

Temperature distribution in a full-scale steel framed building subject to a natural fire

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Abstract. Current fire design codes for determining the temperature within the structural elements that form part of a complete building are based on isolated member tests subjected to the standard fire. However, the standard time-temperature response bears little relation to real fires and doesn't include the effects of differing ventilation conditions or the influence of the thermal properties of compartment linings. The degree to which temperature uniformity is present in real compartments is not addressed and direct flame impingement may also have an influence, which is not considered. It is clear that the complex thermal environmental that occurs within a real building subject to a natural fire can only be addressed using realistic full-scale tests. To study global structural and thermal behaviour, a research project was conducted on the eight storey steel frame building at the Building Research Establishment's Cardington laboratory. The fire compartment was 11 m long by 7 m wide. A fire load of 40 kg/m² was applied together with 100% of the

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permanent actions and variable permanent actions and 56% of live actions. This paper summarises the experimental programme and presents the time-temperature development in the fire compartment and in the main supporting structural elements. Comparisons are also made between the test results and the temperatures predicted by the structural fire Eurocodes.

Keywords: fire design; full-scale tests; steel and composite structures; temperature.

1. Introduction

It has long been recognised that global frame behaviour differs from an assessment based upon the performance of the individual elements, which go to make up a structure. The experience gained from investigations following the catastrophic gas explosion at *Ronan Point* which led to a progressive structural collapse highlighted the need for the engineer to consider global behaviour which, in this instance, conducted to a failure mechanism not considered at the design stage. Subsequent robustness requirements have led to improvements in the design and construction of framed structures. Just as a consideration of overall building behaviour can allow to previously unconsidered modes of collapse so such a philosophy may reveal beneficial aspects of frame behaviour. As well as potential disasters to be avoided there may be potential advantages to be utilised. Alternative methods of sustaining the applied loading may be available. Attempts to demonstrate the enhanced performance available through frame continuity were made as far back as the 1930's (Steel Structures Research Committee). Moore provided a comprehensive justification for testing at full-scale (Moore *et al.* 1993). The principles of assessing the structural performance of individual members when subject to realistic loading regimes and realistic boundary conditions are particularly relevant when considering the fire resistance of a framed structure.

The development of the Large Building Test Facility at the Building Research Establishment's (BRE) Cardington Laboratory provided the construction industry with a unique opportunity to carry out full-scale fire tests on a complete steel framed building designed and built to UK practice but in such a way that it satisfied the requirements of Eurocode 3. Consequently, between the 1995 and early 2003 a series of seven large scale fire tests were conducted on a full-scale steel framed building at Cardington. This paper describes the last of these fire tests and presents the measured temperatures within the compartment, through the main supporting steel and composite floor and the temperature distribution in each of the main beam-to-column and beam-to-beam connections.

1.1. The test facility

To meet the needs for the future, the BRE created the Large Building Test Facility within one of the airship hangar's at Cardington, south of Bedford in the UK. The hangar is approximately 260 m long, 80 m wide and 50 m high and contains a 70 m by 50 m strong floor at one end. This facility can accommodate full-sized buildings up to ten storeys high within a weatherproof envelope.

The opportunities for testing and assessment of methodologies, techniques and materials for buildings and structures erected in the facility are limited only by the closed environment and the unique foundation and as always by the imagination of those undertaking the work. Physical tests involving static loads, dynamic vibrations, fire, explosion, heat and water can all be used in simulations of a wide range of realistic hazard scenarios.

The facility currently contains three large experimental buildings (Moore 1995). These are a six storey timber structure, a seven storey concrete structure and an eight storey steel building.

1.2. The test structure

The first structure to be erected within the LBTF was an eight-storey steel framed building. This building was designed and constructed to resemble a typical modern city-centre, eight-storey office block. The building covers an area of 21 m by 45 m, with an overall height of 33 m. It consists of five 9 m bays along the length of the building and across the width there are three bays spaced at 6 m, 9 m and 6 m. The building has three lift-shafts, one in the centre of the building and two placed at each end. The structure was designed as a braced frame with lateral restraint provided by cross-bracing around the three vertical access shafts. The beams were designed as simply supported acting compositely (via shear studs) with the lightweight composite floor slab. The floor slab is 130 mm deep and consists of a steel trapezoidal deck with lightweight concrete and an A142 steel anti-crack mesh.

The connections were designed and detailed to the BCSA/SCI - Greenbook, *Joints in Simple Construction*. Fin plates were chosen for most of the beam-to-beam connections. In most cases, this meant that the secondary beams were simply sawn, drilled and notched. Flexible end-plates were adopted for the main beam-to-column connections. These provided a little more rigidity to the steel frame during erection.

Throughout the structural design the underlying philosophy was to obtain a structure that was buildable and at all stages of construction and erection reflected normal building practice in the UK rather than specialist research procedures. The building was designed for a dead load of 3.65 kN/m² and an imposed load of 3.5 kN/m².

2. The fire test

The structural integrity fire test (large test n^o.7) was carried out in a centrally located compartment of the building, enclosing a plan area of 11 m by 7 m on the 4th floor (Wald *et al.* 2003). The preparatory works took four months. The fire compartment was bounded with walls made of three layers of plasterboard (15 mm + 12.5 mm + 15 mm) with a thermal conductivity of between 0.19 - 0.24 Wm⁻¹K⁻¹. In the external wall the plasterboard is fixed to a 0.9 m high brick wall. The opening of 1.27 m high and 9 m length simulated an open window to ventilate the compartment and allowed the observation of the element behaviour. The ventilation condition was chosen to produce a fire of the required severity in terms of maximum temperature and overall duration. The columns, external joints and connected beam (about 1.0 m from the joints) were fire protected to prevent global structural instability. A fire protection of 18 - 22 mm of Cafco300 vermiculite-cement spray, with a thermal conductivity of 0.078 Wm⁻¹K⁻¹ was used.

