfied the nominal tensile strength requirements in force at that time, i.e. 430-510 N/mm^2. As no tests on the actual yield strength of steel were made at the time of the test, five numerical simulations have been carried out using five different values of yield strength: 255 (the minimum requirement), 306, 357, 408 and 459 N/mm^2. Figure 9 shows the mid-span vertical deflection of the beam for the five yield strength simulations defined together with the changes in measured deflection during the fire test. Note that the calculated values of deflection include the deflection due to the load applied before the fire, whereas the measured deflection is the increase in deflection during the fire.

The curves confirm that the fire resistance of a structure is increased as the load factor is decreased (the higher the yield strength the lower the load factor). In the particular case of a so called 'natural fire curve' having a cooling down phase, it is possible that the structure remains stable during and after the fire, albeit with residual deflections, provided that the load factor is small enough. Because of the lack of knowledge of the actual yield strength of the steel used, it is not possible to be certain of the ability of the program accurately to predict the fire resistance of the frame. Nevertheless, the shapes of the calculated and measured deflection curves are very similar which suggests that the numerical model can predict the failure mode. It is observed that the fit between the measured and calculated deflection curves is best when using a yield strength of 408 N/mm^2. Although this value is higher than the statistical maximum normally associated with structural steel sections rolled to the Grade 43A specification, it has been used in the sensitivity analyses reported in this paper.

Various case studies ranging from Ia to VIb are illustrated in the Appendix. The calculated fire resistance time for the reference case, Case Ia, is 19 minutes and 12 seconds.

Figure 10 shows the calculated deflections of the frame at the very moment before collapse. The correlation between the calculation deflec-

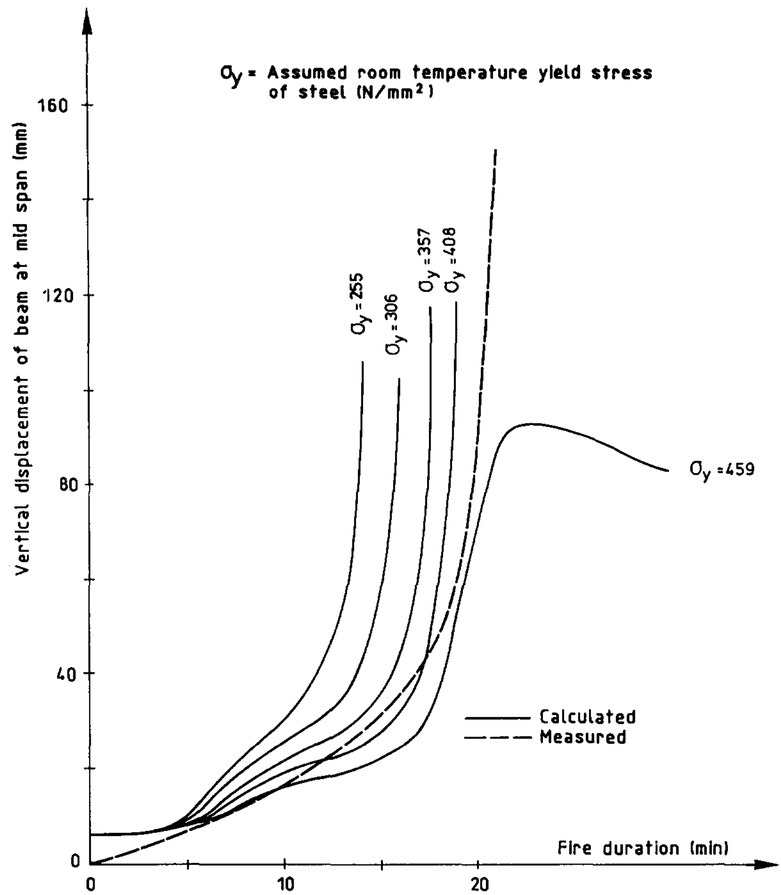


Fig. 9. Vertical displacement of the beam in the fire test.

tions of the column and the measured deflections in Fig. 3 is good. The failure mode is neither solely due to gross flexure of the beam nor buckling in the columns, but seems to concern both phenomena. The situation is complicated by internally induced bending moments caused by temperature gradients across the column and the beam section, Figs 6 and 8.

It has been verified that if the complete frame (rather than half the frame) is modelled assuming the presence of two springs and an initial lateral imperfection (sway) of 0·8H/1000 (where H = height of column), the failure mode is exactly the same and the calculated fire resistance is only very slightly increased from 19 minutes and 12 seconds (Case Ia) to 19 minutes and 22 seconds (Case Ib). This proves that satisfactory results are obtained by simulating only one half of the frame, provided that restraint members are present.

