CI/SfB
| (28.9) | Hh2 | (K4)
Date February 1994

STUDY REPORT

NO. 55 (1994)

REPORT ON A FINITE ELEMENT PROGRAM FOR MODELLING THE STRUCTURAL RESPONSE OF STEEL BEAMS EXPOSED TO FIRE

C.A. Wade

This research was
supported by funding from
the Foundation for Research,
Science and Technology from the
Public Good Science Fund (PGSF)
and the Building Research Levy.

ISSN: 0113-3675

CONTENTS

	Page
PREFACE	1
ABSTRACT	1
INTRODUCTION	2
Aim	2
The Computer Program	2
NISA II - Summary of Capabilities	2
Test Data	2
Failure Conditions	3
Finite Element Model	4
MATERIAL PROPERTIES	5
Steel	5
Stress-Strain Curves	5
Thermal Expansion	6
Density	7
Poissons Ratio	7
Creep	7
BOUNDARY CONDITIONS	8
Specified Displacements	8
Dead Loads	8
Applied Loads and Moments	8
Temperatures	8
COMPARATIVE STUDIES	8
Test 1 (DS2) - Steel Beam Simply Supported	9
Description	9
Results and Comparison with Test Data	9
Test 2 (DS3) - Steel Beam Simply Supported	10
Description	10
Results and Comparison with Test Data	10
Test 3 (DS4) - Steel Beam Simply Supported	11
Description	11
Results and Comparison with Test Data	11
Test 4 (DS5) - Steel Beam Simply Supported	12
Description	12
Results and Comparison with Test Data	12
Test 5 (DS6) - Steel Beam Simply Supported	13
Description	13
Results and Comparison with Test Data	13
Test 6 (DS7) - Steel Beam Simply Supported	14
Description	14
Results and Comparison with Test Data	14
Test 7 (DS8) - Steel Beam Simply Supported	15
Description	15
Results and Comparison with Test Data	15
Test 8 (DS22) - Steel Beam with Rotational End Restraint	16
Description	16
Results and Comparison with Test Data	16
Test 9 (DS25) - Steel Beam with Rotational End Restraint	17
Description	17
Results and Comparison with Test Data	17
Test 10 (DS27) - Steel Beam with Rotational End Restraint	18
Description	18
Results and Comparison with Test Data	18
DISCUSSION	19

SUMMARY 22

REFERENCES 23

SYMBOLS 24

APPENDIX 25

FIGURES

Figure 1 : Typical Cross-section through Test Specimens 4

Figure 2 : General Representation of Test Specimen (Beam) 5

Figure 3 : Stress-Strain Curves for Grade 43 Steel using Ramberg-Osgood Model 7

Figure 4 : Stress-Strain Curves for Grade 50 Steel using Ramberg-Osgood Model 8

Figure 5 : BS Data Sheet 2 Simply Supported Bare Steel Beam 11

Figure 6 : BS Data Sheet 3 Simply Supported Bare Steel Beam 12

Figure 7 : BS Data Sheet 4 Simply Supported Bare Steel Beam 13

Figure 8 : BS Data Sheet 5 Simply Supported Bare Steel Beam 14

Figure 9 : BS Data Sheet 6 Simply Supported Bare Steel Beam 15

Figure 10 : BS Data Sheet 7 Simply Supported Bare Steel Beam 16

Figure 11 : BS Data Sheet 8 Simply Supported Bare Steel Beam 17

Figure 12 : BS Data Sheet 22 Steel Beam with Rotational End Restraint 18

Figure 13 : BS Data Sheet 25 Steel Beam with Rotational End Restraint 19

Figure 14 : BS Data Sheet 27 Steel Beam with Rotational End Restraint 20

Figure 15 : Stress-Strain Curves for Grade 50 Steel using Ramberg-Osgood Model *and Allowance for Creep* 22

Figure 16 : 406 x 178 mm x 60 kg/m Simply-Supported Bare Steel Beam - Consideration of Elevated Temperature Creep 22

Figure 17 : 406 x 178 mm x 60 kg.m Simply-Supported Bare Steel Beam - Effect of Creep on Time to Failure 23

Figure 18 : 356 x 171 mm x 67 kg/m Simply-Supported Bare Steel Beam - Effect of Lower Flange Temperature 24

TABLES

Table 1 : Parameters for Ramberg-Osgood Stress-Strain Relationship 7

Table 2 : Parameters for Ramberg-Osgood Stress-Strain Relationship with Allowance for Creep 22

REPORT ON A FINITE ELEMENT PROGRAM FOR MODELLING THE STRUCTURAL RESPONSE OF STEEL BEAMS EXPOSED TO FIRE.

BRANZ STUDY REPORT SR 55 C.A. WADE

REFERENCE

Report on a finite element programme for modelling the structural response of steel beams exposed to fire. Study Report SR 55, Judgeford, New Zealand.

KEYWORDS

Fire Resistance; Steel; Structural Analysis; Finite Element Method; I-beams, Simulation

ABSTRACT

This report summarises work carried out in a research project aimed at modelling, by computer, the performance of building components when exposed to fire. A finite element computer program (NISA) was used to predict deformations of steel beams. Useful results were obtained.

The study concludes that the finite element computer program (NISA) is useful for analysing fire-resisting performance. Structural steel I-sections have been modelled in this study, with relatively good agreement being achieved between calculated and measured values for temperatures up to about 600°C. At higher temperatures the effect of steel creep needs to be accounted for.

Further work is required to establish the same level of confidence for other materials and types of components.

INTRODUCTION

AIM

This report describes the use of a finite element program (NISA) for modelling deformations within building components exposed to fire. The components considered were bare steel I-section beams exposed to standard fire resistance tests. The work described in this report was carried out to:

- assess how useful the finite element program is likely to be for analysing fire-resisting performance of buildings;
- assess the accuracy of results obtained from the program by comparison with data recorded in standard fire resistance tests; and to
- thereby establish the utility of the program for further applications.

This study was not concerned with predicting the temperature response of the structural elements as that was the subject of a previous study (Wade, 1993).

Analytical modelling techniques will be useful for assessing building performance in cases where standard fire test results cannot be easily obtained experimentally. For example when considering frame actions and member continuity, or where different time-temperature curves are to be considered. It also provides for greater design flexibility at lesser cost than fire resistance tests.

THE COMPUTER PROGRAM

The computer program is a commercially available general-purpose finite element package capable of handling temperature-dependent material properties for non-linear structural analysis. The program is called NISA II and is supplied and supported by the Engineering Mechanics Research Corporation[1]. In addition to the module for non-linear static analysis there are pre- and post-processing modules for ease of model construction, finite element meshing and for analysis of results.

The computer program has already been used in a similar study of the thermal response of building components exposed to fire, carrying out transient heat transfer calculations (Wade, 1993). The results obtained were considered to be within acceptable limits of the experimental test data.

NISA II - SUMMARY OF CAPABILITIES

With respect to problems concerning structural analysis at fire temperatures, NISA II is capable of modelling the following factors (although not all features were required in this study):

- mechanical properties of materials which are temperature-dependent
- variations of temperature with time;
- 1, 2, or 3 dimensional analysis; and
- automatic finite element meshing routines.

A theoretical account of the finite element program will not be given here. It is documented in the NISA II Users Manual (EMRC, 1992) and follows conventional structural analysis theory.

TEST DATA

Experimental data were taken from the "Compendium of UK Standard Fire Test Data - Unprotected Structural Steel" (Wainman and Kirby, 1988). The data represent fire tests carried out in the UK, according to BS 476 Part 8 : 1972, (BSI, 1972) on hot rolled structural steel sections in which the members were either completely unprotected, or partially protected by materials used only in the fabric of a structure.

---

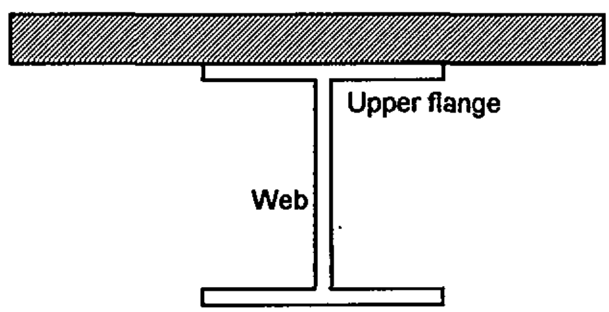
[1] Engineering Mechanics Research Corporation, PO Box 696, Troy, Michigan 48099 USA.

A selection of the test data was made for comparison with results from the computer model. In general, the specimens selected were bare steel I-section beams spanning across a fire resistance test furnace. The top surface was protected by a concrete slab, thus the beams are exposed to fire from three sides from underneath. A typical cross-section is shown in Figure 1. In all cases there was no mechanical fixing between the concrete slab and the steel section beneath, except for a thin gauge steel tang welded to the beam and cast into the concrete, designed to hold the units in position. The concrete slab was typically segmented into smaller lengths of approximately 1125 mm long and between 630 mm and 695 mm wide. Therefore, it was assumed that no composite action would occur between the concrete slab and steel beam.

In addition, vertical loads were applied to the top surface of the test specimen, through the concrete cover slab. Hydraulic jacks located at four points (viz 1/8, 3/8, 5/8 and 7/8 of the beam span) were used to apply the vertical loads.

The support conditions were simply-supported with or without rotational end restraint. Both cases are included in the tests selected for comparison.

