Structural behaviour and design of partially fire-exposed slender steel columns

by

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Contents

1 Abstract

Practical experience of the behaviour of partially (along the horizontal circumference) fire-exposed “slender” columns ($\lambda > 0,75$) seems to be completely missing. Only results on “short” columns are presented in literature. Due to this lack of knowledge a theoretical study on partially fire-exposed steel columns was performed in a major project “Multi-storey Structures, Behaviour in Case of Fire” /1/ was carried out 1998-2001. This paper presents the main results from this analytical study on slender columns.

Comprehensive analyses performed with the softwares Super-Tempcalc and Global Collapse Analysis (GCA) have shown that partially fire-exposed slender columns suffer considerably from a thermal gradient over the cross-section causing thermal bowing and increasing load eccentricity (see figure 5.3). The thermal moment caused by this cross-sectional gradient was found to be too large to be neglected. Exposure of about 50% of circumference was found to be the worst scenario for slender columns. It can be concluded that the collapse time will be reduced up to 75% at a load utilisation degree of 60 %. At a load utilisation of 25 % the reduction is “only” 35 % at 50 % exposure. Thus the fire resistance is also increasing by load utilisation level.

The best way to avoid the negative influence of partial fire exposure between 20 and 100% is to avoid this kind of exposure for slender columns. Otherwise the passive fire protection must be designed to resist partial fire exposure. In the design of partially fire-exposed slender columns figure 8.1-8.4 can be used for load utilisation levels 25-60% and a slenderness ratio up to 1.5.

In this study was also design methods of Eurocode 3 /2/ compared with non-linear analysis (GCA) and fire test results with the following conclusions.

Design methods including both plastic capacity and the critical Euler load, e.g. according to Eurocode 3, were found to be too advantageous compared with non-linear simulations. The simplified design methods of Eurocode gave in all calculations better fire resistance than results from FE-software. The safety factor to collapse obtained in tests with all 4 sides of the column exposed was too low and if a more comprehensive study was performed maybe the result will be even on the unsafe side. This is for beams mainly due to the fact of using full plastic capacity and a design strength related to 2% strain in combination. Similar discussion can be made for columns.

Keywords: fire resistance, slender columns, Eurocode, computer simulation, software, fire test

2 Introduction

In literature no full-scale fire tests on partially exposed columns of slenderness $\geq 0,75$ (“slender columns”) have been reported. Fire tests on partially exposed columns with slenderness $< 0,75$ (“short columns”) have been performed but no negative influence on the load-bearing capacity compared with fully exposed columns have been observed /9/. This is mainly due to the fact that the tested columns have been too short to be seriously influenced by a bowing effect due to the thermal gradient over the cross-section. Computer simulations indicate that there may exist a considerable problem for slender columns partially fire-exposed along its circumference.

This paper describes the results of a theoretical study on partially fire-exposed steel columns performed in a major project “Multi-storey Structures, Behaviour in Case of Fire” /1/ which is a project financed by the Swedish board of Fire Research. The results are discussed and a design proposal is presented.

3 Description of Analyses

3.1 Parameter Study for columns

A parameter study was carried out to analyse how different factors effect the load-bearing capacity for geometrical differences of fire:

- Fire exposure (ISO 834)
- Slenderness
- Axial load levels with different eccentricities
- Partially restrained elongation (not presented in this paper)
- Insulation (intumescent paint)

3.2 Simulation of fire tests

Simulations of fire tests were performed for verification of the non-linear Global Collapse Analysis (GCA) approach and corresponding, applicable design approach. Measured results from fire tests of unloaded and loaded steel columns and beams in terms of stress, strain and deformations are compared to the results from computer simulations of identical conditions. This concerns profiles that are exposed on all surfaces and cases where the degree of fire exposure vary.

Full scale fire tests of a beam and a fully exposed column conducted at Firto Borehamwood, U.K. 1984 /7/ have been simulated successfully.

4 Properties of Steel

The knowledge of thermal and mechanical properties are necessary in the thermal and structural analysis of fire-exposed steel columns. The temperature-dependence of the material properties are described in this section.

Thermal properties are essential to heat transfer calculations. These are, essentially, heat conductivity, heat capacitivity and thermal expansion, all of which vary with increasing temperature.

Stress-strain relationships (σ-ε curves) at elevated temperatures must be known to be able to calculate stress and deformations of fire exposed steel structures. Tension and compression strength and modulus of elasticity as function of temperature can be derived from σ-ε curves. All properties needed for computer calculations are taken from /2/

5 Fire Exposure

5.1 Definition of ~1/4, 1/2, ~3/4 and 1/1 fire exposure

In order to protect the exposed surfaces of the steel profiles, HEB300 and HEB200, a layer of intumescent paint (Hensotherm 4 Ks) is applied. This means that the temperature increase of the steel will be delayed and a fire resistance of about 50-60 minutes for 100% exposure and 40 % load is obtained. Four cases of partial fire exposure are studied viz. ~1/4 (19 %), 1/2 (50 %) , ~3/4 (83%) and 1/1 (100%), all defined as percentages of exposed surface for the profile HEB300. 19% exposure means that only the lower part of the bottom flange is exposed and the rest is not in contact with fire. This is arranged by surrounding the unexposed part with a material of concrete-type, which is illustrated in figure 4.1. 50% means that half of the profile is exposed and ~3/4 (83 %) means that only the upper part of the upper flange is not exposed to fire.