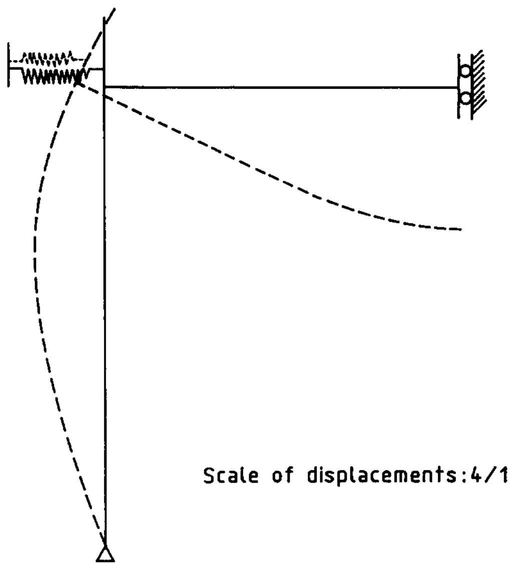


Fig. 10. Calculated failure mode of the restrained frame.

The photographs of the frame after the fire test indicated that no relative rotation at the connection occurred. The temperature in the vicinity of the connections remained lower than elsewhere in the compartment during the fire. This fact together with the additional mass of the connections compared with the members justified treating the connections as fully rigid in fire.

4.1 Influence of axial restraint to beam

Because of the bending stiffness of the columns and mainly because of the axial restraints, the beam cannot expand freely along its axis when it heats up. Thus an axial compressive force develops in the beam during the fire and this could influence the stability of the frame.

For the first sensitivity analysis, the fire behaviour of the frame has been simulated assuming that axial restraint does not exist. Symmetry about mid-span of the beam is assumed (Case IIa). Figure 11 shows that the axial compression force in the beam reaches peak values of 124 kN and 43 kN when restraint is present and absent respectively. At the moment of collapse, the axial compression force in the beam is reduced from 103 kN to 21 kN when lateral restraint is removed. Nevertheless, the failure mode remains the same and the fire resistance is only increased by 2%—from 19 minutes and 12 seconds with restraint (Case Ia) to 19 minutes and 35 seconds without restraint (Case IIa).

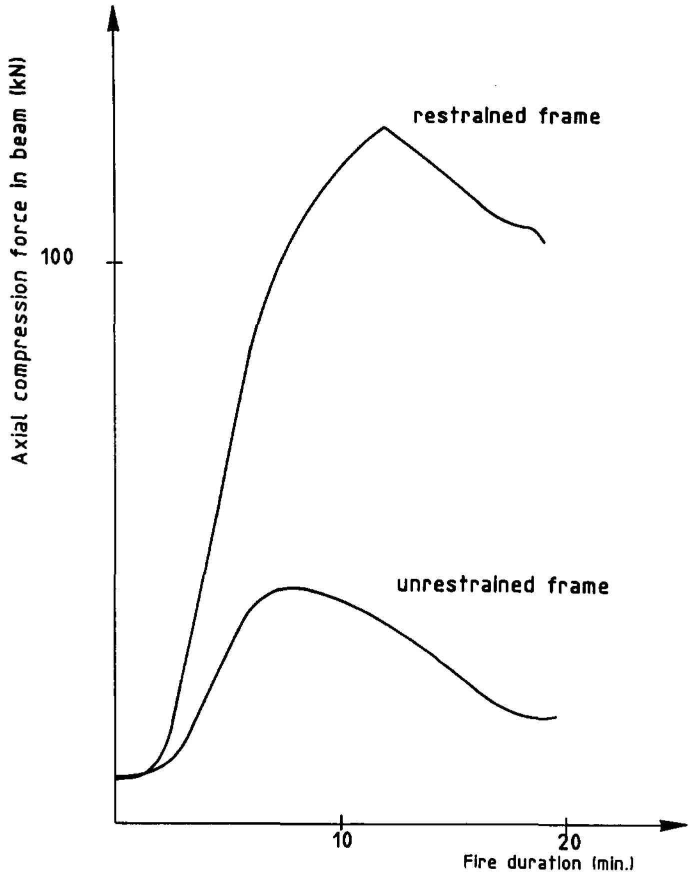


Fig. 11. Calculated axial force in the beam.

The complete frame with an initial lateral imperfection (out of plumb by 2·5 mm) has also been simulated without lateral restraint (Case IIb). This time, the failure mode of the frame is completely different if lateral restraint is removed (compare Figs 10 and 12). The structure sways with little vertical mid-span deflection of the beam which, at this moment, still possesses a high measure of stiffness. The fire resistance is reduced by 29%—from 19 minutes and 22 seconds for the restrained frame (Case Ib) to 13 minutes and 45 seconds for the same frame unrestrained (Case IIb).