FIGURE 1 : TYPICAL CROSS-SECTION THROUGH TEST SPECIMENS



FAILURE CONDITIONS

Failure occurs when the specimen can no longer support the test load. The onset of failure is preceded by an increasing rate of deflection of the beam and it is common for fire resistance test standards to define failure of load-bearing capacity in terms of the magnitude and the rate of beam deflection. BS 476 Part 20 (BSI, 1987) defines failure in the following way:

"The test specimen shall be deemed to have failed if it is no longer able to support the test load. For the purposes of this standard, this shall be taken as either of the following, whichever is exceeded first:

(a) a deflection of $L/20$; or

(b) where the rate of deflection (in mm/min), calculated over 1 min intervals, starting at 1 min from the commencement of the heating period, exceeds the limit set by the following equation:

$$\text{rate of deflection} = \frac{L^2}{9000d}$$

where

L is the clear span of the specimen (in mm)
d is the distance from the top of the structural section to the bottom of the design tension zone (in mm).

However, this rate of deflection limit shall not apply before a deflection of $L/30$ is exceeded."

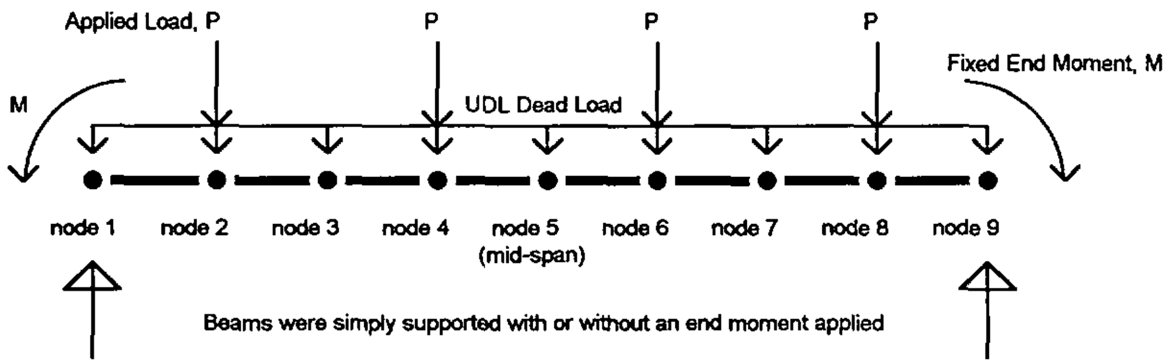
For modelling purposes, beam deflection can be used as a design criteria and therefore in this context, overestimates of deflection are conservative.

FINITE ELEMENT MODEL

The test specimens were modelled by eight 2-node finite (beam) elements as shown in Figure 2. This arrangement allowed the forces to be applied as concentrated nodal forces at Nodes 2, 4, 6, and 8, corresponding to the test load locations.

The prismatic beam elements had three translational and 3 rotational degrees of freedom at each node. The cross-section of the beam was described by specifying integrated quantities such as the cross-sectional area; moment of inertia; product of inertia; torsional constant; eccentricity; and depth. The latter dimension was to model the thermal strain due to the temperature difference between the upper and lower flanges.

FIGURE 2: GENERAL REPRESENTATION OF TEST SPECIMEN (BEAM)



MATERIAL PROPERTIES

STEEL

The test data used for comparison purposes in this study were obtained from the United Kingdom and the specimens were constructed with local Grade 43 and Grade 50 structural steel. These grades are roughly equivalent to AS 3679 Part 1 (SAA, 1990) steel types 250 and 350.

Stress-Strain Curves

The stress-strain behaviour of steel was approximated using a Ramberg-Osgood stress-strain curve as follows:

$$\varepsilon = \frac{\sigma}{E} + \frac{3}{7}\frac{\sigma_{0.7}}{E}\left(\frac{\sigma}{\sigma_{0.7}}\right)^n$$

where,

ε, σ = *the uniaxial stress and strain respectively,*

E = modulus of elasticity,

$\sigma_{0.7}$ = the uniaxial stress at a secant modulus of $0.7E$,

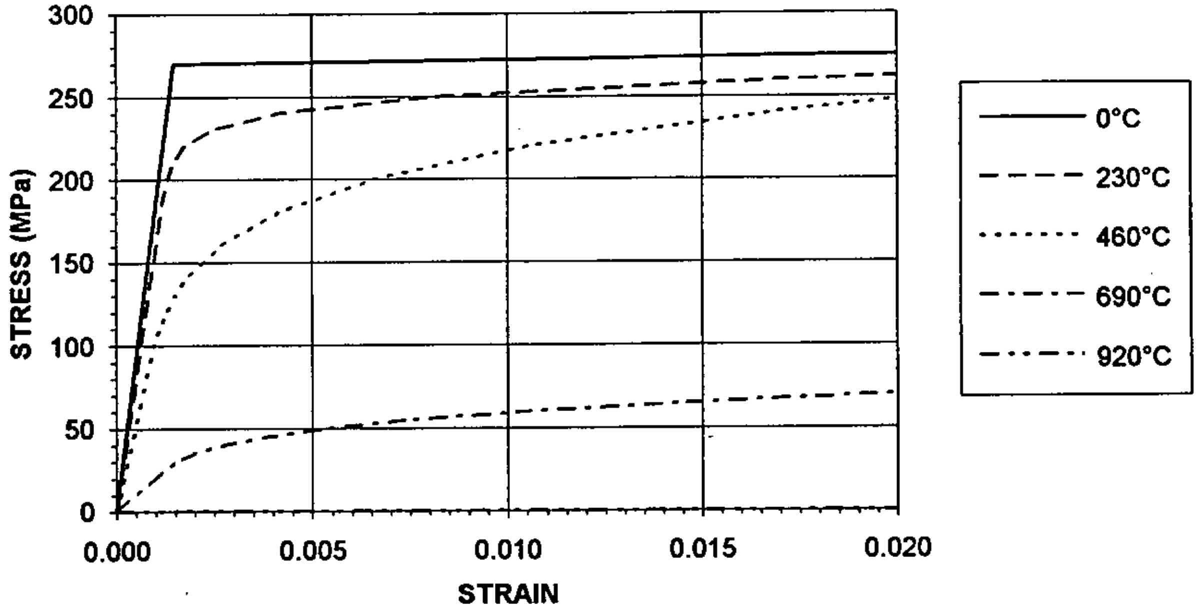
n = hardening index.

Values of $\sigma_{0.7}$, E *and* n *were selected to fit tabulated stress-strain data for British Grade 43 and Grade* 50 structural steel as given by SCI (1990) at 5 temperatures. The stress at 920°C was reduced to 0 in an attempt to keep the overall model conservative. The parameter values are given in Table 1. At intermediate temperatures the model linearly interpolates between the given values of $\sigma_{0.7}$, E and n, and thus may not match the tabulated data as closely as they do for the specified temperatures. The stress-strain curves for Grade 43 and Grade 50 structural steel, as were used in this study, are shown in Figures 3 and 4 respectively.

TABLE 1 : PARAMETERS FOR RAMBERG-OSGOOD STRESS-STRAIN RELATIONSHIP

		0°	230°	460°C	690°C	920°C
Grade 43	$\sigma_{0.7}$ (MPa)	274	225	140	35	0
	n	922	22.4	6.1	4.8	0
	E (MPa)	182900	158300	111100	25000	0
Grade 50	$\sigma_{0.7}$ (MPa)	354	305	206	67	0
	n	1105	30	7	8.6	0
	E (MPa)	188900	159400	111500	22500	0

FIGURE 3 : STRESS-STRAIN CURVES FOR GRADE 43 STEEL USING RAMBERG-OSGOOD MODEL

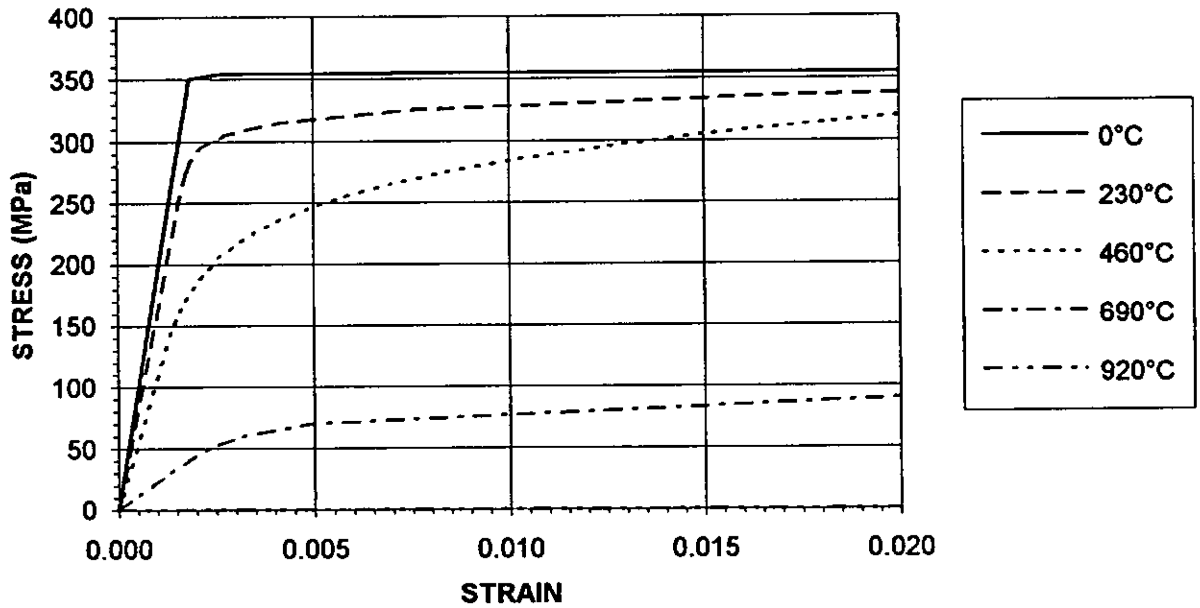


Thermal Expansion

The thermal expansion of steel is also a function of temperature. The coefficient of thermal expansion of steel (α) is given by Lie (1992) as follows:

$$\alpha = (0.004T + 12) \times 10^{-6} \ (°C^{-1}) \ \text{for } T < 1000°C$$
$$\alpha = 16 \times 10^{-6} \ (°C^{-1}) \ \text{for } T \geq 1000°C$$

FIGURE 4 : STRESS-STRAIN CURVES FOR GRADE 50 STEEL USING RAMBERG-OSGOOD MODEL



Density

The density of steel was assumed to remain constant at 7850 kg/m^3 for all grades of steel.