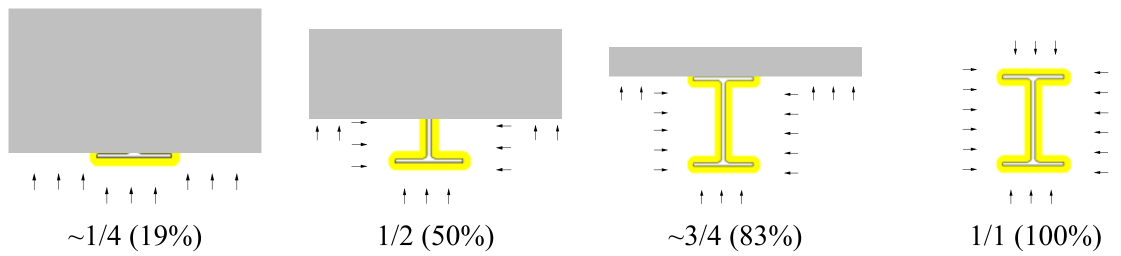


Figure 4.1 *Four cases of fire exposure studied.*

5.2 ISO 834 standard fire exposure

The standard ISO 834 time temperature development is described by

$$T(t) = 345 \cdot \log(480t + 1) + T_0 \qquad t > 0 \qquad \text{(eq. 4-1)}$$

where

- t = time (h)
- $T(t)$ = gas temperature at time t (°C)
- T_0 = initial temperature (°C)

The time temperature development is presented in a chart in figure 4.2.

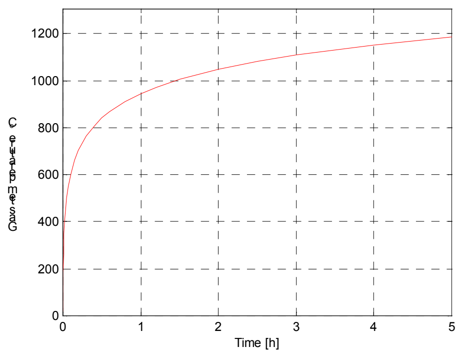


Figure 4.2 *Time-temperature development of ISO 834 fire exposure /2/.*

5.3 Boundary Conditions

The heat is transferred from the fire gases to the exposed structure through radiation and convection (see equation 4-2). The radiation, which dominates at high temperatures, is expressed by the resultant emissivity factor (second term of equation 4-2). The convection is calculated from the temperature difference between the structure and ambient gases, depending on the gas velocity (first term of equation 4-2). Resulting emissivity and convection factors used, are shown in table 4.1. These are in accordance with recommendations by ISO and Eurocode 1 /4/.

Emissivity/Convection	ε_r [-]	H_C [W/m^{2}K]
Exposed surface	0.56	25

Unexposed surface	0.8	9

Table 4.1 *Resulting emissivity and convection factor for ISO 834 fire exposure /2/.*

A boundary where no heat is allowed to pass ($q_n = 0$) is often referred to as an adiabatic boundary. These are for example symmetry lines.

6 Modelling Steel Columns

Four magnitudes of slenderness have been studied, viz. $\lambda = 1.5$, 1.0, 0.75 and 0.5. Since the cross-section profile has been selected (HEB300) and the material parameters are determined, the corresponding lengths are 8.7 m, 5.8 m and 2.9 m respectively for a column restrained at the bottom and free at the top.

Steel cross-sections are generally divided into four classes depending on their ability to form a plastic hinge. The method is practically the same in all design codes; here the Eurocode version is represented.

6.1 Definition of Slenderness

Slenderness expresses the degree of sensitivity to the buckling phenomenon (flexural buckling), and links the strength, the length, the stiffness and the cross-section dimensions together. The slenderness ratio, λ, is defined in equation 5-1. The ratio relates the characteristic compression resistance, $N_{c.R}$, to the critical axial buckling force, N_{cr}, according to the Euler buckling theory.

$$\lambda = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \frac{l_c}{\pi i}\sqrt{\frac{f_y}{E}} \qquad \text{(eq. 5-1)}$$

$$N_{c.R} = A \cdot f_y \qquad \text{(eq. 5-2)}$$

$$N_{cr} = \frac{\pi^2 EI}{l_c^2} \qquad \text{(eq. 5-3)}$$

where

$l_c =$ buckling length of column

Equation 5-1 indicates the turning point between two different modes of failure. Values of λ exceeding 1.0 applies to a buckling failure mainly in accordance with equation 5-3 (flexural buckling). For lower values of λ the failure will be governed by the cross-sectional stress reaching the yield strength. Relating the loading to the characteristic compression resistance and making it a function of the slenderness ratio, the curve N_{cr}/N_{pl} shown in fig 5.1 will be obtained.

The following parameters are all affecting steel columns in one way or the other:

- initial out of straightness
- unintentional eccentricity
- secondary geometrical effects
- initial stresses
- plasticizing during the buckling process

Taking these effects into account, the principal reduction curve in fig 5.1 is obtained.