4.2 Influence of frame continuity

It has been said that the provision of continuity could increase the fire resistance of a structure and that a complete structure does not behave as

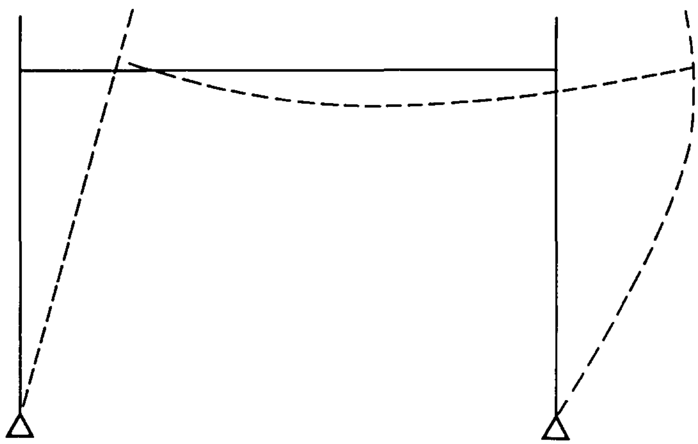


Fig. 12. Calculated failure mode of the sway frame.

the sum of its separate members. To illustrate and quantify this claim, the column and beam of the test frame have been analysed separately.

In addition to the load applied by the hydraulic jack, the column is subjected to the vertical force and bending moment introduced by the beam, Fig. 13. The values of the loads come from the analysis of the frame

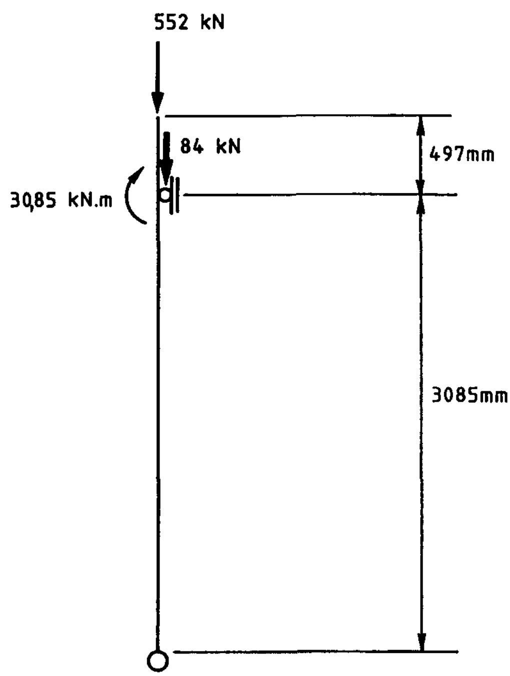


Fig. 13. The column as a separate member.

at ambient temperature and they remain constant in the analysis of the column as a separate member. The beam/column connection is not allowed to displace horizontally but is free to move vertically and rotate (Case IIIb).

Figure 14 shows the horizontal displacement at mid height of the column when acting as a separate member. At the beginning of the simulation, the column bows outwards because of the bending moment introduced by the beam. The development of the thermal gradient between the flanges, acting on a section that is still mainly elastic, then bows the column inwards. Later, when the flanges yield, the effect of the bending moment combines with the effect of the thermal gradient and the bowing is