The steel exposed structure consists of two secondary beams (section 305x165x40UB, steel S275 measured $f_y = 303$ MPa; $f_u = 469$ MPa), an edge beam and two primary beams (section 356x171x51UB, steel S350 measured $f_y = 396$ MPa; $f_u = 544$ MPa) and four columns (internal column sections: 305x305x198UC and external column sections: 305x305x137UC, steel S350) (Bravery 1993). Flexible end-plates (also called header plates) were used for the beam-to-column connections and fin-plates were used for the beam-to-beam connections. In both cases S275 steel and M20, grade 8.8 bolts were used. Composite behaviour was achieved by using 19 mm diameter shear studs ($f_u = 350$ MPa) to

connect the primary and secondary to the light-weight concrete and profiled metal deck composite floor slab. The geometry and measured material properties of the flooring system are summarised by Wald *et al.* (2003).

The applied load was simulated using 1 100 kg sandbags applied over an area of 18 m by 10.5 m on the floor immediately above the fire compartment. The sandbags represent 100% of the permanent actions, 100% of variable permanent actions and 56% of live actions. The applied load was designed to fail the floor, based on analytical and FE simulations. To provide a fire load of 40 kg/m², wooden cribs with moisture content 14% were used.

2.1. Instrumentation

The instrumentation included thermocouples, strain gauges and displacement transducers. A total of 133 thermocouples were used to monitor the temperature of the connections, steel beams, slab and gas compartment, see Fig. 1. An additional 14 thermocouples were used to measure the temperature of the protected columns.

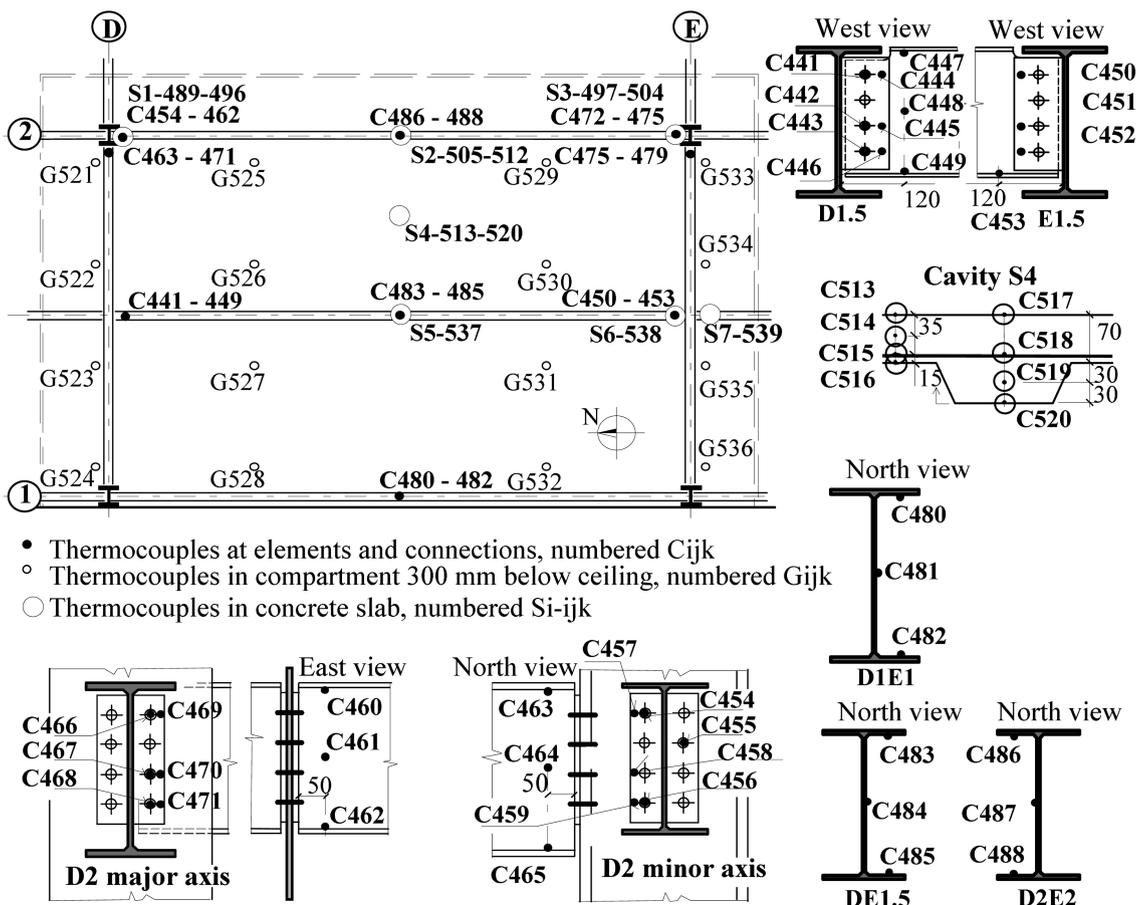


Fig. 1 Location of thermocouples in the compartment below the ceiling and on steel structure

High and ambient temperature strain gauges were used to measure the strain in the elements. In the exposed and un-protected elements (fin plate and end plate - minor axis), 9 high temperature strain gauges were used. In the protected columns and on the slab a total of 47 ambient strain gauges were installed.

25 displacement transducers were attached along the 5th floor to measure the vertical deformation of the concrete slab. An additional 12 transducers were used to measure the horizontal movement of the columns and the slab. 10 video cameras and 2 thermal-imaging cameras recorded the fire and smoke development, the deformations and the temperature distribution (Wald *et al.* 2003).

3. Fire development and compartment temperature

The quantity of fuel and the dimensions of the opening in the facade wall were designed to achieve a representative fire in an office building. Fig. 2 shows the measured time-temperature curve within the compartment. In the initial stage of the fire, the temperature within the compartment grows rapidly to reach a maximum temperature of 1107.8 °C after about 54 min. The maximum recorded compartment temperature occurred near the internal wall (2 250 mm from D2) of the compartment. Fig. 2 also compares these values with the temperatures predicted by the parametric curve given in EN 1991-1-2: 2004. The parametric curve predicts a maximum temperature of 1078 °C after 53 min, which is a good agreement with the test results; see (Wald *et al.* 2004b). During the heating phase the isotherms shown in Fig. 3(c) indicate that the maximum temperatures were reached towards the back of the compartment.

The measured maximum gas temperatures are summarised in Table A1. The average gas temperature is taken from all sixteen thermocouples within the compartment.