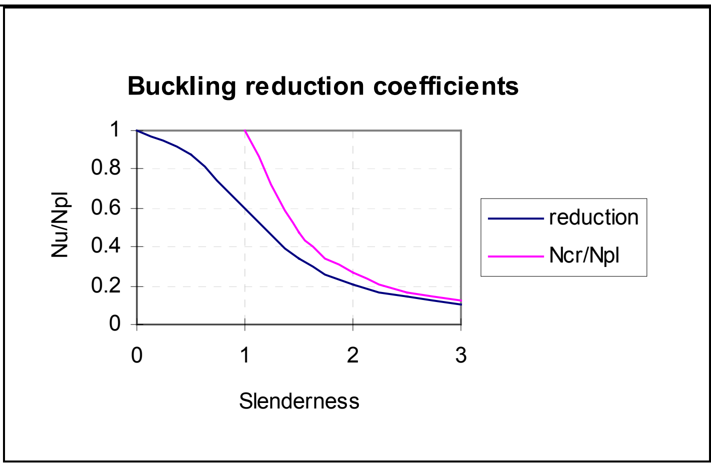


Figure 5.1 *Load-bearing capacity for steel columns as a function of the slenderness ratio.*

The reduction parameter in fig 5.1 is denoted χ. It is defined by the ratio between the characteristic buckling compression resistance and the characteristic plastic compression resistance (equation 5-4).

The characteristic buckling resistance of a compression member can be defined as:

$$N_{b.R} = \chi \cdot N_{c.R} \qquad \text{(eq. 5-4)}$$

Slenderness and stiffness, defined as the product *EI*, are two essential parameters concerning columns. A decrease in stiffness will cause an increase in slenderness, and vice versa.

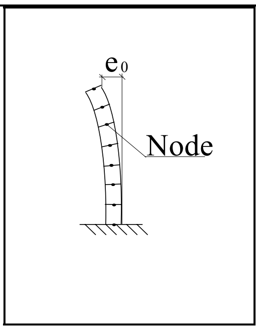
6.2 Eccentricities

Eccentricities as initial out of straightness and thermal bending will vary along the length of the column. These eccentricities influence the behaviour of fire exposed columns because it will implicate a moment due to the axial load in accordance with equation 5-5.

$$M = N \cdot e \qquad \text{(eq. 5-5)}$$

The columns were modelled structurally with a geometrical eccentricity in the nodes to compensate for initial out of straightness, as indicated in figure 5-3. The initial out of straightness is modelled according to the first eigenmode with the maximum deflection taken as the system length divided by 400.

The difference in temperature over the cross-section implies a varying desire to expand thermally. The Bernoulli assumption states that the cross-section plane remains perpendicular to the beam axis during the progress of deformation. It is thus concluded that the beam axis must bend and a thermal eccentricity is obtained. The stress contribution from the moment will be superposed to the compression stress generated by the axial force. During the progress of the fire scenario the temperature gradient grows larger and the ”thermal eccentricity” is increasing continuously, with escalating stresses as a result, and finally the critical design strength will be attained. This is the reason why partial fire exposure is so critical to a column’s load-bearing capacity and thus needs to be investigated thoroughly in order to determine the most appropriate approach of design. Presuming a structural model as showed in figure 5-3 the total thermal eccentricity, e_{th}, is indicated.



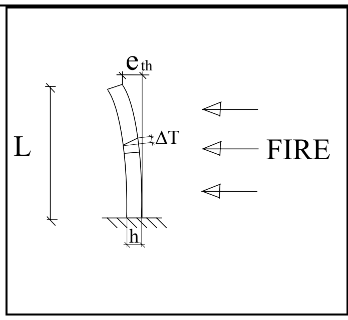


Figure 5.3 *Structural model of studied columns. Lower end fixed, upper end free. Initial eccentricity modelled in the nodes.*

Indication of thermal eccentricity, ε_{th}, descending from the temperature difference between the flanges, ΔT, caused by partial fire exposure.

6.3 Degree of Loading

The degree of loading is taken as a percentage of the characteristic buckling resistance load in the fire state, represented by the expression defined in equation 5-4. Appropriate magnitudes of relative loading were found to be 25%, 40%, 50% and 60%. Supplementary analyses were done with a relative loading between 0-80% (see table 5-1).

Slenderness λ	~0	0.5	0.75	1.0	1.5
Column length	(0.3 m)	2.37 m	3.56 m	4.74 m	7.11 m
0 % load	-	-	-	0 MN	-
25 % load	1.39 MN	1.23 MN	1.03 MN	0.80 MN	0.45 MN
40 % load	2.23 MN	1.96 MN	1.66 MN	1.28 MN	0.71 MN
50 % load	2.78 MN	2.45 MN	2.07 MN	1.60 MN	0.89 MN
60 % load	3.34 MN	2.95 MN	2.48 MN	1.92 MN	1.06 MN
70 % load	-	-	-	2.24 MN	-
80 % load	-	-	-	2.56 MN	-

Table 5-1 *Applied degrees of loading for HEB 300 column and steel with a characteristic strength of 390 MPa.*

7 Thermal Analysis

The information about thermal properties, boundary conditions and fire exposure are essential when calculating nodal temperatures as function of time during a specified fire scenario:

The key engineering tool in this analytical procedure is the finite element temperature calculation program, Super-Tempcalc /6/, which facilitates calculations of heat transfer, temperature redistribution and temperature development in modelled materials. The features and background theory of Super-Tempcalc are described in detail in section 6.1.