Poissons Ratio

Poissons ratio for steel was assumed to remain constant at 0.3 for all grades of steel.

Creep

Creep is a time-dependent deformation of a material which is significant for steel at temperatures greater than approximately 600°C. Harmathy and Stanzak (1970), and Anderberg (1986) discuss the creep properties of structural steels. The effect of creep will be discussed later in this report.

BOUNDARY CONDITIONS

SPECIFIED DISPLACEMENTS

Horizontal and vertical translation were prevented at the left end of the beam (Node 1) and vertical movement was prevented at the right end (Node 9). Rotation about the local x-axis was also prevented at both ends of the beam.

DEAD LOADS

The dead loads were applied as a uniformly distributed load along the span of the beam. These represent the self-weight of the steel beam and the self-weight of the concrete floor slab above.

APPLIED LOADS AND MOMENTS

The applied loads were in the form of concentrated nodal forces applied vertically at 1/8, 3/8, 5/8, and 7/8 positions along the beam span i.e., at Nodes 2, 4, 6, and 8. These corresponded to the loads used in the actual test.

In some tests there was some rotational end restraint, and in these cases, a moment was applied to Nodes 1 and 9 corresponding to the actual moment applied in the test. The general arrangement of the model is shown in Figure 2.

TEMPERATURES

Two characteristic temperatures were required to define the behaviour at each node. Firstly, the temperature difference between the top and bottom beam surfaces was required in order to calculate the beam deflection due to differential thermal expansion. The temperature gradient over the section depth was assumed to be linear.

A second temperature was required to define the stress-strain curve which applied to the whole cross-section. This required an approximation to be made, as the actual temperature varied over the depth of the member. It was decided to use the average lower flange temperature as this was generally higher than either the web or upper flange temperatures in the later stages of the test, and therefore gave a conservative estimate of the effect of the elevated steel temperature. A comparison of the effect of using the lower flange temperature was made using a more accurate approximation of the temperature distribution. This is shown in the Appendix and demonstrates that use of the lower flange temperature is likely to produce a conservative result. Use of the lower flange temperature is also supported by SCI (1990).

In this study the actual measured temperatures from the test data were used. The temperatures could also be predicted using NISA as shown by Wade (1993). Wainman et al. (1990) also publish predicted temperature profiles for a range of unprotected steel floor beams. However, a thermal analysis was beyond the scope of the present study which considered the structural (deformation) response only.

COMPARATIVE STUDIES

The following pages show comparisons between measured and calculated deflections at the mid-span of the steel beam for tests carried out according to the 1972 version of BS 476 Part 8 (BSI, 1972). The deflection limit of these tests was span/30 and hence the tests were normally terminated when this limit was reached. The limits have been extended in the more recent edition of the standard BS 476 Part 20 (BSI, 1987) as discussed earlier in this report.

TEST 1 (DS2) - STEEL BEAM SIMPLY SUPPORTED

Description

A 259 mm deep by 148 mm wide unprotected steel beam I-section (42.7 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 2. The upper flange was covered by a dense concrete slab 650 mm wide by 125 mm thick (1.785 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 13°C. The duration of the test was 23 minutes.

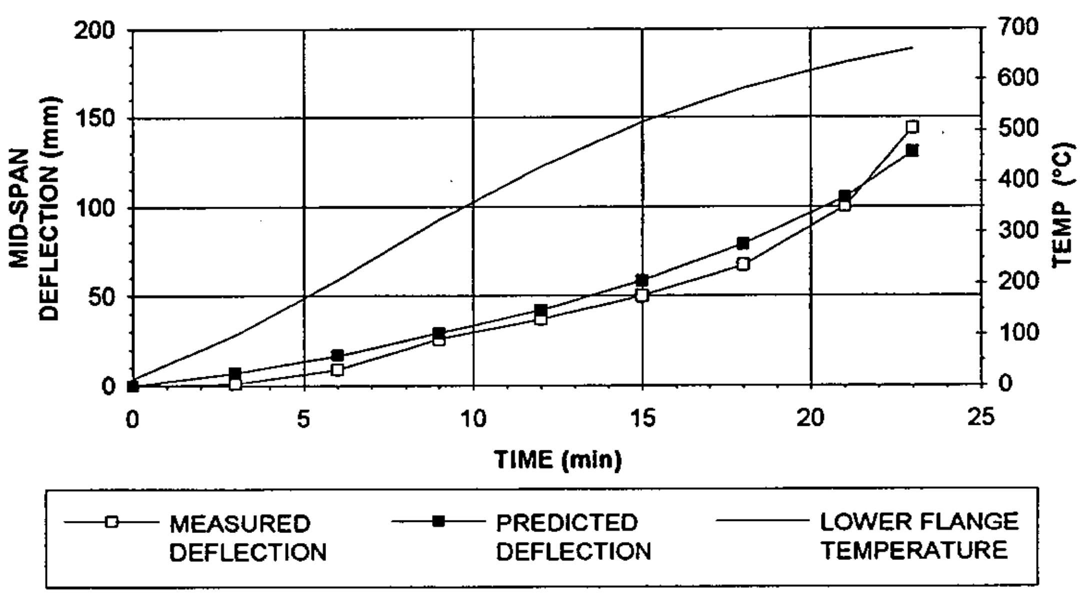
The properties of the beam were:

Flange thickness	12.3 mm
Web Thickness	7.6 mm
Cross-sectional Area	5.418E-03 m^2
Moment of Inertia (XX)	6359.8E-08 m^4
Moment of Inertia (YY)	677E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.204 kN/m
Concentrated Applied Load (each point)	46.73 kN
Span	4.585 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 5. A good agreement is observed between the measured and predicted deflection, although the rate of deflection toward the end of the test was reducing in agreement.

FIGURE 5 : BS DATA SHEET 2 - SIMPLY SUPPORTED BEAM



TEST 2 (DS3) - STEEL BEAM SIMPLY SUPPORTED

Description

A 260 mm deep by 147 mm wide unprotected steel beam I-section (43.3 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 3. The upper flange was covered by a dense concrete slab 650 mm wide by 125 mm thick (1.785 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 16°C. The duration of the test was 22 minutes.

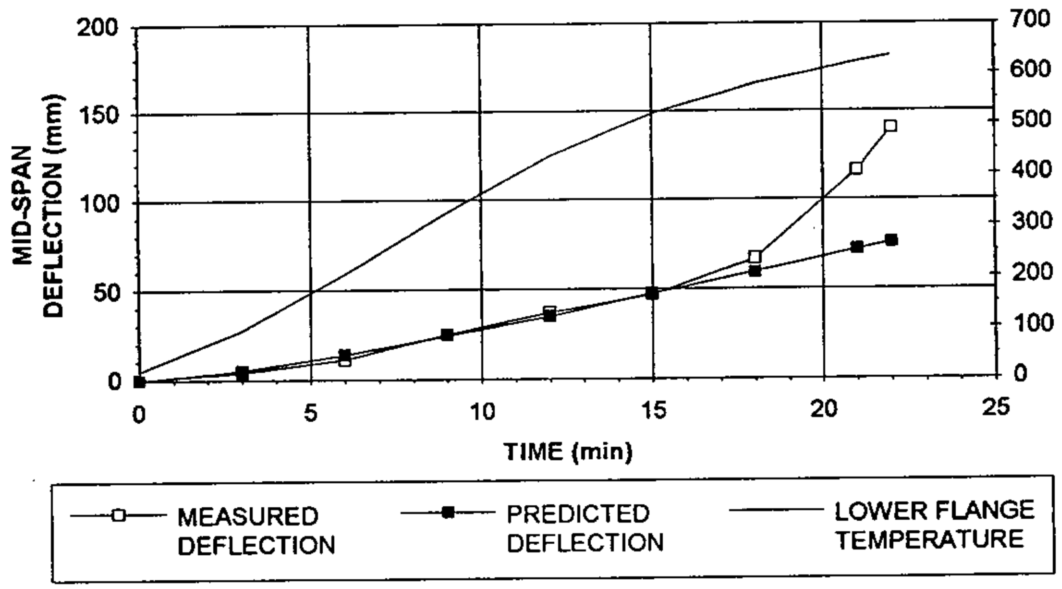
The properties of the beam were:

Flange thickness	12.7 mm
Web thickness	7.3 mm
Cross-sectional Area	5.4E-03 m^2
Moment of Inertia (XX)	6464E-08 m^4
Moment of Inertia (YY)	677E-08 m^4
Yield Strength at Room Temperature	275 MPa - grade 43
Modulus of Elasticity at Room Temperature	182900 MPa
Total Dead Load	2.21 kN/m
Concentrated Applied Load (each point)	32.54 kN
Span	4.585 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 6. There is excellent agreement between measured and calculated values for the first 18 minutes (and lower flange temperature up to 600°C), but thereafter the model predicts a lesser deflection than was measured in the test.