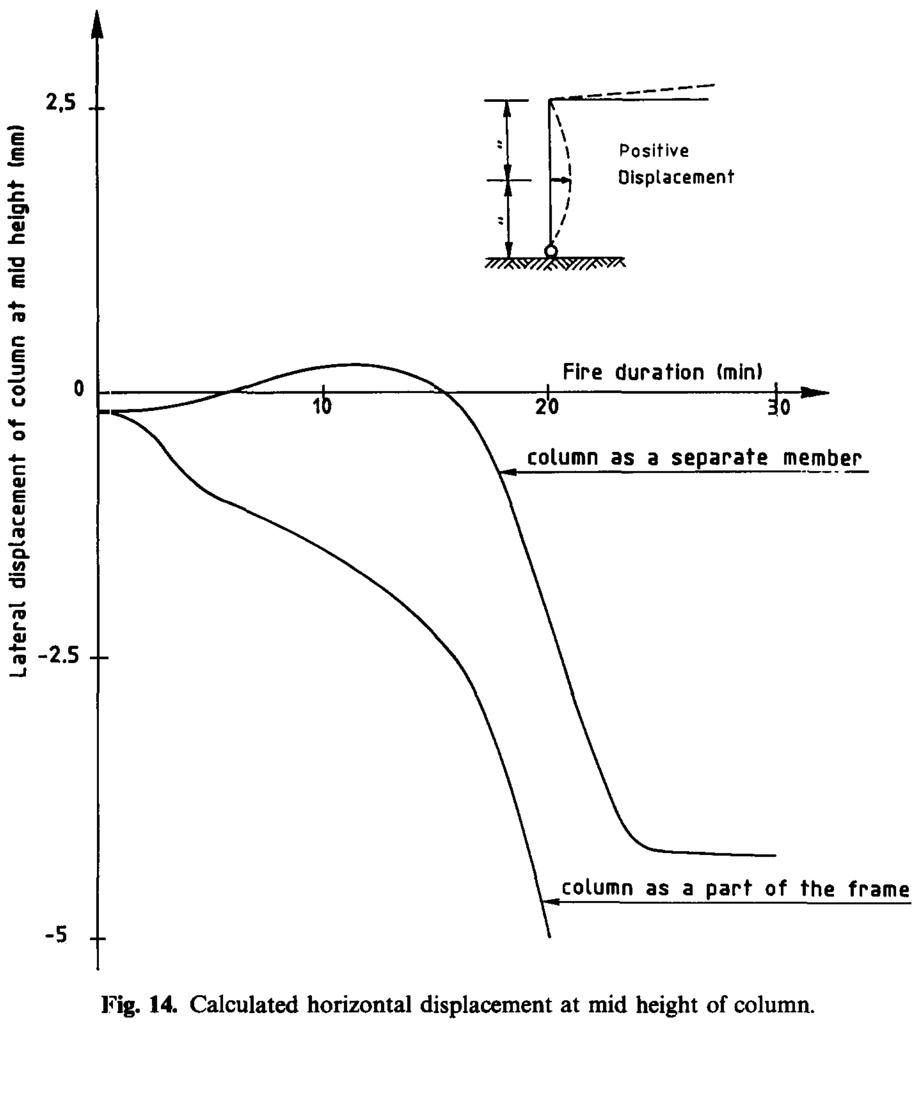


Fig. 14. Calculated horizontal displacement at mid height of column.

again to the outside. After approximately 25 minutes the section begins to cool down and, because the mechanical properties of structural steel are unaffected after cooling from 600°C,[24] the column remains stable and no collapse is observed.

The horizontal displacement at mid height of the column when acting as part of the frame is also shown in Fig. 14. The effect of the thermal gradient across the section of the beam and the elongation of the beam causes outward bowing of the column throughout the fire duration resulting in buckling after 19 minutes and 12 seconds (Case Ia).

The loads acting when the beam is considered on its own are shown in Fig. 15. In addition to the loads applied by the hydraulic jacks, the beam is subjected at its ends to the axial load and bending moment representing the frame effect at ambient temperature. The beam is free of externally applied axial restraint (Case IIIa). In the later stage of the fire the absence of beneficial restraint from the columns (which still have a large amount of stiffness) results in a reduction of fire resistance (15 minutes and 30 seconds) and larger displacements than for the complete frame.

In this particular case, the calculated fire resistance of the frame is increased by 24% when, instead of the sum of separate members, it is considered as a whole structure. This highlights the need for theoretical or numerical tools which enable the benefits of composite action to be quantified and reflected in more economic design.

4.3 Influence of thermal expansion

The numerical program CEFICOSS takes account of non-uniform temperature distributions in the cross-section and, to some extent, along the

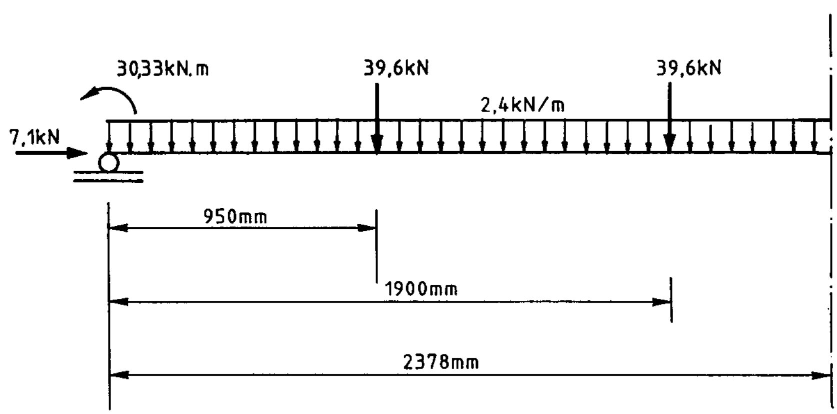


Fig. 15. The beam as a separate member.

length of the member, the influence of thermal strains, second order effects, non linear stress-strain relationships, and effects of large displacements, etc. This is integrated in a time history analysis, which means that the state of the structure is determined minute after minute up to the moment when no state of equilibrium can be found or, if desired, when a prescribed deflection is reached. This rigorous approach involves significant theoretical and numerical effort to write and validate the program, not to mention the degree of experience required by the user when simulating non-linear problems before confidence can be placed in the results obtained. The need for such complex analysis is debatable. Perhaps an ultimate state plastic design of the structure, considering the temperature dependent properties of steel, would be sufficient for most situations?