The difference between the four cases is the modelling technique in the temperature simulations. figure 6.1 illustrates how ~1/4, 1/2, ~3/4 and 1/1 exposure are being modelled in the temperature calculation.

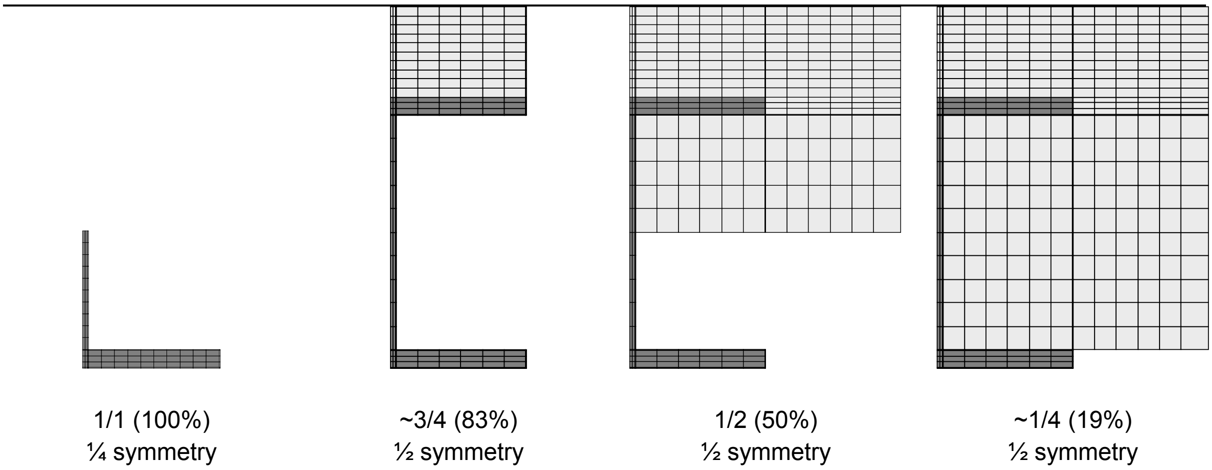


Figure 6.1 *Modelling of the four fire exposure degrees (insulation alternatives). Exposed steel is insulated with intumescent paint.*

7.1 Super-Tempcalc

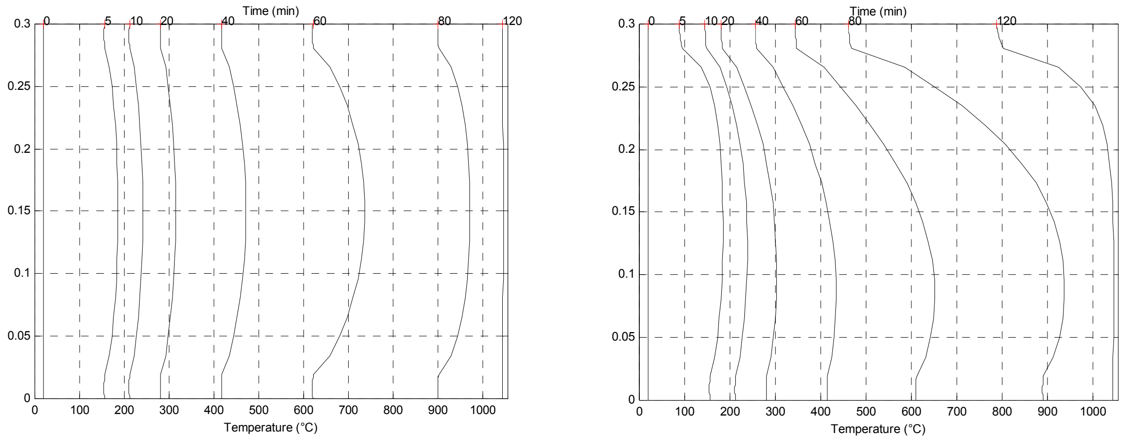
Super-Tempcalc /6/ is a fire-adapted two-dimensional finite element program developed by FSD for use on personal and mainframe computers. It is a further development of Tempcalc, originally developed in 1985.

The program is widely used in the field of passive fire protection, and as part of structural analysis, in buildings and on offshore platforms. It is accepted for North Sea applications by a number of countries and organisations.

The program solves the two-dimensional, transient, heat transfer differential equation.incorporating thermal properties that vary with temperature. The program allows the use of rectangular or triangular finite elements, in cylindrical or rectangular co-ordinates. Heat transferred by convection and radiation at the boundaries can be modelled as a function of time. Structures comprising several materials can be analysed and the heat absorbed by any existing void in the structure is also taken into account. Falling off of boards attached to structural members and spalling of concrete of concrete-based material can be simulated. When the geometry is changed the calculation stops and the new geometry is automatically updated and the calculation continues.