FIGURE 6 : BS DATA SHEET 3 - SIMPLY-SUPPORTED BEAM



TEST 3 (DS4) - STEEL BEAM SIMPLY SUPPORTED

Description

A 259 mm deep by 148 mm wide unprotected steel beam I-section (42.7 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 4. The upper flange was covered by a dense concrete slab 650 mm wide by 125 mm thick (1.785 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 15°C. The duration of the test was 30 minutes.

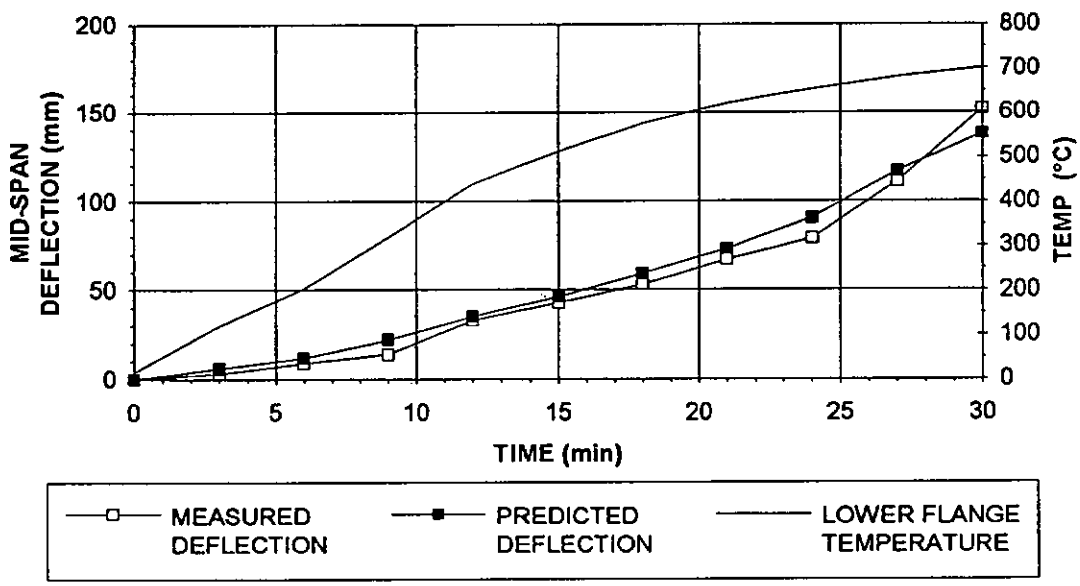
The properties of the beam were:

Flange thickness	12.4 mm
Web thickness	7.5 mm
Cross-sectional Area	5.4E-03 m^2
Moment of Inertia (XX)	6377.1E-08 m^4
Moment of Inertia (YY)	677E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.204 kN/m
Concentrated Applied Load (each point)	33.92 kN
Span	4.465 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 7. Good agreement is observed between the calculated and predicted deflections, again toward the end of the test the rate of deflection predicted is less than that measured.

FIGURE 7 : BS DATA SHEET 4 - SIMPLY-SUPPORTED BEAM



TEST 4 (DS5) - STEEL BEAM SIMPLY SUPPORTED

Description

A 363 mm deep by 172 mm wide unprotected steel beam I-section (65.6 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 5. The upper flange was covered by a dense concrete slab 670 mm wide by 125 mm thick (1.84 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 14°C. The duration of the test was 27 minutes.

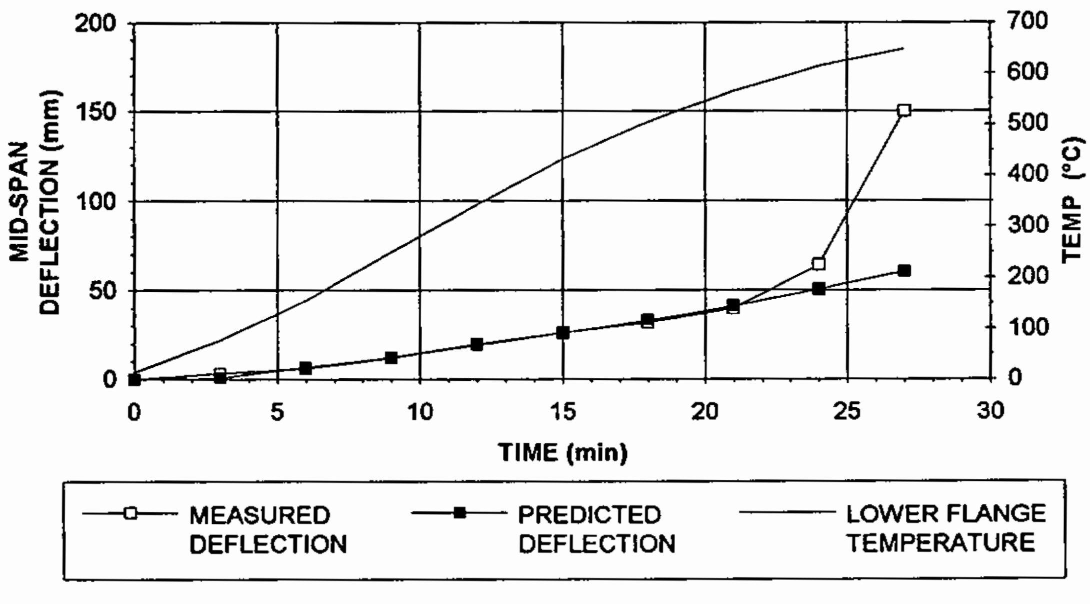
The properties of the beam were:

Flange thickness	15.3 mm
Web thickness	9.2 mm
Cross-sectional Area	8.33E-03 m^2
Moment of Inertia (XX)	18740E-08 m^4
Moment of Inertia (YY)	1362E-08 m^4
Yield Strength at Room Temperature	275 MPa - grade 43
Modulus of Elasticity at Room Temperature	182900 MPa
Total Dead Load	2.483 kN/m
Concentrated Applied Load (each point)	72.9 kN
Span	4.5 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 8. Excellent agreement is apparent for the first 21 minutes of the test (up to about 600°C), but thereafter the rapid increase in the measured rate of deflection is not predicted by the model.

FIGURE 8 : BS DATA SHEET 5 - SIMPLY-SUPPORTED BEAM



TEST 5 (DS6) - STEEL BEAM SIMPLY SUPPORTED

Description

A 403 mm deep by 173 mm wide unprotected steel beam I-section (59.8 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 6. The upper flange was covered by a dense concrete slab 665 mm wide by 125 mm thick (1.872 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 20°C. The duration of the test was 23 minutes.

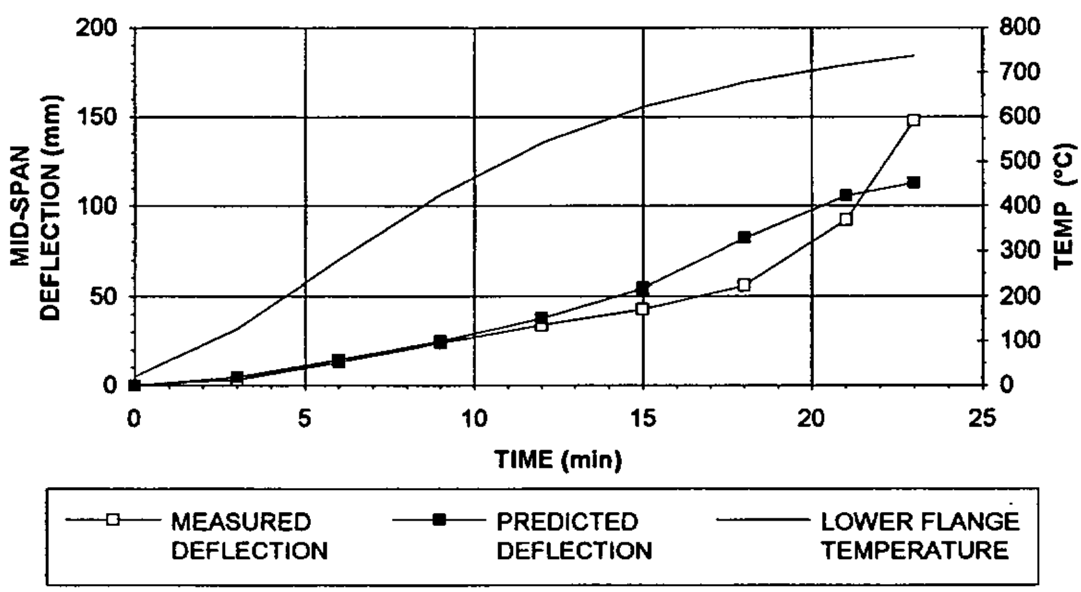
The properties of the beam were:

Flange thickness	12.8 mm
Web thickness	8.3 mm
Cross-sectional Area	7.564E-03 m^2
Moment of Inertia (XX)	20706E-08 m^4
Moment of Inertia (YY)	1199E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.413 kN/m
Concentrated Applied Load (each point)	72.62 kN
Span	4.5 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 9. There is excellent agreement for the first 12 minutes, then the differences become larger but conservative until 22 minutes when the predicted deflections are lower than those measured and the comparative rate of deflection is quite different.