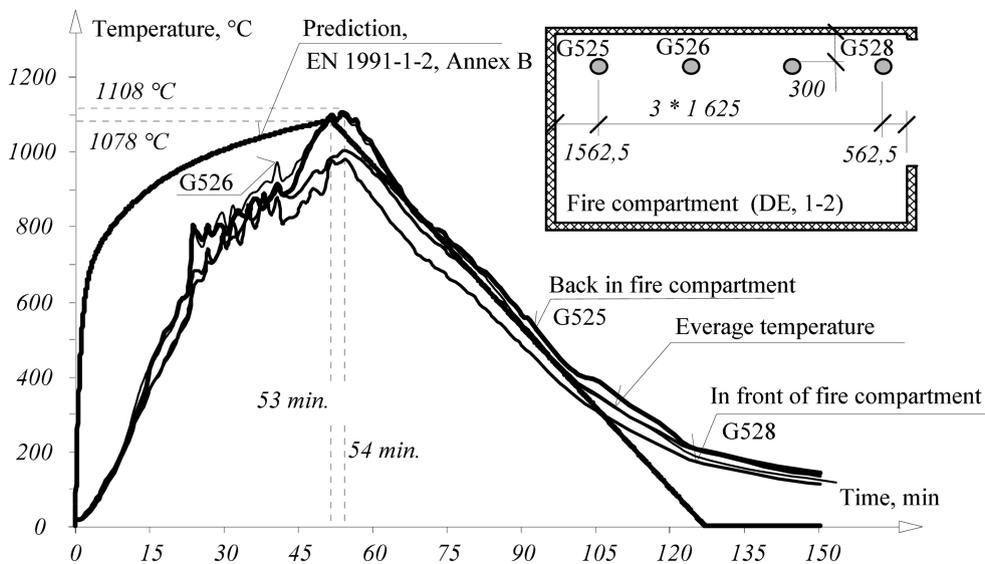


Fig. 2 Comparison between measured and predicted gas temperature

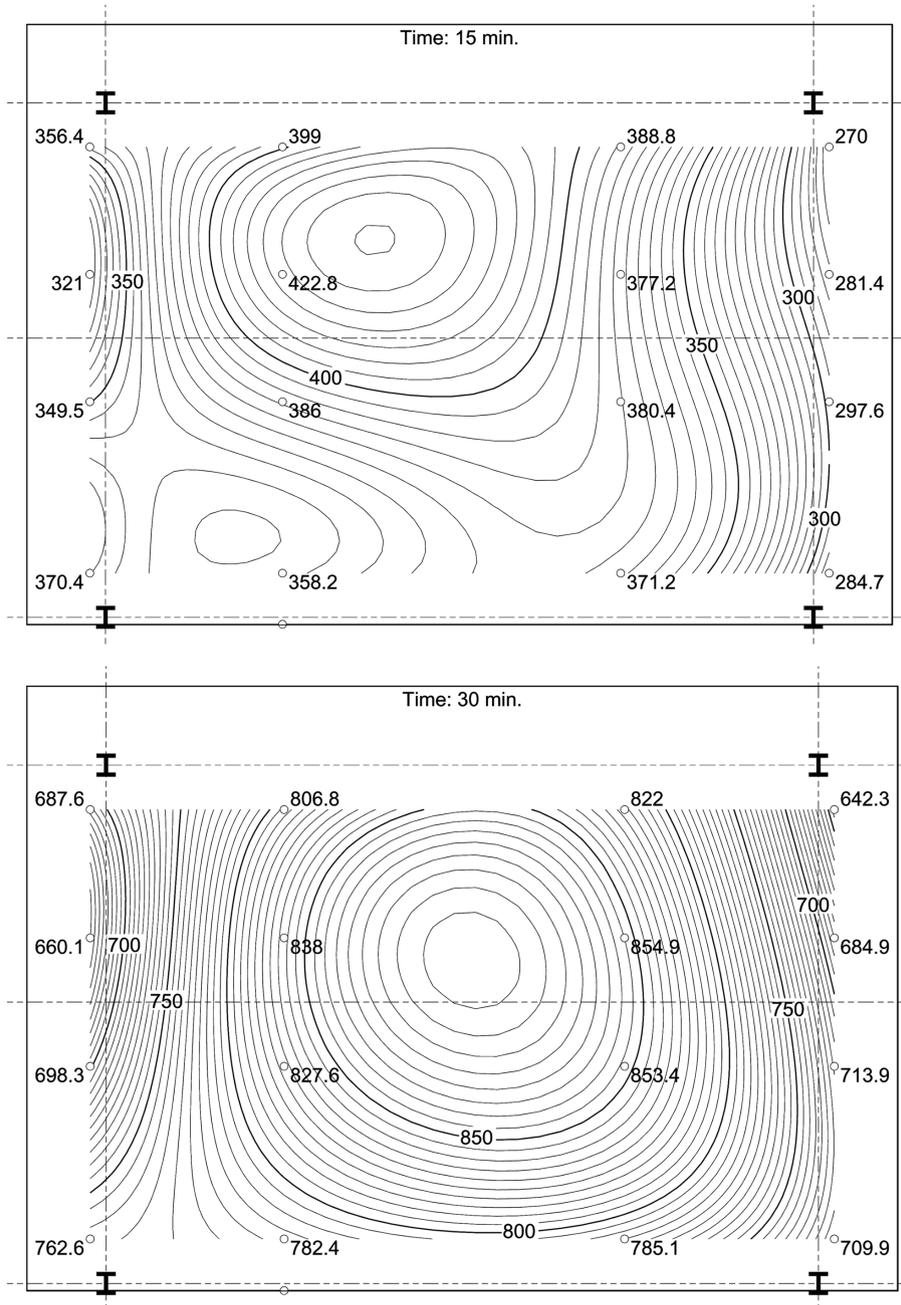
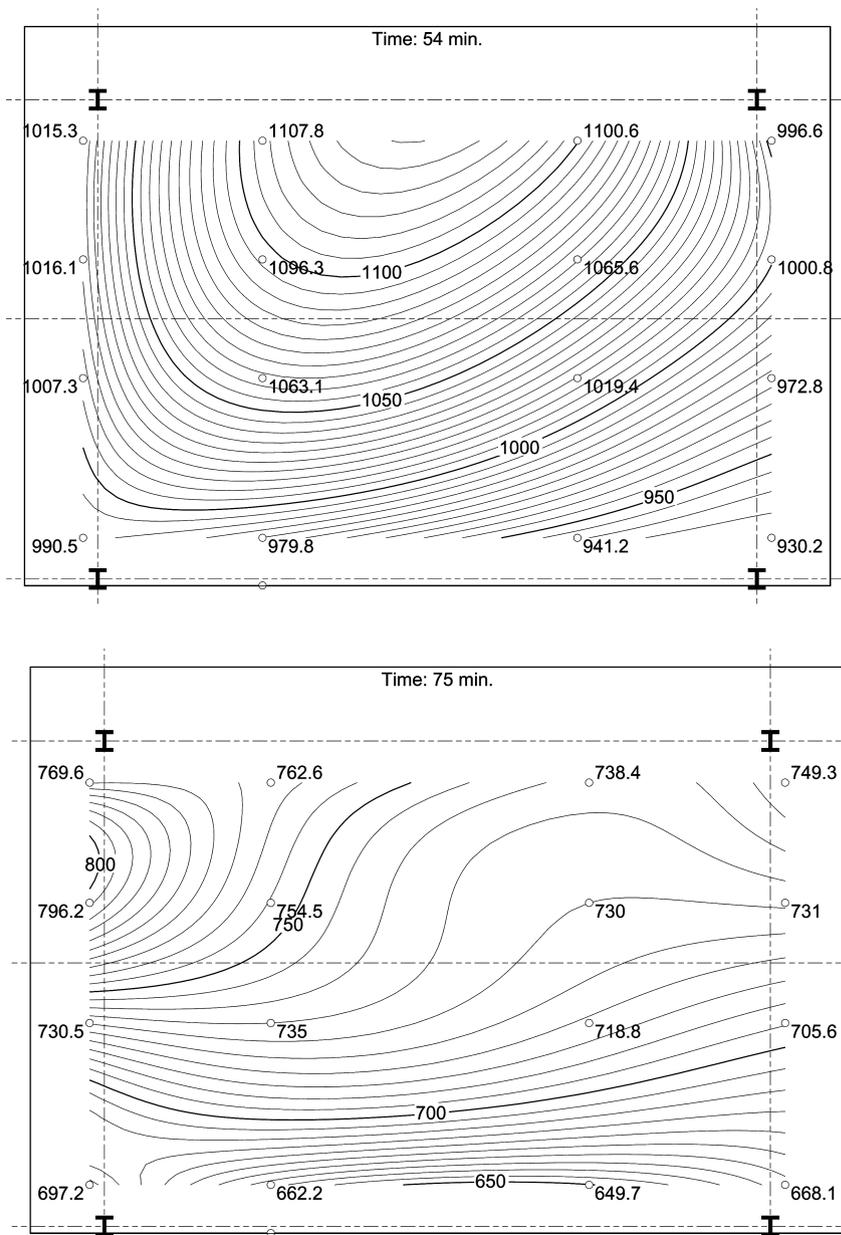