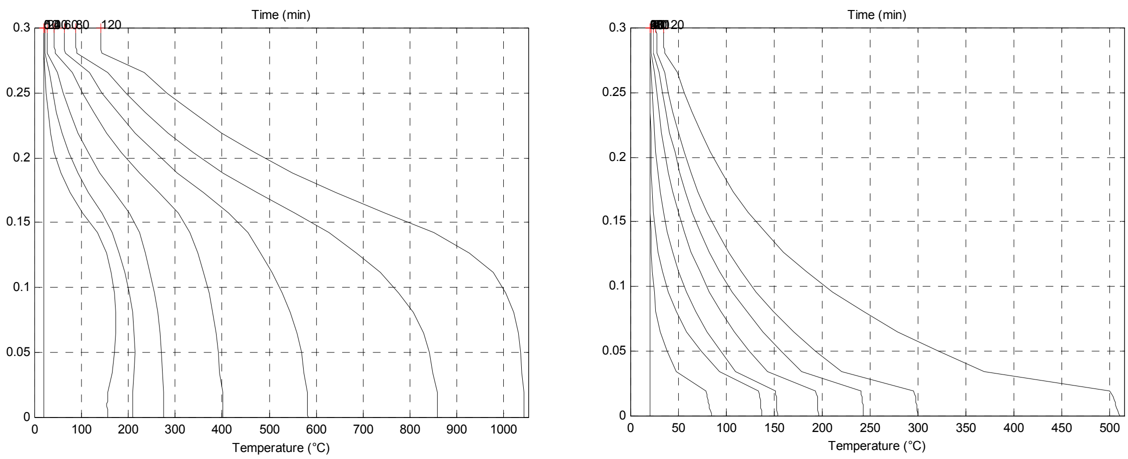
7.2 Results from Thermal Analyses

The steel profile has been subjected to a heat transfer calculation for a given fire scenario, resulting in a cross-sectional temperature gradient over time. The temperature development of the profile denotes an essential input data for the subsequent global collapse analysis. Gradients over the HEB 300 profile height can be found for the four different exposure alternatives in figures 6.2 – 6.3.



a) b)

Figure 6.2 *a) 100 % and b) 83 % fire exposure. Temperature gradient along the height of the HEB300 profile after 0, 5, 10, 20 40, 60, 80 and 120 minutes.*



a) b)

Figure 6.3 *a) 50 % and b) 19 % fire exposure. Temperature gradient along the height of the HEB 300 profile after 0, 5, 10, 20 40, 60, 80 and 120 minutes.*

8 Software for Structural Analysis

The results of the thermal analysis are incorporated in the structural ditto for computerised prediction of structural stability. Elevating temperatures affect the steel by reduced strength and modulus of elasticity, stress-strain relationship and the ability to expand thermally.

The main tool for current analyses has been the program Global Collapse Analysis. Comparisons of tests have been done with the software Fire Design, the beam and column tools integrated in TCD/7/. Columns differ significantly from other structures, being subjected to effects from instability. This means that failure generally occurs from the axial load reaching the critical load (equation 4-5), and not from the cross-sectional strain attaining the yield limit. The primary task of a column is generally to transfer loads vertically down to the foundation, i.e. columns in structures are carrying axial loads. This makes the column a very essential part of a structure.

8.1 Global Collapse Analysis (GCA)

In some elastic solid mechanics problems the governing differential equations are linear and with a linear form of stress-strain relationship. However, in fire-related structural problems the linearity of constitutive relations is not preserved. The problem is actually a combination of material non-linearity together with geometrical non-linearity.

Global Collapse Analysis (GCA) is a finite element program providing computer prediction of the structural behaviour of load-bearing systems.

Essential input to the program comprise:

- cross-sectional geometry
- cross-sectional time-temperature fields from thermal analysis
- geometry (column and beam geometry)
- boundary conditions (rigidity, external attachments)
- material data (steel strength, variation of stress-strain relationship)
- external loads

GCA is integrated with the temperature calculation program Super-Tempcalc, thus incorporating steel temperature field data versus time of the adopted fire scenario.

The overall stability of a structure, due to a local fire, can be analysed and global progressive and local collapse respectively, can be predicted. Upgrading of identified separate critical members provides possible extension of initially calculated fire resistance time.

Generation of restraint stresses and strains, due to rigid external connections combined with the thermal expansion in the steel, makes the model reasonably accurate.

Results in terms of stresses, strains, cross-sectional stiffness, deflections, displacements, forces and moments may be presented at selected times through out the fire scenario. The 2-dimensional Bernoulli beam element with 6 degrees of freedom is used in the finite element model. The kinematic assumption of this element is that plane sections normal to the beam axis remain plane and normal to the beam axis during the deformation.

8.2 TCD - FIRE DESIGN

FIRE DESIGN is a set of structural fire design tools that are interfaced with SUPER-TEMPCALC in the application TCD (Temperature Calculation and Design) /7/. The tools are SBEAM, CBEAM and COMPRES. The governing equations and the background theory for the calculations undertaken in FIRE DESIGN are outlined in this section. Output from SUPER TEMPCALC provides the essential heat transfer data upon which the FIRE DESIGN calculations are based. The design principals follow the intentions in the Eurocode documents ENV 1991-1-2 /4/, ENV 1992-1-2, ENV 1993-1-2 /2/ and ENV 1994-1-2 /3/.

8.2.1 SBEAM

For beams and slabs no second order geometry effects apply and thus the load bearing capacity (moment capacity) of a member can be calculated solely by studying the cross-section and its temperatures and strength relations.

SBEAM calculates the moment capacity of fire exposed structural steel beams in the ultimate limit state. Steel is an isotropic material with equal tensile and compressive properties. Hence the plastic sagging bending moment capacity of a beam is calculated based on the tensile capacity at elevated temperatures in the lower part of the beam cross-section and similarly the compressive capacity in the upper part.