FIGURE 9 : BS DATA SHEET 6 - SIMPLY-SUPPORTED BEAM



TEST 6 (DS7) - STEEL BEAM SIMPLY SUPPORTED

Description

A 314 mm deep by 166 mm wide unprotected steel beam I-section (53.2 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 7. The upper flange was covered by a dense concrete slab 660 mm wide by 125 mm thick (1.813 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 26°C. The duration of the test was 22.5 minutes.

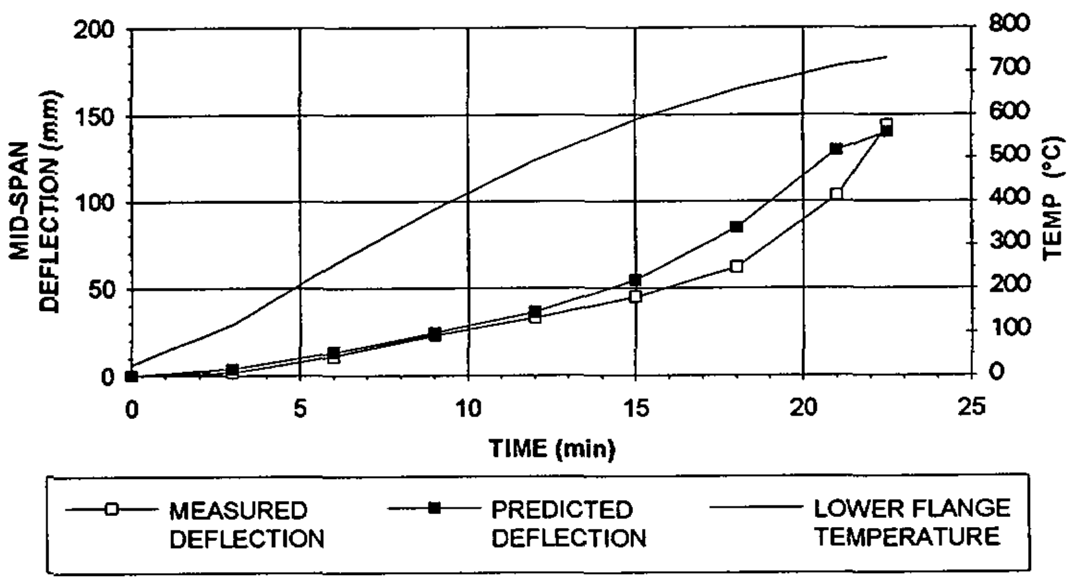
The properties of the beam were:

Flange thickness	13.5 mm
Web thickness	7.9 mm
Cross-sectional Area	6.75E-03 m^2
Moment of Inertia (XX)	11695E-08 m^4
Moment of Inertia (YY)	1061E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.334 kN/m
Concentrated Applied Load (each point)	51.6 kN
Span	4.53 m

Results and Comparison with Test Data

Comparison between measured and predicted deflections is shown in Figure 10. Agreement is excellent for the first 12 minutes, then the differences become larger but the predicted deflections remain conservative for the duration of the test. The rate of deflection predicted at the end of the test is less than that measured.

FIGURE 10 : BS DATA SHEET 7 - SIMPLY-SUPPORTED BEAM



TEST 7 (DS8) - STEEL BEAM SIMPLY SUPPORTED

Description

A 311 mm deep by 164 mm wide unprotected steel beam I-section (47.2 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 8. The upper flange was covered by a dense concrete slab 670 mm wide by 125 mm thick (1.84 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 25°C. The duration of the test was 22 minutes.

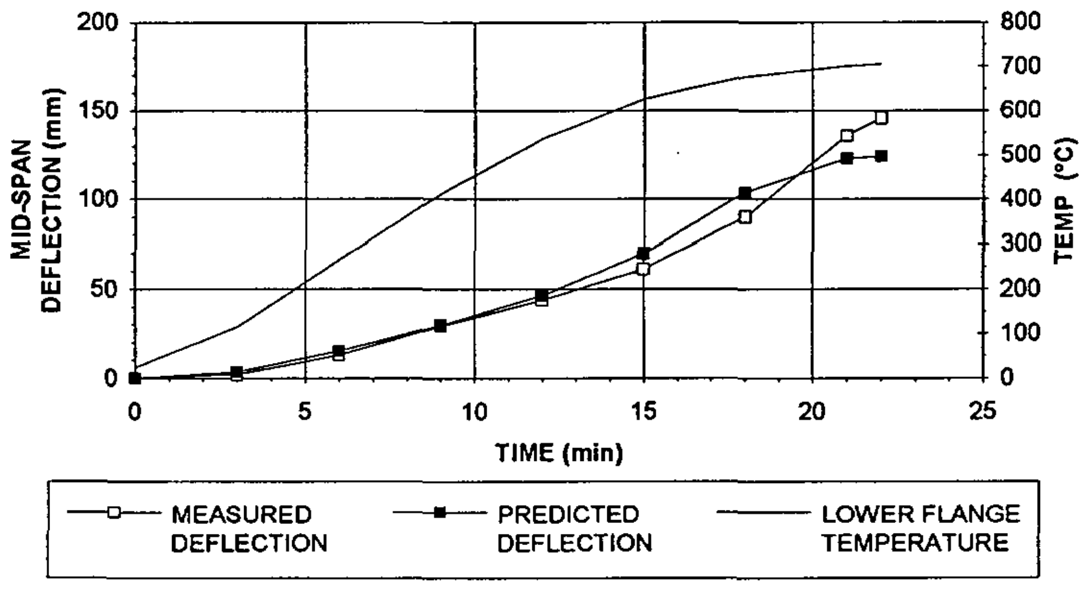
The properties of the beam were:

Flange thickness	11.5 mm
Web thickness	7.7 mm
Cross-sectional Area	5.99E-03 m^2
Moment of Inertia (XX)	9984.7E-08 m^4
Moment of Inertia (YY)	897E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.303 kN/m
Concentrated Applied Load (each point)	44.15 kN
Span	4.53 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 11. Similar trends as for the other tests are shown, with good agreement up to 15 minutes and lesser deflections predicted thereafter.

FIGURE 11 : BS DATA SHEET 8 - SIMPLY SUPPORTED BEAM



TEST 8 (DS22) - STEEL BEAM WITH ROTATIONAL END RESTRAINT

Description

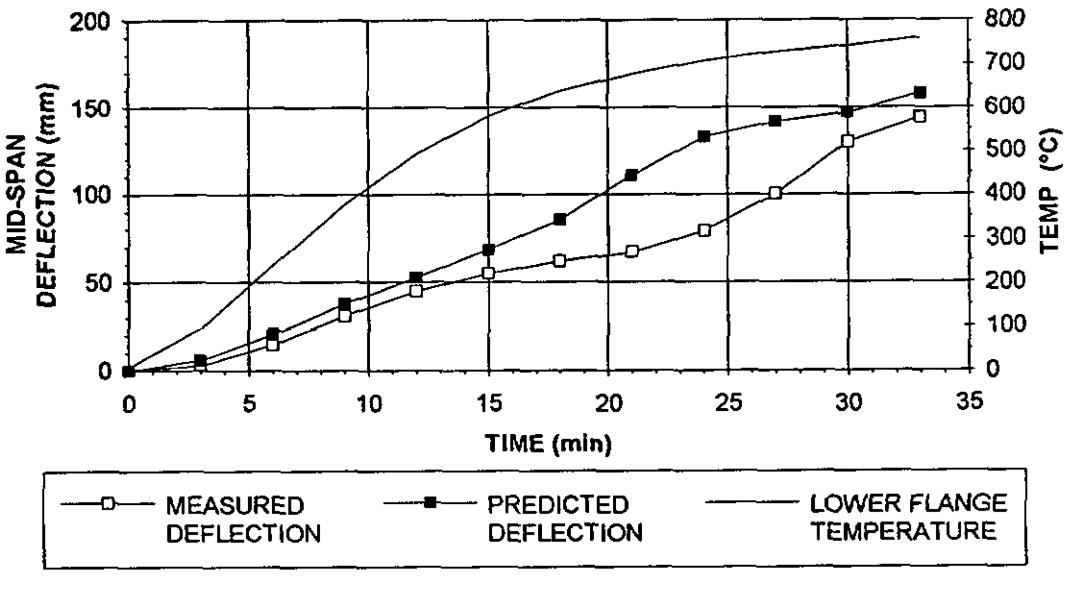
A 210 mm deep by 133 mm wide unprotected steel beam I-section (29.7 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 22. The upper flange was covered by a dense concrete slab 630 mm wide by 130 mm thick (1.8 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 7°C. The duration of the test was 33 minutes. The properties of the beam were:

Flange thickness	9.5 mm
Web thickness	6.5 mm
Cross-sectional Area	3.769E-03 m^2
Moment of Inertia (XX)	2929E-08 m^4
Moment of Inertia (YY)	384E-08 m^4
Yield Strength at Room Temperature	355 MPa
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.091 kN/m
Concentrated Applied Load (each point)	17.88 kN
Applied End Moments	13.46 kNm
Span	4.53 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 12. Agreement is good for the first 15 minutes, but thereafter the predicted deflections increase with lesser agreement with measured values. Agreement is better toward the end of the test with the model underestimating time to failure by 4 minutes.