Fig. 3 Isotherms of compartment temperatures

4. Beam temperatures

Measurements of the temperature in the mid-span beams were taken in the bottom flanges, web and upper flange. A summary of the temperatures recorded in the mid-span of the beams is given in Fig. 4.



Note: The thermocouples were located approximately 300 mm below the ceiling. The temperatures given in each of the figures represents the maximum temperature achieved between T and T-5mins where T is the time.

Fig. 3 Continued

The maximum recorded steel temperature of 1087.5°C occurred after 57 minutes in the bottom flange at the mid-span beam DE2 (see the results for thermocouple C488, Table A2).

By using an iterative calculation procedure for the transfer of heat into the unprotected steel structure (see expression 4.25 and B1 in EN 1993-1-2: 2004), it is possible to predict that a maximum steel

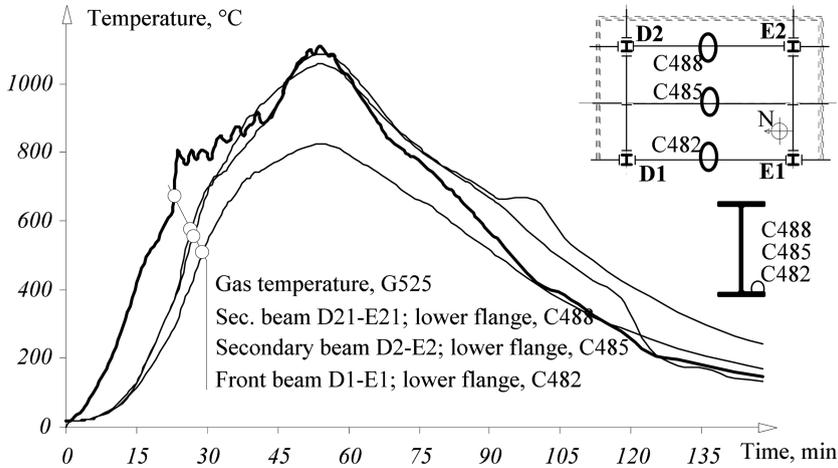


Fig. 4 Temperature variation within the beams D1-E1; D1.2-E1.2. D2-E2

temperature of 1067°C is reached after 54 min. This compares well with the measured data. The temperature of the beam’s flanges and web can also be calculated by using clause 4.3.4.2.2 of EN 1994-1-2: 2004 (see Buchanan 2003). The values given in Figs. 3 and 4 are calculated based on measured gas temperature in thermocouple G525. The shadow effect is not taken into account.

Fig. 5 compares the measured temperatures in the beam lower flange with a calculation procedure based on Eq. (B1) with a section factor for unprotected steel members $A_m / V = 208 \text{ m}^{-1}$ exposed on three sides. An alternative calculation procedure based on the mass of plates according to EN 1994-1-2: 2004 is shown in Fig. 6. The figure relate to a constant value for the specific heat of steel and a value varying with temperature according to 3.4.1.2 of EN 1993-1-2:2004. A constant value for specific heat provides an acceptable but conservative solution compared to the measured data.