8.2.2 COMPRES

COMPRES calculates the plastic yield compression resistance, critical Euler buckling load and design load of fire exposed structural steel compression members in the fire limit state in accordance with the guidelines in Eurocode 3 and 4 /2, 3/. Each individual material is accounted for by considering its contribution to the overall strength and stiffness of the composite structure.

9 Structural Analysis

9.1 Structural Behaviour for Partially Exposed Columns

The behaviour and collapse mode of partially exposed columns have been studied in 80 computer simulations with centric loading. In the simulations of no eccentricity, the combinations of percent exposed area, slenderness and axial load are presented in tables 8.1 – 8.5. In these tables the times at collapse obtained from the GCA-analysis are presented.

The fire exposure was 120 minutes ISO 834. Modelled steel profile was HEB 300 that was insulated with 0.93 mm of intumescent paint in order to extend the overall time to collapse for the purpose. Structural steel with a yield strength of 390 MPa was chosen. The degree of loading in the fire limit state is expressed as a percentage of the characteristic buckling compression resistance load at room temperature.

From the figures 8.1-8.4 it can be observed that minimum time for collapse occurs at about 50% of partial exposure with a slenderness exceeding 0.5. The reason for this exception is that virtually no buckling occurs but only axial compressive stresses arise. However, the thermal gradient causes thermal bending and an extra eccentricity for the axial load. Due to that the horizontal deformation and the eccentricity increases when exposed area decreases from 100 % to 50 %. This means that partial exposure causes much earlier collapse of the column compared with full exposure. The fire resistance is furthermore decreasing with increasing load. These figures can be used in the design or redesign of partially fire-exposed slender columns.

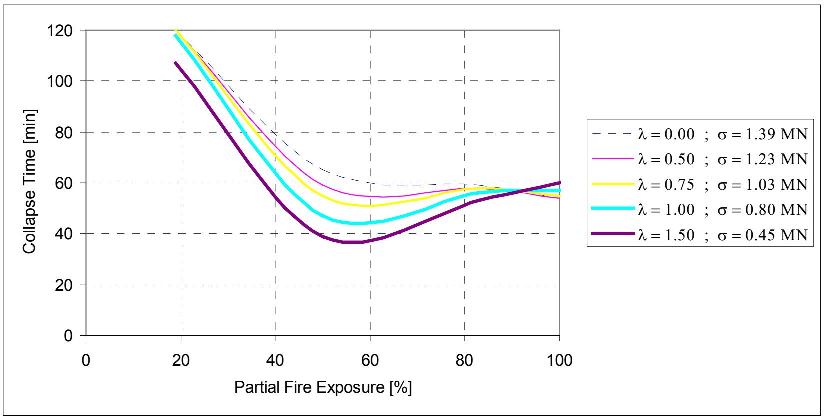


Figure 8.1 *Calculated column fire resistance times with a degree of loading of 25%.*

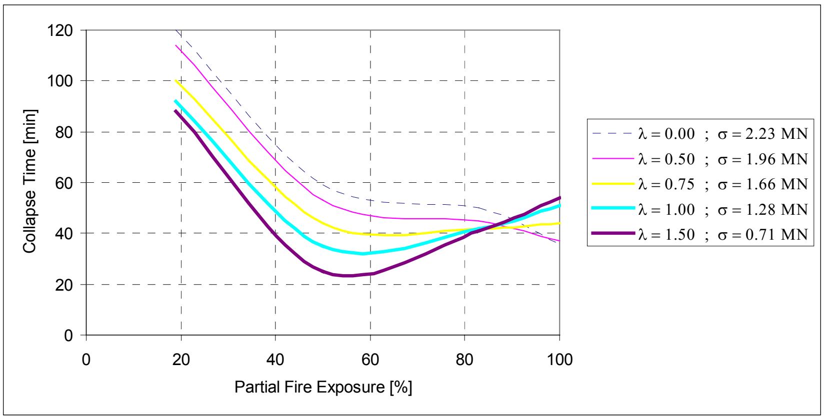


Figure 8.2 *Calculated column fire resistance times with a degree of loading of 40%.*

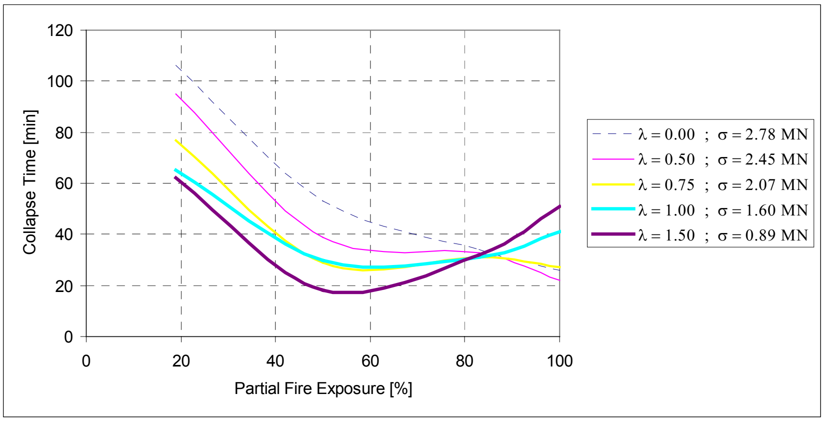


Figure 8.3 *Calculated column fire resistance times with a degree of loading of 50%.*