In an attempt to answer part of this question, the influence of thermal expansion has been investigated. The Cardington frame has been simulated assuming that steel does not expand when heated, a hypothetical condition (Case IVa). The time history simulation illustrates two main differences in frame behaviour.

First the axial force in the beam increases much less when thermal expansion is zero. From a value of 7 kN at ambient temperature, it reaches a peak value of 21 kN after 17 minutes instead of a maximum value of 124 kN for normal steel. However, axial force has been shown to have little effect on the stability of the frame. The bending moments developed in the frame are also smaller because the thermal gradients in the sections cause no thermal bowing. The positive bending moment in the beam at mid-span reduces as the stiffness and ultimate capacity reduce at that point. This reduction leads to an equivalent increase in the negative bending moment at the ends of the beam. The changes are caused by the progressive formation of a plastic hinge in the central part of the beam and not the thermal strains, and are less severe than the variations observed in the frame with normal steel.

Secondly, the absence of thermal expansion of the beam delays the lateral displacement of the beam-to-column connection, and the absence of thermal bowing (which was caused by the thermal gradients in the sections) delays the outward bowing of the column.

As a consequence, the simulation shows that the frame made of a hypothetical 'non-expanding' steel does not collapse but remains stable up to 30 minutes and probably well beyond this time because of cooling of the steel as the fire decays. The frame made of normal steel however, had a fire resistance of 19 minutes and 12 seconds (Case Ia). Such an important difference is mainly due to the decay of the fire and, as a consequence, the decrease in temperature of the steel sections. A detailed analysis of the results of Case IVa shows that the frame is very near to collapse when, after 21 minutes, the lower flange of the beam begins to cool down. A simulation of the same frame exposed to fire with increasing severity (the ISO 834 temperature-time curve for example) would lead to less spectacular differences between expanding and non-expanding steel.

4.4 Influence of non-uniform temperature

Part 10 of Eurocode 3[15] recommends a simplified method which assumes that the temperature of steel is uniform throughout the cross-section of the member. The uniform temperature has been calculated for the column and beam using the one-dimensional equation given in.[15]

For the column, the thermal conductivity and specific heat of the lightweight concrete blocks have been assumed to be zero. This allows the steel profile to be considered as thermally uninsulated but exposed to the action of the fire only on its flanges. The section factor (massivity) of the profile (heat-exposed perimeter/cross section area) is then equal to $(2B+4T)/A = 69\,m^{-1}$. To allow for the fact that the emissivity of steel is 0·30 on the outer flange and 0·50 on the inner, the simplified calculation has been made with an emissivity of 0·40. The simplified method in the Eurocode is not intended to cover the configuration for a partially insulated section and it is therefore not surprising that the uniform temperature calculated in this way is quite different to the real temperature.

For the beam, the simplified method is said, in Ref. 15, to be directly relevant. The massivity of the section, assuming that the top surface of the upper flange is not exposed to fire, is $193\,m^{-1}$. However, the heat sink effect of the cover slab is neglected so that the calculated uniform temperature is higher than in reality.

Figure 16 presents, for the column and beam, the uniform temperature calculated by the Eurocode simplified method and the mean temperature (mean value of the non-uniform temperature distribution calculated by the two-dimensional approach). In the column, a temperature of 500°C is reached 5 minutes earlier for the uniform temperature. This uniform temperature is also 125°C higher than the mean temperature at the end of the test. These differences may be due to the fact that the way the column section is insulated is not one of the traditional types foreseen for the simplified method. Of more concern is the difference between the uniform and mean temperatures in the beam since the simplified method is supposed to be valid for this type of unprotected section. The uniform temperature is higher than the average temperature by 102°C at 15 minutes. 817°C is the maximum value of the uniform temperature, reached after 20 minutes, whereas the maximum value of the mean temperature is 735°C.