FIGURE 12 : BS DATA SHEET 22 - SIMPLY SUPPORTED BEAM WITH ROTATIONAL END RESTRAINT



TEST 9 (DS25) - STEEL BEAM WITH ROTATIONAL END RESTRAINT

Description

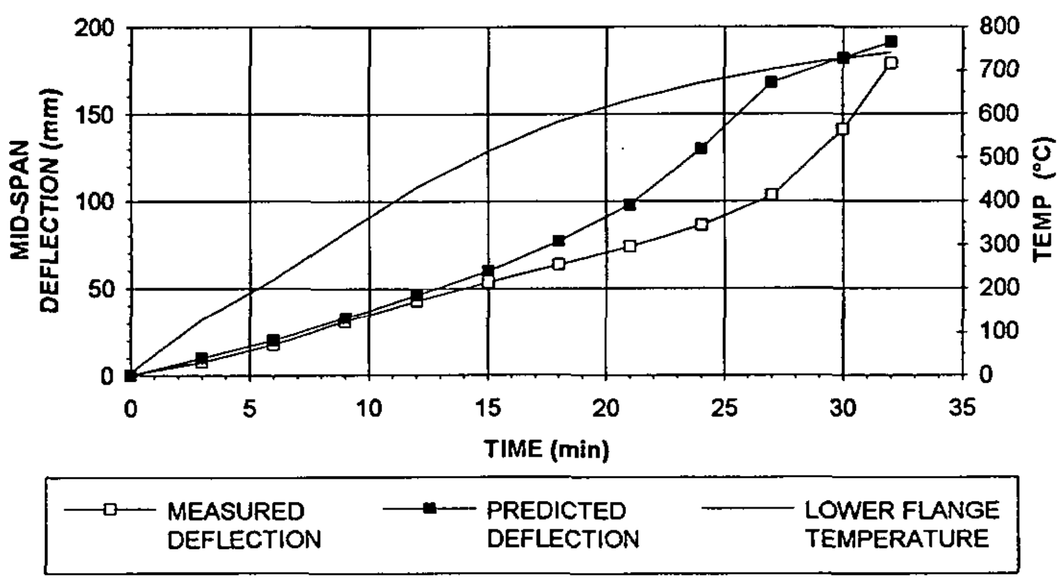
A 208 mm deep by 202 mm wide unprotected steel beam I-section (51 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 25. The upper flange was covered by a dense concrete slab 695 mm wide by 145 mm thick (2.214 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 8°C. The duration of the test was 32 minutes. The properties of the beam were:

Flange thickness	12.4 mm
Web thickness	8.1 mm
Cross-sectional Area	6.475E-03 m^2
Moment of Inertia (XX)	5201.2E-08 m^4
Moment of Inertia (YY)	1770E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.714 kN/m
Concentrated Applied Load (each point)	33.61 kN
Applied End Moments	8.2 kNm
Span	4.5 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 13. Agreement is very good in the early stages of the test, but as for the preceding test, the predicted deflections increase in the latter stages (but remain conservative). Agreement is closer toward the end of the test, with the model underestimating time to failure by about 5 minutes.

FIGURE 13 : BS DATA SHEET 22 - SIMPLY-SUPPORTED BEAM WITH ROTATIONAL END RESTRAINT



TEST 10 (DS27) - STEEL BEAM WITH ROTATIONAL END RESTRAINT

Description

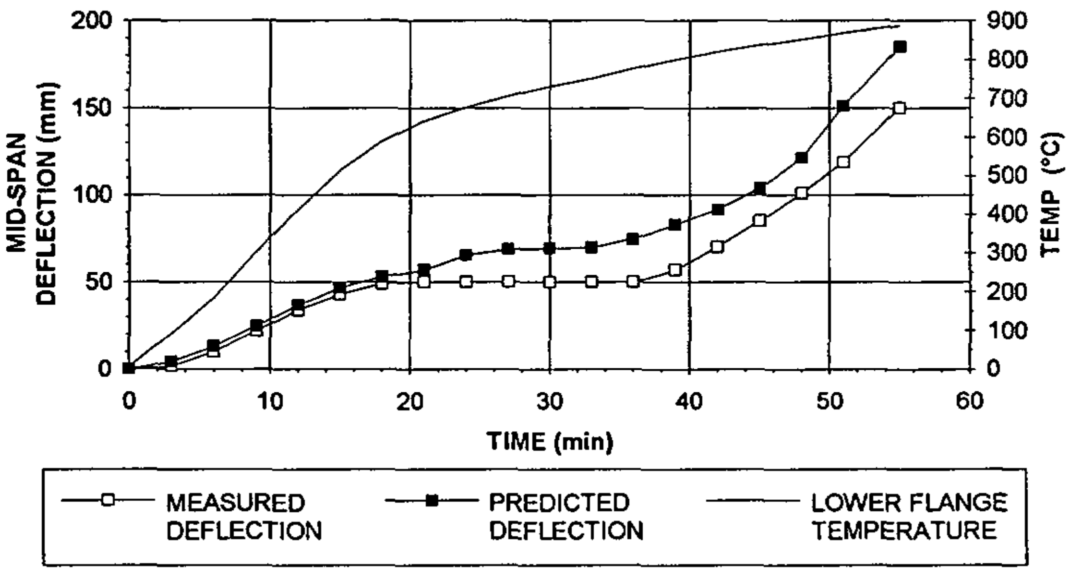
A 206 mm deep by 202 mm wide unprotected steel beam I-section (51.3 kg/m) was fire tested as described by Wainman and Kirby (1988) in Data Sheet 27. The upper flange was covered by a dense concrete slab 690 mm wide by 130 mm thick (1.971 kN/m). The time-temperature curve followed BS 476 Part 8 (BSI, 1972) and the initial temperature was 6°C. The duration of the test was 55 minutes. The properties of the beam were:

Flange thickness	12.5 mm
Web thickness	8.1 mm
Cross-sectional Area	6.507E-03 m^2
Moment of Inertia (XX)	5131.5E-08 m^4
Moment of Inertia (YY)	1770E-08 m^4
Yield Strength at Room Temperature	355 MPa - grade 50
Modulus of Elasticity at Room Temperature	188900 MPa
Total Dead Load	2.474 kN/m
Concentrated Applied Load (each point)	33.74 kN
Applied End Moments	56.79 kNm
Span	4.5 m

Results and Comparison with Test Data

A comparison between measured and predicted deflections is shown in Figure 14. Agreement is excellent for the first 21 minutes. Beyond that time differences are larger as shown, but predicted deflections remain conservative with the model underestimating time to failure by about 4 minutes.

FIGURE 14 : BS DATA SHEET 27 - SIMPLY-SUPPORTED BEAM WITH ROTATIONAL END RESTRAINT



DISCUSSION

Agreement for simply supported members was very good up to lower flange temperatures of 500 to 600°C. Thereafter the general trend was for the predicted deflection to fall short of the measured deflection. The most likely reason for this is creep at elevated temperatures. Anderberg (1986) has presented a comparison using finite element methods for a fire-exposed simply supported beam spanning 1.14 m, with and without consideration of creep. He noted a difference of about 3 minutes in the predicted collapse times.

A simplified method of approximating creep is to incorporate it implicitly in the stress-strain curves (by adjusting the curves downwards at the higher temperatures). A suitable series of stress-strain curves for Grade 50 structural steel steel is shown in Figure 15 (compare with Figure 4). Here the stress-strain curve at 690°C has been moved down (by 22 MPa at 0.02 strain) and the other curves remain the same. The Ramberg-Osgood parameters for the modified curves are given in Table 2.

This modified stress-strain model would be useful for design as it provides a better estimate of the time to failure.

The effect of using the modified stress-strain curve is shown in Figure 16, which compares the measured and predicted deflection for a simply-supported beam (loaded as described earlier for test 5). The predicted deflections are shown with and without consideration of creep, and using both the actual measured temperatures in the test, and the predicted temperatures given by Wainman et al. (1990).

The comparison using the actual temperatures on the lower flange of the beam shows that consideration of creep provided a better estimate of time to failure, although the predicted deflection still does not closely match that measured over the full test. The comparison between the two curves using predicted lower flange temperatures shows that the difference in time to failure (at about 150 mm mid-span deflection) is three to four minutes, consistent with Anderberg's (1986) observation.