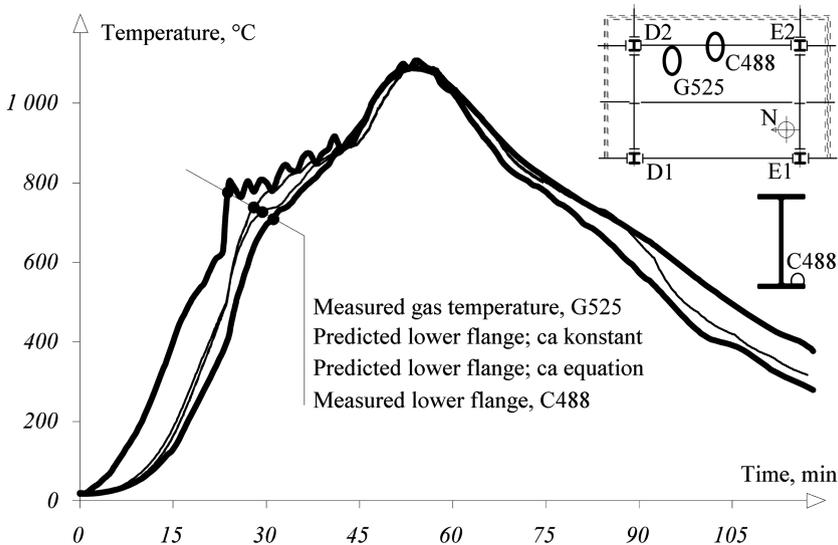


Fig. 5 Prediction of beam lower flange temperature. thermocouple C488

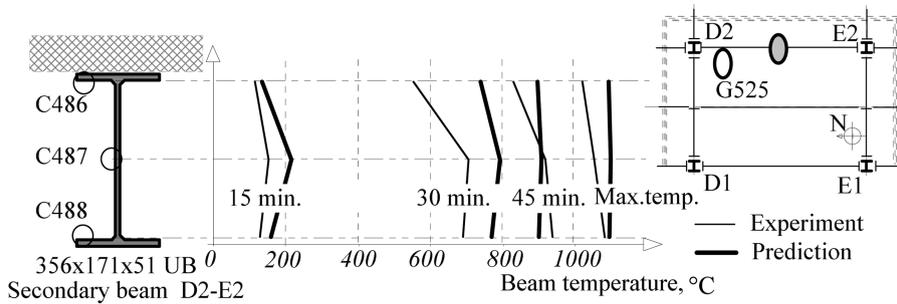


Fig. 6 Comparison between measured and predicted beam temperature (gas temperature: thermocouple G525)

The method in 4.3.4.2.2 of EN 1994-1-2:2004 is applied for Fig. 6. The web temperature was calculated separately. The bottom flange section factor is based on a four sided exposure and the top flange a three sided exposure (provided at least 85% of the top flange is in contact with the slab or the voids are filled).

5. Column temperatures

The temperatures of the columns were measured at three sections - at mid height, 500 mm from the floor, and 500 mm below the ceiling. At each section, measurements on both flanges and on the web were taken. The internal columns were fire protected up to the underside of the primary beam leaving the length of column adjacent to the connection unprotected. Temperatures recorded on columns D1 and D2 are presented in Fig. 7. The maximum recorded temperature in the insulated part of the column was 426°C, which occurred after 106 minutes.

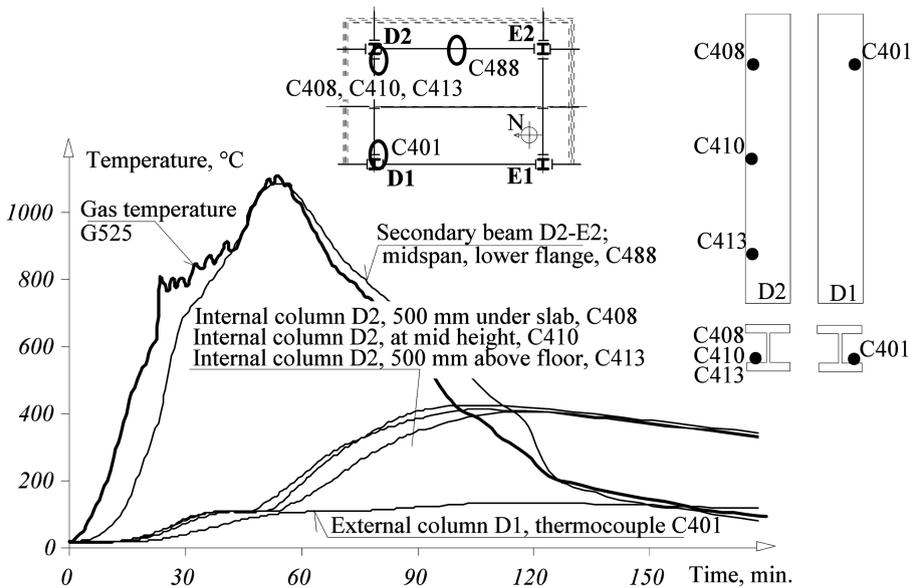


Fig. 7 Comparison between gas and steel temperature

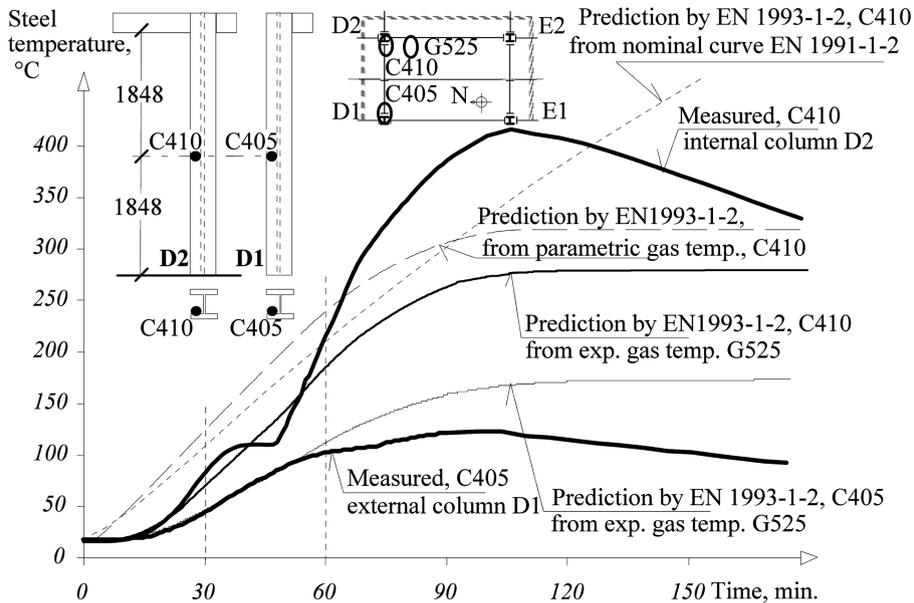


Fig. 8 Comparison between predicted and measured temperature (thermocouple C408)