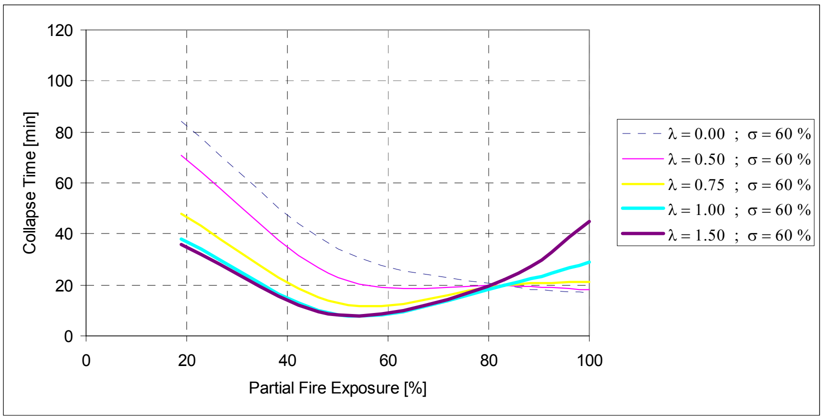


Figure 8.4 *Calculated column fire resistance times with a degree of loading of 60%.*

9.2 Fire tests and computer simulations – A comparison

9.2.1 Computer Simulation of Steel Column Fire Test Conducted in UK

A full scale fire test on a steel column exposed to fire on 100 % of the profile circumference that was tested at Firto Borehamwood, U.K. in 1984 /9/ has been simulated (see Fig 8.5). The test will be referred to as fire test 41. The column has an exposed length of 3 m. The temperature as function of time over the cross-section has been calculated by Super-Tempcalc. The fire exposure curve has been taken from documented readings during the tests and has been used in the calculations. The temperature for the 100 % exposure is presented in figure 8.6 a where measured and computed temperatures are compared for the exposed web and unexposed flange respectively. The agreement between calculated and measured temperatures is considered adequate.

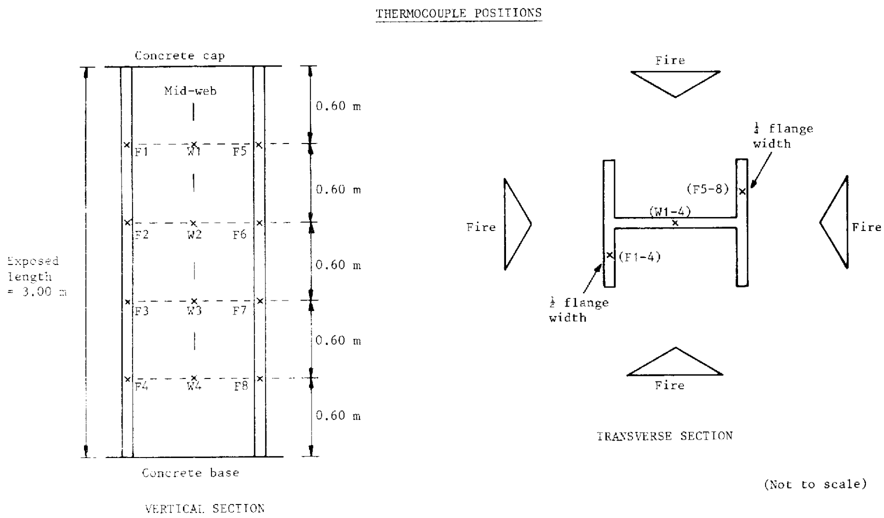


Figure 8.5 *Description of fire test 41 on steel column (h*w*t_w*t_f =206x204x8x12.5 mm) fully fire exposed /9/*

The tested steel column has an effective length of 2.1 m (fully exposed). The slenderness ratio is 0.45 (weak axis) and 0.24 (strong axis) respectively i.e. a short column with almost no buckling influence. The deformation behaviour and the collapse time for the fire tested column (100% fire exposure and 60% of allowed maximum load according to test standard) has been predicted by Global Collapse Analysis and compared with measured results.

The measured value of steel strength was 349 MPa, to be compared with the design value of 255 MPa. The actual degree of loading is therefore less than the allowed 60%. The actual, measured value of strength was used in the simulations. The modulus of elasticity modulus was set to 210 GPa since it was not measured during the test.

In figure 8.6b the predicted and measured axial deformation as function of time is compared. The concordance in deformation process is good and the collapse time from GCA is predicted to 19 minutes compared to the measured 23 minutes. Due to all testing uncertainties and the normal variation of results (sometimes up to 50% for columns) from fire testing on identical specimens this difference is quite acceptable.