FIGURE 15 : STRESS- STRAIN CURVES FOR GRADE 50 STEEL USING RAMBERG-OSGOOD MODEL AND ALLOWANCE FOR CREEP

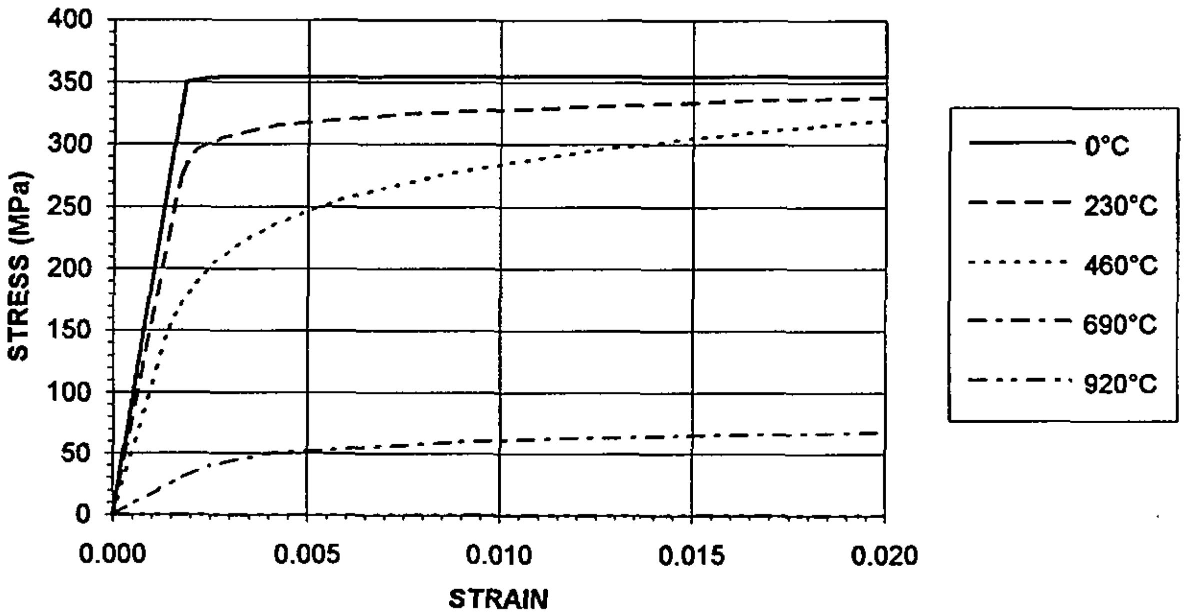


TABLE 2 : PARAMETERS FOR RAMBERG-OSGOOD STRESS-STRAIN RELATIONSHIP WITH ALLOWANCE FOR CREEP

		0°	230°	460°C	690°C	920°C
Grade 50	$\sigma_{0.7}$ (MPa)	354	305	206	51	0
	n	1105	30	7	8.8	0
	E (MPa)	188900	159400	111500	17000	0

FIGURE 16 : 406 x 178 MM x 60 KG/M SIMPLY SUPPORTED BARE STEEL BEAM - CONSIDERATION OF ELEVATED TEMPERATURE CREEP

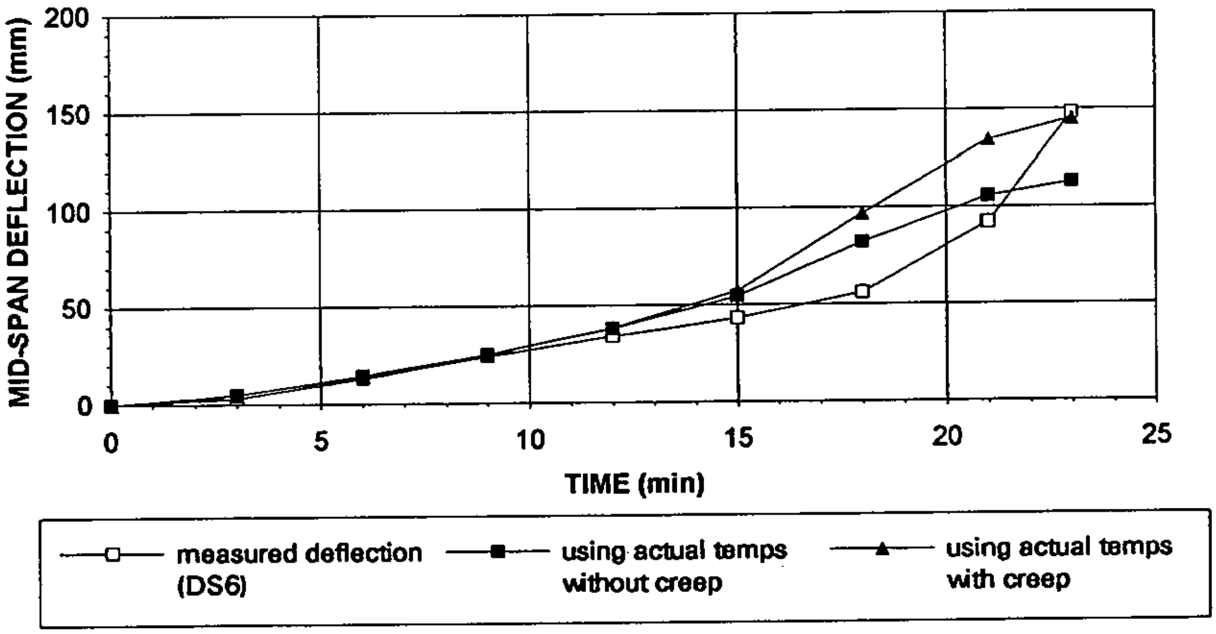
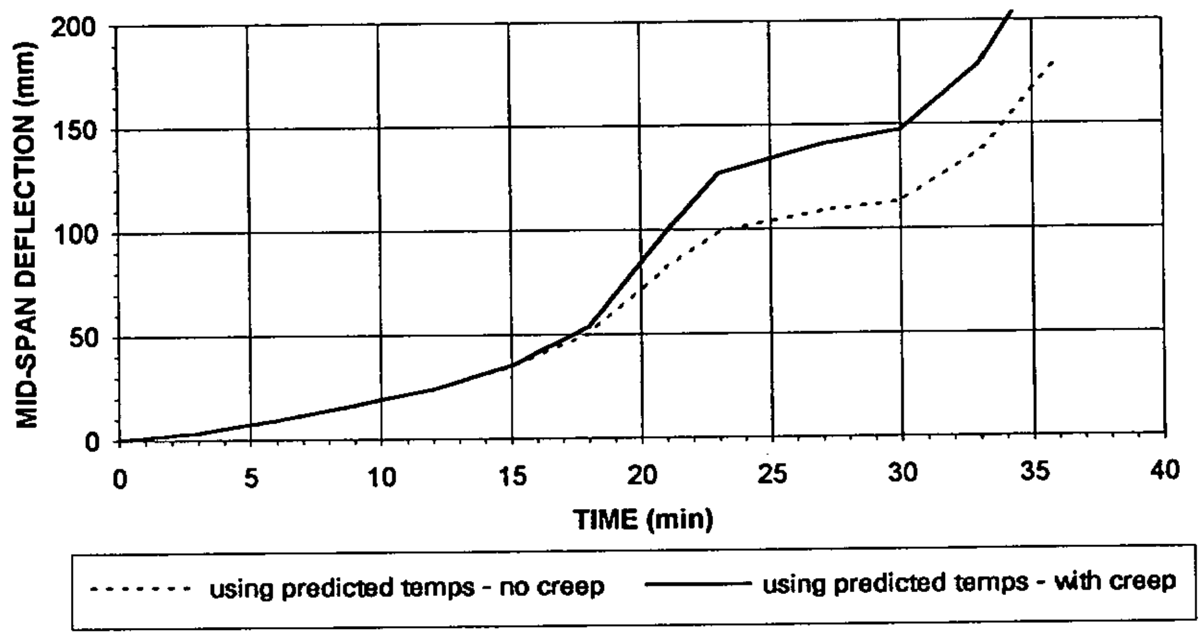


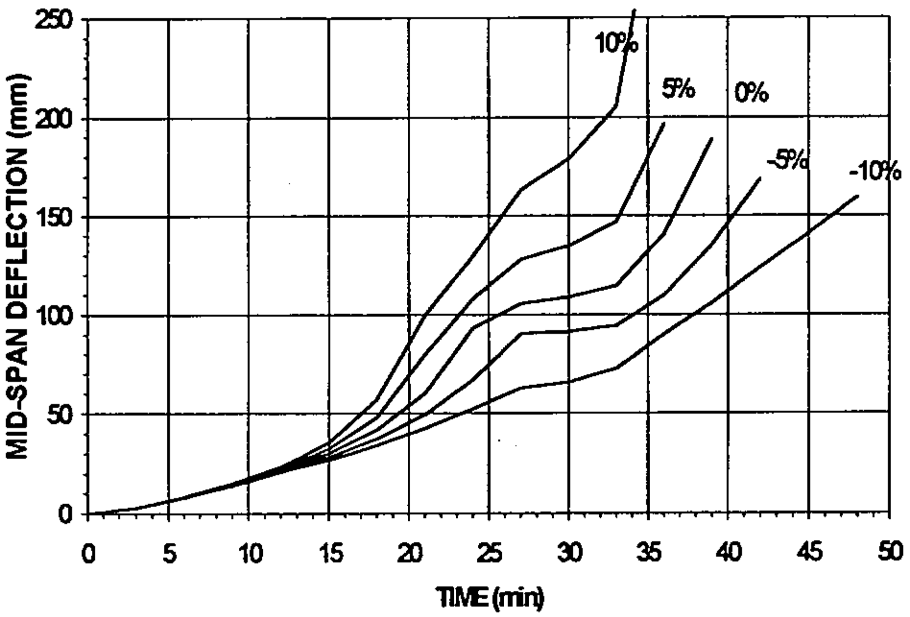
FIGURE 17 : 406 x 178 MM x 60 KG/M SIMPLY SUPPORTED BARE STEEL BEAM - EFFECT OF CREEP ON TIME TO FAILURE



Comparing Figures 16 and 17 above also indicates the sensitivity of the predicted deflection to the value of the lower flange temperature. A further comparison is shown in Figure 18 for a simply supported bare steel beam (loaded as described earlier for test 4). The lower flange temperatures were taken from Wainman and Kirby (1990) and the stress-strain curve given in Figure 3 was used. The model was run five times, changing the lower flange temperature by ±5% and ±10% as shown in the Figure. In this comparison the temperature difference between the top and bottom flanges was not changed.