Once again an iterative heat transfer procedure was used to calculate the temperature of the protected column (see expression 4.27 in EN 1993-1-2: 2004, Eq. (B2)). It was assumed that the fire protection material had a unit mass of $\rho_p = 310 \text{ kg m}^{-3}$; a thickness of $d_p = 0.02 \text{ m}$; a specific heat of $c_p = 1200 \text{ J kg}^{-1} \text{ K}^{-1}$; a thermal conductivity of $\lambda_p = 0.078 \text{ W m}^{-1} \text{ K}^{-1}$ and a moisture contents $p = 12\%$. Fig. 8 compares the predicted and measured temperatures. Three predictions are shown in Fig. 8; they are based on the measured gas temperature in thermocouple G525, on the calculated parametric temperature (Wald *et al.* 2004a) and on the nominal standard fire (ISO 834). All three predictions compare reasonably well during the heating phase; however, during the cooling phase some discrepancies were observed. These discrepancies are caused by the radiation from the compartment walls that is high due to the location of the column in the rear corner of the compartment.

6. Connection temperatures

Measurements of the temperature in the connections were taken within the beams adjacent to the connection, in the plate (end-plate or fin plate) and in the bolts, see Fig. 1. The temperatures recorded in the connections are summarised at Annex A, Table A3-A5, and presented in Fig. 9 for the beam to column minor axes connection D2-E2, in Fig. 10 for the beam to column minor axes connection D2-D1, and in Fig. 11 for the beam to beam fin plate connection D1.2-E1.2.

From the experimental results, it is observed that, in the heating phase, the joint temperature is significantly lower than the remote bottom flange, which is usually the critical element as regards the designation of the limiting temperature of the beam; in contrast, the cooling down in the joints was slower. At the maximum temperature, the joints temperature were around 200°C lower than the limiting

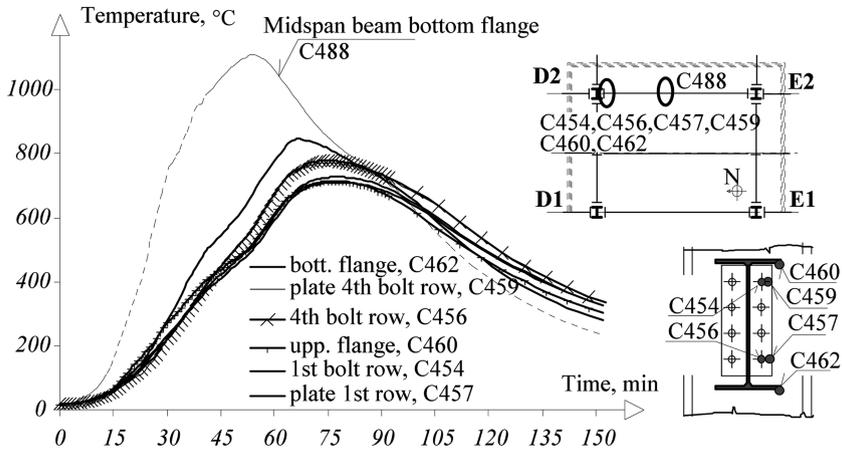


Fig. 9 Temperatures within the beam-to-column minor axes end plate connection D2-E2

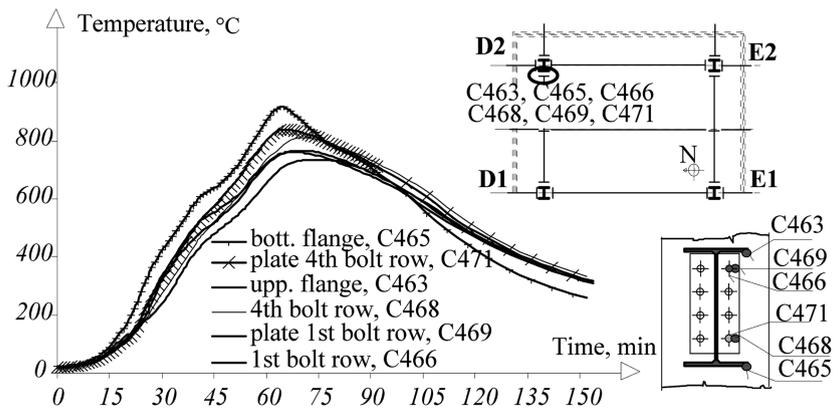


Fig. 10 Temperatures within the beam-to-column major axis end plate connection D2-D1