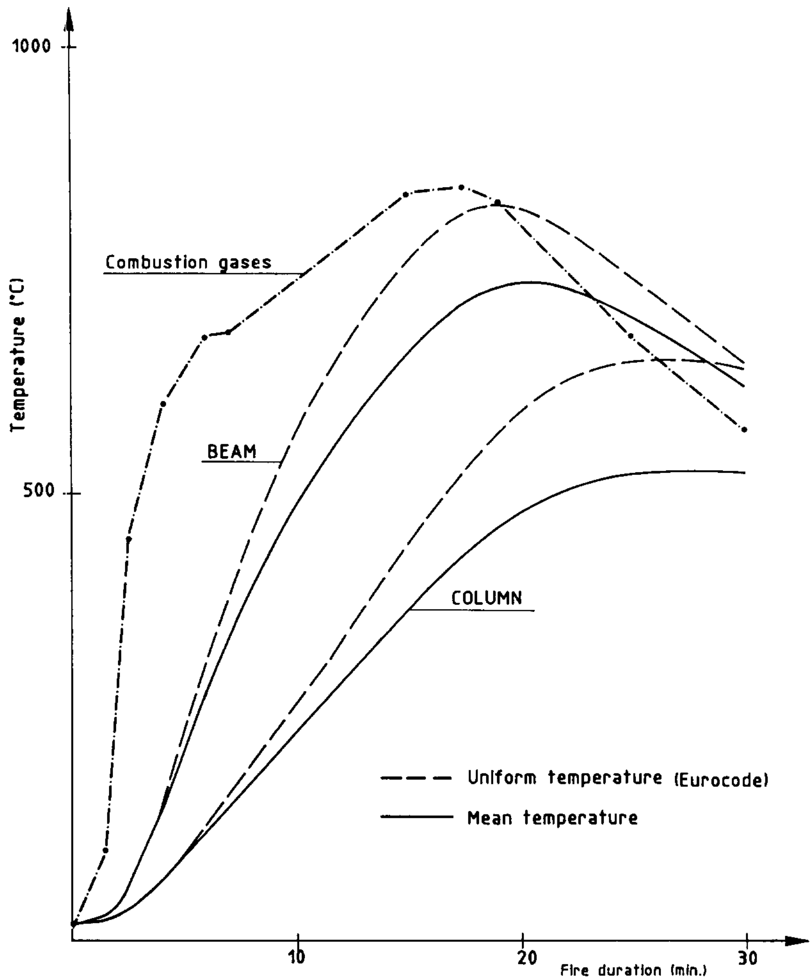


Fig. 16. Calculated uniform and mean temperatures.

Nevertheless, the unfavourable fact that the uniform temperature is higher than the mean temperature is partly compensated by the fact that the bowing caused by thermal gradients is not considered. The fire resistance calculated by the structural part of CEFICOSS on the basis of a simplified uniform temperature (Case Va) is 18 minutes and 15 seconds which is only one minute less than the reference Case Ia.

5 SIMPLIFIED PLASTIC DESIGN OF THE FRAME

The simple method described in Part 10 of Eurocode 3[15] is used here to assess the fire resistance of the members of the Cardington frame. The uniform temperatures reported in the previous paragraph are used. A yield strength of 408 N/mm^2 is used, as in the earlier simulations.

In the simplified method, the critical temperature of the elements is directly related to $k \times n$

where k = adaptation factor
n = load factor

For the beam, (Case VIa), the adaptation factor is 0·70 (because the beam is heated on three sides) multiplied by 0·85 (because the beam is hyperstatic, i.e. rotationally restrained, if the columns are assumed to remain stable for longer than the beam)

i.e., $k = 0{\cdot}595$.

The load factor is equal to the ratio between the isostatic (free) bending moment in the beam divided by twice the plastic moment of the section (plastic hinges are assumed to develop in the central part of the beam and at the supports).

$$n = \frac{M_{\text{centre}} - M_{\text{support}}}{2M_p}$$

$$= \frac{114{,}600{,}000}{2 \times 408 \times 7{,}048{,}000}$$

$$= 0{\cdot}134$$

so that $k \times n = 0{\cdot}595 \times 0{\cdot}134 = 0{\cdot}08$.

This ratio is reached when the steel has a critical temperature of 860°C. The uniform calculated temperature of the beam never reaches this value during the test and the fire resistance of the beam assessed by the simplified method is therefore greater than 30 minutes.

For the column, (Case VIb), the adaptation factor is 1·20 assuming the column is subjected to an axial force and a bending moment.

The load factor can be calculated assuming the following:

— buckling length in the plane of the frame = 0·80 × 3085 = 2468 mm according to Annex E of Part 1 of Eurocode 3[25]
— out of plane buckling length = 0·50 × 3582 = 1791 mm if out of plane rotation is prevented at the support and at the level of the hydraulic jack.