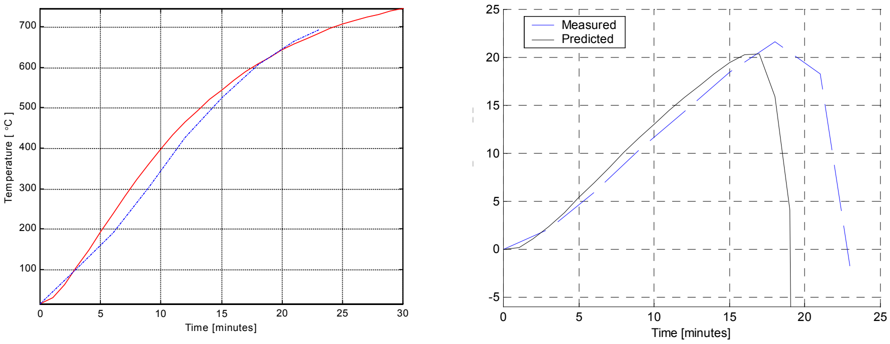


Figure 8.6 a) b)
Comparison of predicted and measured a) temperature and b) axial deformations of fire exposed steel column ($w*h*t_w*t_f$ =204x206x12.5x8 mm) for fire test 41 under 100 % fire exposure of column /1/. (Dotted line = measured, solid line = predicted)

A comparison was done with the column analysed using the tool COMPRES. With a measured initial value of steel strength the resistance time was determined to 21.7 minutes. This value is almost 15 % greater than the GCA-calculation and ought to be less than 19 min because it is a simplified mthod copared with an "accurate" method.

9.2.2 Computer Simulation of Steel Beam Fire Test Conducted in UK

A full scale fire test on a simply supported steel beam exposed to ISO fire on three sides and with a concrete slab cast onto the upper flange that was tested at Firto Borehamwood, U.K. in 1984 /9/ has been simulated (see fig 8.7). This test will be referred to as fire test 11. The length of the beam was 4.5 m.

The standard ISO 834 gas temperature curve was used to simulate the fire test. The temperature as function of time over the cross-section has been calculated by Super-Tempcalc. The temperatures at top flange, web and exposed bottom flange were applied to the structural analysis.

The structural resistance has been calculated using both the non-linear approach of GCA and the Eurocode 3 design method implemented in SBEAM. This was done in order to get an indication of whether the Eurocode 3 method, which does not include the effect of added thermal moment due to temperature gradient, can be considered relevant.

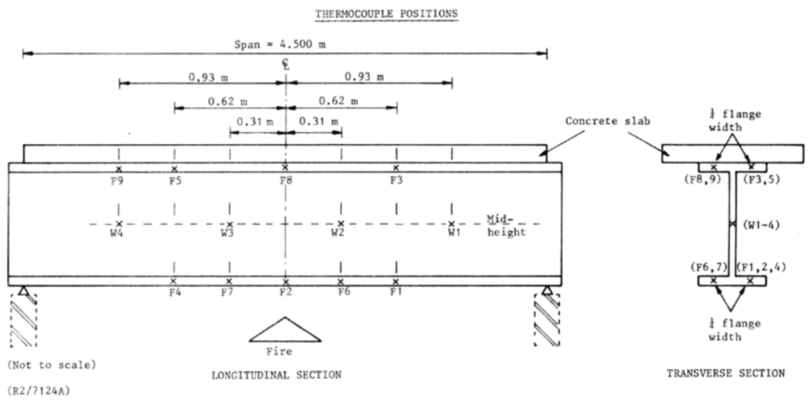
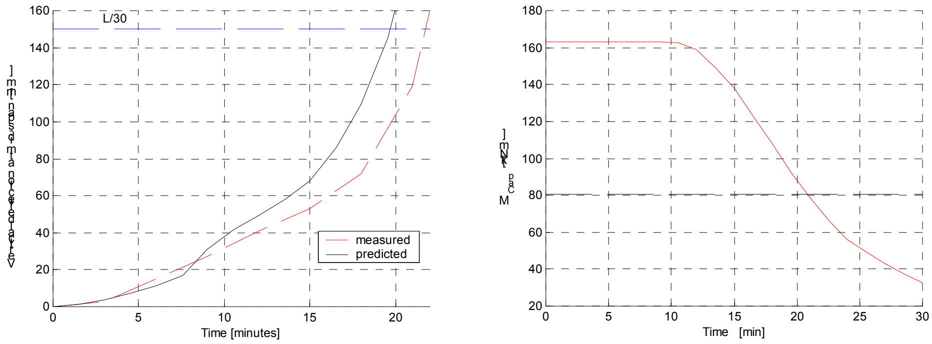


Figure 8.7 *Description of fire test 11 on steel beam $h*w*t_w*t_f$ = 256x145x7.47x12.79 mm. /9/*

A maximum allowed deflection of L/30 was used as stability criteria. With a span of 4.5 m this means a maximum deflection of 150 mm. A comparison of measured deflections and deflections as calculated with GCA is presented in figure 8.8a. The fire resistance of the tested beam was 21.7 minutes and corresponding GCA calculation resulted in 19.6 minutes of fire resistance.



a) b)

Figure 8.9 a) *Deflections at centre of beam. The collapse time of the GCA simulation was 19.6 minutes. The resistance time of the test was 21.7 minutes./1/*

b) *Moment capacity as calculated using SBEAM. The fire resistance time was 20.8 minutes, to be compared with 21.7 minutes of the test./1/*

A comparison to design with the tool SBEAM is shown in figure 8.9b. The SBEAM calculation, that does not consider deflections, resulted in 20.8 minutes of structural stability during fire. Compared with the GCA analysis this value is too advantageous for a simplified method depending on the use of design strength based on 2% strain and plastic theory. This corresponds to the time when the load bearing capacity reached the value of the load effect of 80 kNm.

The deviation between measured results and simulations is too small to conclude that the effect of the thermal moment would be significant for beams in general.

10 References

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