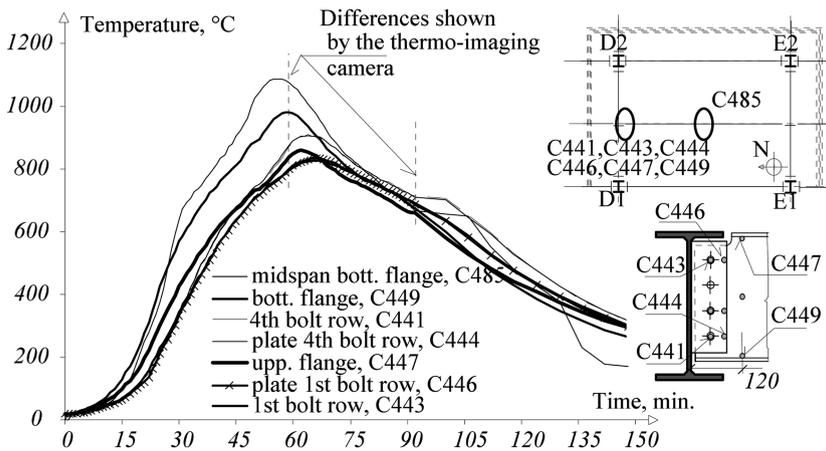
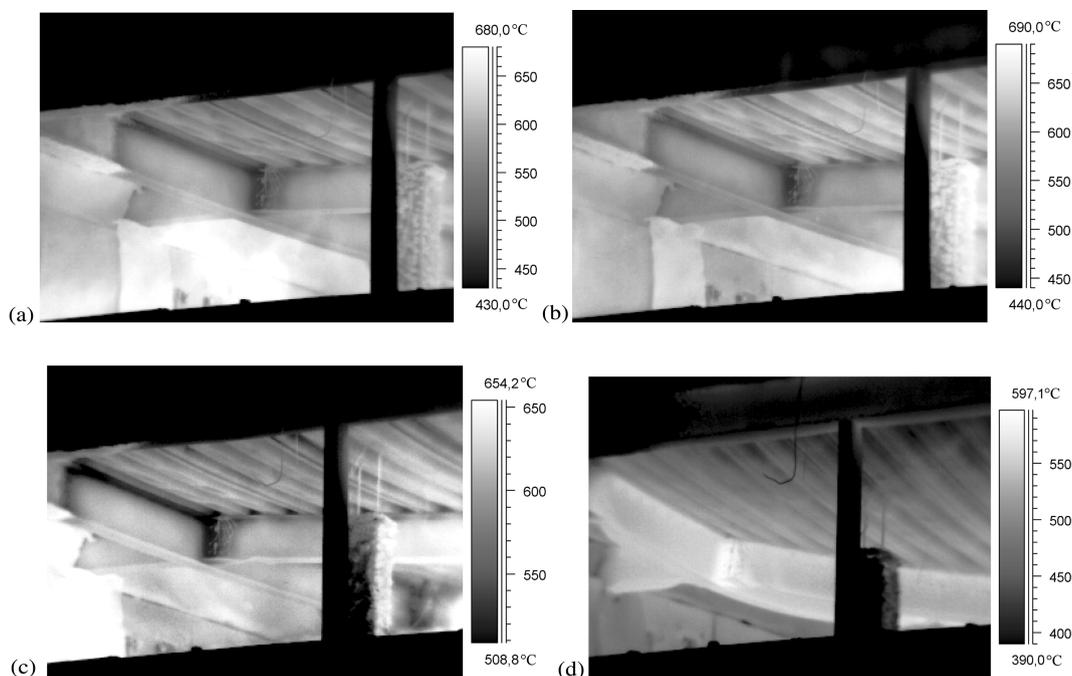


Fig. 11 Temperature at beam-to-beam fin plate connection D1.2-E1.2



Note: Scale of colours on figures is different to visualise contours and temperatures.

Fig. 12 Fin plate connection D1.2-E1.2 recorded by thermo imaging camera (a) after 32 min.; (b) after 33 min.; (c) after 35 min. (local buckling of lower flange is visualised) and (d) during cooling after 92 min

temperature of the beam. Using the thermal-imaging cameras it was possible to observe this effect, see Fig. 12 (Wald *et al.* 2004b). A set of different colours is used to visualise the temperature distribution of the structure. Darker colours represent cooler areas while lighter colours represent hotter areas. In each figure a scale is shown; this scale relates the structures temperatures with different colours. The quality of the images is so good that it is possible to detect the point at which the bottom flange of the secondary beam buckled. This occurred at $t = 32$ min.

At the maximum gas temperature, the temperature of the joints was approximately 200°C lower than the temperature of the beam; see Figs. 7 to 9 and Table B2-B4. For all joints, the temperature of the bolt row closest to the ceiling was cooler than that of the lower rows of bolts because of shielding by the adjacent slab. EN 1993-1-2: 2004 recognises this effect and contains a set of recommendations for calculating the temperature distribution across end-plate connections. The effect of the thermal mass of the floor slab on the connection temperature distribution is illustrated in Figs. 7 to 9. EN 1993-1-2: 2004 gives two methods for calculating the temperature of a connection. These approaches are briefly explained below:

- 1) based on the concentration of mass in the connected parts (see expression D3.1(1));
- 2) beams are supporting concrete slabs. In this case simplified expressions are given for calculating the temperature distribution in the connection based on the temperature of the bottom flange of the supported beam at mid-span, see expression D3.1(4).

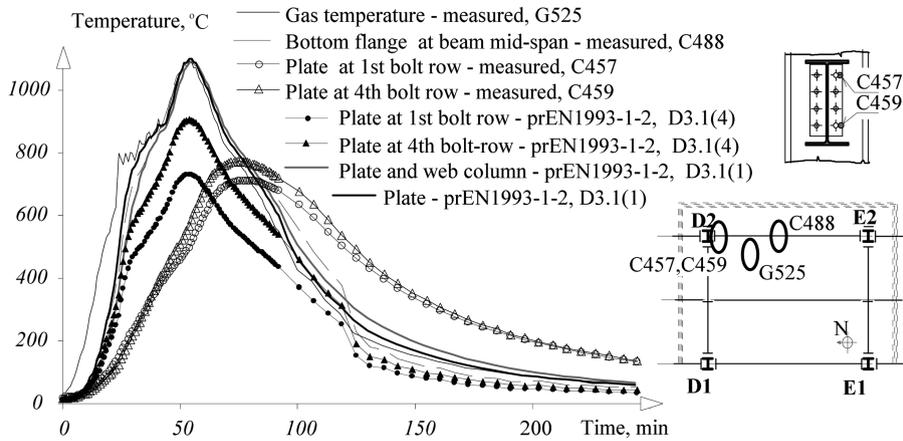


Fig. 13 Comparison between measured and predicted temperatures within the beam-to-column minor axes connection D2-E2