It can be shown that the minimum reduction factor is for in-plane buckling (curve c) and is equal to 0·903. According to 5.5.2 of Ref. 25,

$$n=\frac{636{,}000}{0{\cdot}903\times 408\times 6640}+\frac{1{\cdot}01\times 30{,}850{,}000}{408\times 568{,}100}=0{\cdot}394$$

and $k\times n=1{\cdot}20\times 0{\cdot}394=0{\cdot}473$.

This ratio is reached when the steel has a critical temperature of 500°C. In the column, this temperature is reached after 20 minutes and 30 seconds, i.e. only 7% more than the value provided by the numerical method.

6 CONCLUSIONS

(a) The numerical model, based on acknowledged principles of the theory of structures and utilising the recommendations for material properties given in Part 10 of the Structural Eurocodes, proved able to simulate with reasonable agreement the thermal and the structural behaviour of a full size steel frame tested in a fire compartment in the Fire Research Station's Large Laboratory at Cardington in 1987. The behaviour of the frame was correctly predicted up to failure, except for local buckling of the beam that occurred at the moment of failure which cannot be modelled using the beam finite element.

(b) The computer model has been used to highlight the influence of several physical parameters on the behaviour of the test frame:
- The value of yield strength at ambient temperature (which determines the load factor) has an important influence on the fire resistance of the structure.
- The increase of axial force in the beam due to the external restraint resulting from thermal expansion is significant, but it has a very limited influence on the fire resistance.
- A variation of the lateral in-plane restraint (provided to the frame to ensure a symmetrical failure mode) has a major effect on the fire resistance of the structure.
- The behaviour of the column and beam considered as separate members (i.e. no composite action) during the fire is totally different from the behaviour of the frame as a whole. The fire resistance of the weakest member (the beam) is considerably less than the fire resistance of the complete frame.
- The influence of the thermal expansion of steel cannot be neglected in the frame simulation because it proves to have a significant effect.

(c) The simple (Eurocode) method applied to the frame members to calculate the temperature in the sections as well as the structural behaviour provides a fire resistance time that is reasonably close to that of the frame when simulated by the general method using the more rigorous numerical model. Less safe results are provided by the simple method when applied to structures where there is no heat sink effect. The present work also suggests that the simple method should not be applied to sway frames.
(d) A good application of general (rigorous) computer programs similar to the one described here could be to identify which types of structure can be analysed by the simplified method and which types require the use of the general method.

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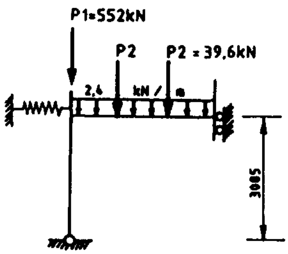
REFERENCES

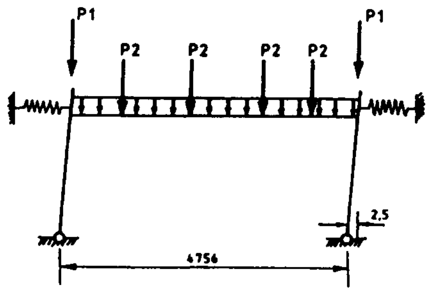
1. Arnault, P. *et al.*, Résistance au feu des systèmes hyperstatiques en acier. CTICM, doc *CECM 3-74/6 F*, 1974.
2. Cooke, G. M. E. & Latham, D. J., The inherent fire resistance of a loaded steel framework. *Steel Construction Today*, **1** (1987) 49–58.
3. Franssen, J. M., Etude du comportement au feu des structures mixtes acierbéton. Thèse de doctorat. Univ. de Liège, publications de la FSA, No 111, 1987.
4. Schleich, J. B. REFAO-CAFIR. Computer Assisted Analysis of the Fire Resistance of Steel and Composite Concrete - Steel Structures. CEC, Final Report *EUR 10828 EN*, 1987.
5. Burgess, I. W. *et al.*, A secant stiffness approach to the fire analysis of steel beams. *J. Construct. Steel Research.*, **11** (1988) 105–20.
6. Jeyarupalingam, N. & Virdi, K. S, Steel beams and columns exposed to fire hazard. In *Structural Design for Hazardous Loads—the role of physical testing*, (eds Clarke, J. L. *et al.*) E & F N Spon, London, 1992.
7. Forsen, N. E., *Steelfire—Finite Element Program for Non linear Analysis of Steel Frames Exposed to Fire. Users Manual.* Multiconsult A/S, Oslo, 1983.
8. Schaumann, P. Zur Berechnung Stählerner Bauteile und Rahmentragwerke unter Brandbeanspruchung. Technisch-wissenschaftliche Mitteilungen Nr. 84–4, Institut für Konstructiven Ingenieurbau, Ruhr-Universität Bochum, Dissertation, 1984.