It can be seen that relatively small changes in the lower flange temperature can result in significant differences in the recorded time to failure (e.g., a mid-span deflection of approximately 150 mm). A 3 to 4 minute difference is noted for a 5% variation in temperature, and 9 to 10 minutes for a 10% variation in temperature. Thus for design purposes it will be important not to underestimate the lower flange temperatures.

FIGURE 18 : 356 x 171 MM x 67 KG/M SIMPLY-SUPPORTED BARE STEEL BEAM - EFFECT OF LOWER FLANGE TEMPERATURE



SUMMARY

The finite element model (NISA II) described in this report can be usefully applied in analytical studies of the thermal response of steel I-beams exposed to fire.

The lower flange temperature of bare steel I-beams may be used as a characteristic temperature for determining the effect of elevated temperatures on the structural performance of the beam.

In particular it has been shown that good agreement between measured and predicted mid-span deflections are achieved for unprotected steel I-beams for lower flange temperatures up to 600°C.

At greater lower flange temperatures, agreement is not as good and it is concluded that elevated temperature creep has an important influence. For design purposes, or in order to achieve conservative estimates of likely time to failure, the stress-strain curves may be modified to incorporate an allowance for steel creep.

Further work is required to consider materials of construction and types of building element that differ from those considered in this study.

REFERENCES

Anderberg, Y. 1986. Modelling Steel Behaviour. Report LUTVDG/(TVBB-3028). Lund Institute of Technology. Lund, Sweden.

British Standards Institution. 1972. BS 476. Fire Tests on Building Materials and Structures. Part 8. London.

British Standards Institution. 1987. BS 476. Fire Tests on Building Materials and Structures. Part 20. Method for Determination of the Fire Resistance of Elements of Construction (general principles). London.

Engineering Mechanics Research Corporation (EMRC). 1992. NISA II User's Manual. Troy, Michigan.

European Convention for Constructional Steelwork (ECCS). 1985. Design Manual on the European Recommendations for Fire Safety of Steel Structures. Publication No 35. Brussels.

Harmathy, T.Z. and Stanzak, W.W. 1970. Elevated-Temperature Tensile and Creep Properties of Some Structural and Prestressing Steels. Fire Test Performance, ASTM STP 464, American Society for Testing and Materials. Philadelphia. pp186-208.

Lie, T.T. (ed). 1992. Structural Fire Protection. ASCE American Society of Civil Engineers. United States of America.

Standards Australia (SA). 1990. Hot-rolled Bars and Sections. SA 3679 Part 1. Sydney.

Standards New Zealand (SNZ). 1992. Steel Structures Standard. NZS 3404 Part 1. Wellington.

Steel Construction Institute (SCI). 1990. Fire Resistant Design of Steel Structures - A Handbook to BS 5950 : Part 8. Berkshire.

Wade, C.A. 1993. Summary Report on a Finite Element Program for Modelling the Thermal Response of Building Components Exposed to Fire. BRANZ Study Report No 53. Building Research Association of New Zealand. Judgeford.

Wainman, D.E.; Kirby, B.R. 1988. Compendium of UK Standard Fire Test Data Unprotected Structural Steel - 1. Ref. No. RS/RSC/S10328/1/87/B. British Steel Corporation, Swinden Laboratories, Rotherham.

Wainman, D.E.; Kirby, B.R. 1989. Compendium of UK Standard Fire Test Data Unprotected Structural Steel - 2. Ref. No. RS/R/S1199/8/88/B. British Steel Technical, Swinden Laboratories, Rotherham.

Wainman, D.E.; Kirby, B.R.; Tomlinson, L.N.; Kay, T.R.; Preston, R.R. 1990. The Behaviour of Unprotected Steel Floor Beams in the Standard Fire Resistance Test - Compendium of Predicted Temperature Profiles. British Steel Technical, Swinden Laboratories, Rotherham.

SYMBOLS

d = Distance from top of section to bottom of design tension zone (mm)
E_{20} = Modulus of Elasticity at 20°C (MPa)
E_T = Modulus of Elasticity at temperature T (MPa)
L = Clear span (mm)
n = Hardening index
T = Temperature (°C)
α = Coefficient of thermal expansion (m / m°C)
ε = Uniaxial steel stress
σ = Uniaxial steel strain
$\sigma_{0.7}$ = Uniaxial steel stress at a secant modulus of $0.7E$

APPENDIX

Justification for the use of the lower flange temperature as a characteristic temperature for determining the behaviour of the beam at elevated temperature.

A beam (data sheet 91) described by Wainman and Kirby (1989) was used as an example. The beam was a 356 mm x 171 mm x 67 kg/m I-section, with a flange thickness of 15.7 mm and a web thickness of 9.1 mm. The temperatures recorded at various locations on the beam at 12, 20 and 23 minutes into the test are as follows:

LOCATION	at 12 min	at 20 min	at 23 min
upper flange	233°C	390°C	456°C
web - 10 mm from upper flange	293°C	463°C	524°C
web - 30 mm from upper flange	376°C	547°C	601°C
web - 50 mm from upper flange	419°C	592°C	642°C
web - 100 mm from upper flange	483°C	650°C	694°C
web - mid-height	501°C	670°C	714°C
web - 100 mm from lower flange	489°C	673°C	714°C
web - 50 mm from lower flange	476°C	666°C	708°C
web - 30 mm from lower flange	447°C	650°C	697°C
web - 10 mm from lower flange	428°C	643°C	693°C
lower flange	447°C	657°C	705°C

The beam was divided into 11 sections each at the temperature noted above, and the elastic modulus as a proportion of its room temperature value was determined according to the following expression for the elastic modulus as a function of temperature from NZS 3404 (SNZ, 1992) and Lie (1992).

$$\frac{E_T}{E_{20}} = 1.0 + \left\{ \frac{T}{2000\left[\ln\left(\frac{T}{1100}\right)\right]} \right\} \text{ when } 0°\text{C} < T \leq 600°\text{C}$$

$$= \frac{690\left(1 - \frac{T}{1000}\right)}{T - 53.5} \text{ when } 600°\text{C} < T \leq 1000°\text{C}$$

A transformed section approach was used to calculate the effective second moment of area for the steel section. This was done by multiplying the width of each section by the ratio of the elevated temperature elastic modulus to its room temperature. The second moment of area for the I-beam was then calculated using the reduced section widths. The position of the neutral axis was also determined.

The results were :

	at 12 min	at 20 min	at 23 min
effective second moment of area (Ixx)	15878 cm^4	10245 cm^4	8569 cm^4
neutral axis above bottom surface (y)	216 mm	230 mm	239 mm

It is noted that the position of the neutral axis at 23 minutes had moved up by 57 mm from its original position at the start of the test.

Using Lower Flange Temperature - 657°C

The same approach was followed using a single temperature (that of the lower flange for the entire section) and the following results were obtained.

	at 12 min	at 20 min	at 23 min
effective second moment of area (Ixx)	12400 cm^4	6468 cm^4	5153 cm^4
neutral axis above bottom surface (y)	182 mm	182 mm	182 mm

It is noted that since a constant temperature was assumed across the entire cross-section the position of the neutral axis does not change with time.

This effective second moment of area (using the lower flange temperature) was lower than that obtained for the variable temperature distribution. The stiffness was therefore lower and the deflections would be greater.

Conclusion

The lower flange temperature gives a lower (i.e., conservative) effective second moment of area and therefore a lower stiffness than was obtained from the more accurate temperature distribution. Therefore it is conservative to use the lower flange temperature as a single characteristic temperature for the beam.

THE RESOURCE CENTRE FOR BUILDING EXCELLENCE

BRANZ MISSION

To promote better building through
the application of acquired knowledge,
technology and expertise.

HEAD OFFICE AND RESEARCH CENTRE

Moonshine Road, Judgeford
Postal Address - Private Bag 50908, Porirua
Telephone - (04) 235-7600, FAX - (04) 235-6070

REGIONAL ADVISORY OFFICES

AUCKLAND
Telephone - (09) 524-7018
FAX - (09) 524-7069
118 Carlton Gore Road, Newmarket
PO Box 99-186, Newmarket

WELLINGTON
Telephone - (04) 235-7600
FAX - (04) 235-6070
Moonshine Road, Judgeford

CHRISTCHURCH
Telephone - (03) 366-3435
FAX - (03) 366-8552
GRE Building
79-83 Hereford Street
PO Box 496