9. Peterson, A., Finite Element Analysis of Structures at High Temperatures, with Special Application to Plane Steel Beams and Frames. Lund Inst. of Technology, Division of Struct. Mec., Report *TVSM-1001*, 1984.
10. Baba, S. & Nagura, H., Effect of material properties on the deformation of steel frame in fire. In *Proc. JSCE. Struct, Engng/Earthquake Engng.*, **2**, 1985.
11. Van Foeken, R. J. & Snijder, H. H., Steel column and frame stability analysis using finite element techniques. *Heron*, **30** (1985).
12. Jungbluth, O. & Gradwohl, W., Berechnen und Bemessen von Verbundprofilstaben bei Raumtemperatur und unter Brandeinwirkung. Ernst & Sohn, Verlag fur Architektur und technische Wissenschaften, Heft 382, 1987.
13. Valente, J., Simulacoa do comportamento das estruturas metalicas sujetitas a altas temperatures. Dissertacao. Instituto Superior Technico da Universidade Technica de Lisboa, 1988.
14. Furumura, F., A study on in-plane elasto-plastic creep behaviour of steel beam-columns at elevated temperatures. Report of the Research Laboratory of Engineering Materials, Tokyo Institute of Technology, Number 13, 1988.
15. Eurocode No 3. Design of Steel Structures, Part 10: Structural Fire Design. Draft April 1990.
16. Eurocode No 4. Design of Composite Structures. Part 10: Structural Fire Design. Draft April 1990.
17. European Recommendations for the Fire Safety of Steel Structures. *ECCS-TC3*, Elsevier, 1983.
18. Association of Structural Fire Protection Contractors and Manufacturers Ltd (ASFPCM), Fire protection for structural steel in buildings (2nd edition), 1988.
19. Luyckx, P., Elaboration d'un programme éléments-finis pour le calcul de la distribution de température dans les éléments de constuction soumis à l'incendie. Univ. of Liège, FSA, Travail de fin détudes, 1991.
20. Bresler, R. *et al.*, FIRES-T3, A computer program for the fire response of structures—thermal. Report No *UCB FRG 77-15*. University of California, Berkeley, 1977.
21. Boeraeve, P., Contribution à l'analyse statique non linéaire des structures mixtes planes formées de poutres, avec prise en compte des effets différés et des phases de construction. Thèse de doctorat, Univ. de Liège, Départment MSM, 1991.
22. Jennings, A., Frame analysis including change of geometry. *J. Struct. Engng.*, ASCE, **94** (1967).
23. Franssen, J. M., Modélisation et influence des contraintes résiduelles dans les profils métalliques soumis à l'incendie. *Construction Métallique*, CTICM No 3, 1989.
24. Kirby, B. R. *et al.*, *The reinstatement of fire damaged steel and iron framed structures*. BSC, Swinden Laboratories, 1986.
25. Eurocode No 3 Design of Steel Structures. Part 1. General Rules and Rules for Buildings. Draft December 1988.

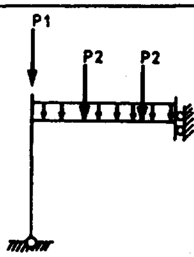
APPENDIX: LOAD CASES EXAMINED BY CALCULATION



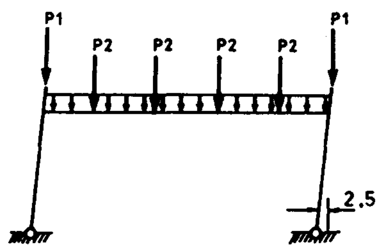
CASE I a ; Rf=19min.12sec. (CEFICOSS)
CASE IVa ; > 30min. If no thermal expansion (+CEFICOSS)
CASE Va : Rf=18min.15sec. If uniform temperature (+ CEFICOSS)



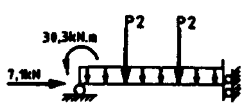
CASE I b ; 19min.22sec. (CEFICOSS)



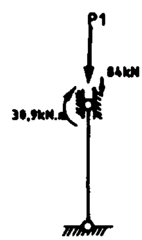
CASE IIa ; 19min.35sec. (CEFICOSS)



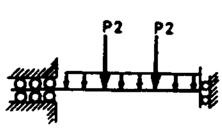
CASE IIb ; 13min.45sec. (CEFICOSS)



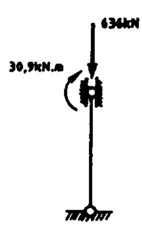
CASE III a ; 15min.30sec. (CEFICOSS)



CASE III b ; > 30min. (CEFICOSS)



CASE VIa ; > 30min. (simple method)



CASE VIb ; 20min.30sec. (simple method)