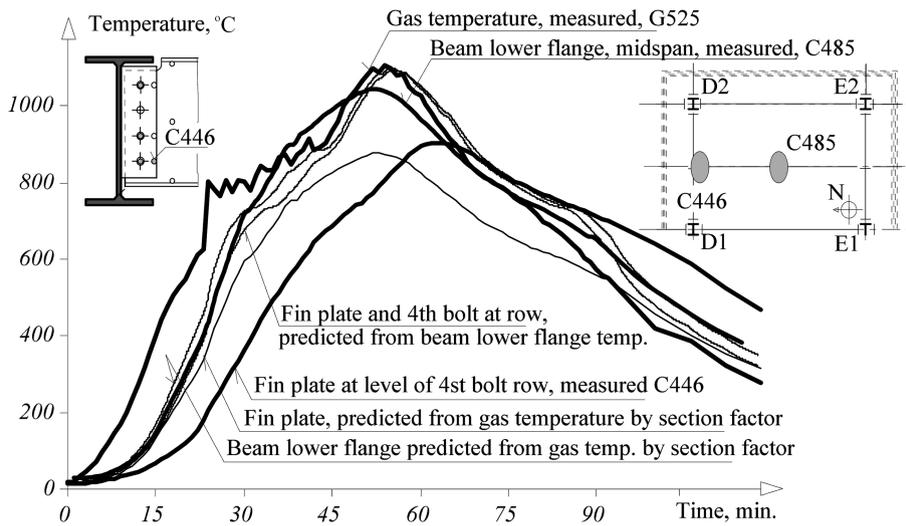


Fig. 14 Comparison between measured and predicted temperatures within the beam-to-beam fin plate connection D1.2-E1.2

The predictions by both methods are based on the measured steel temperature and are compared with the experimental results for the beam-to-column minor axes connection D2-E2 (Fig. 13). The local concentration of mass was calculated using two different approaches. The first approach is based on the thickness and additional front surface ($A_m/V = 141 \text{ m}^{-1}$) of the end-plate and column web while the second is based on the cumulated thickness of the end-plate and column web and the additional front surface ($A_m/V = 92 \text{ m}^{-1}$).

It is observed that with both approaches, the maximum temperatures are higher than the test values but occur at approximately the same time. During the cooling phase, the calculated end-plate temperatures are lower than the measured temperatures. Comparing both analytical approaches, the method based on the local mass by section factor is more conservative than the simplified expressions.

These results support a numerical study that shows that EN1993-1-2: 2003 gives conservative values during the cooling phase (Franssen and Brauwiers 2002).

Fig. 14 compares the predicted and measured temperatures observed on the fin plate connection D1.2-E1.2. The predictions are based on the temperature of the lower flange of the beam and on the gas temperature. The measured temperature of the beam bottom flange at mid-span was used to predict the temperature of the fin plate at the level of the fourth bolt row.

7. Composite slab temperatures

Slab temperatures were measured in seven locations as shown in Fig. 1. In locations S1 - S4 temperatures were measured in the ribs on the lower surface of the metal decking (0 mm), in the concrete 30 mm above the metal decking, on the reinforcement approximately 75 mm above the metal decking and on the upper surface of the concrete 130 mm above the metal decking. Temperatures were also measured next to the ribs on the lower surface of the metal decking (0 mm), in the reinforcement

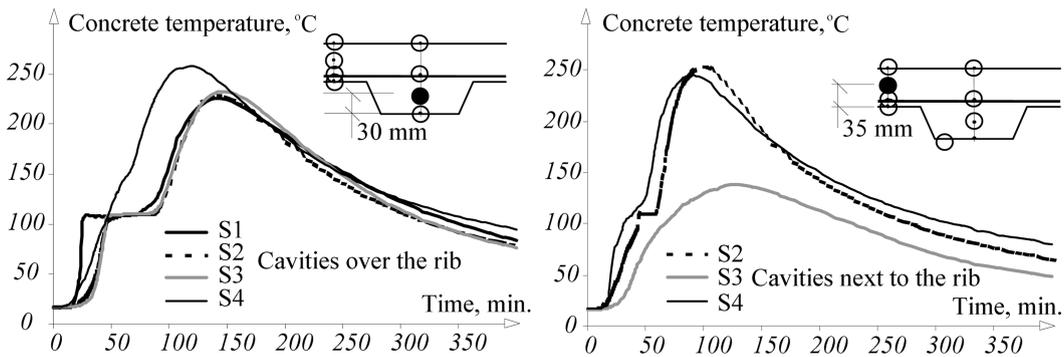


Fig. 15 Temperatures in the middle of the rib and in the middle height next to the rib

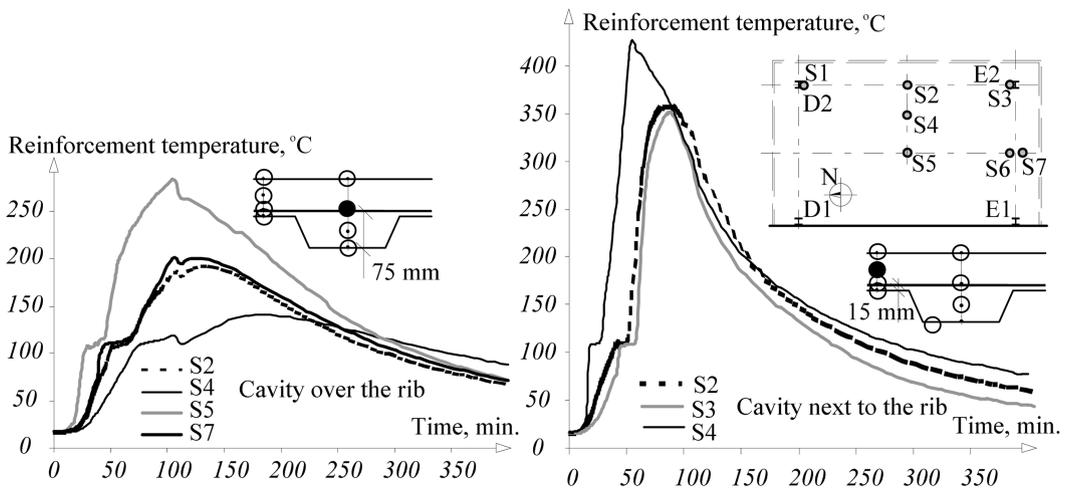


Fig. 16 Temperatures of the reinforcement over the rib and next to the rib

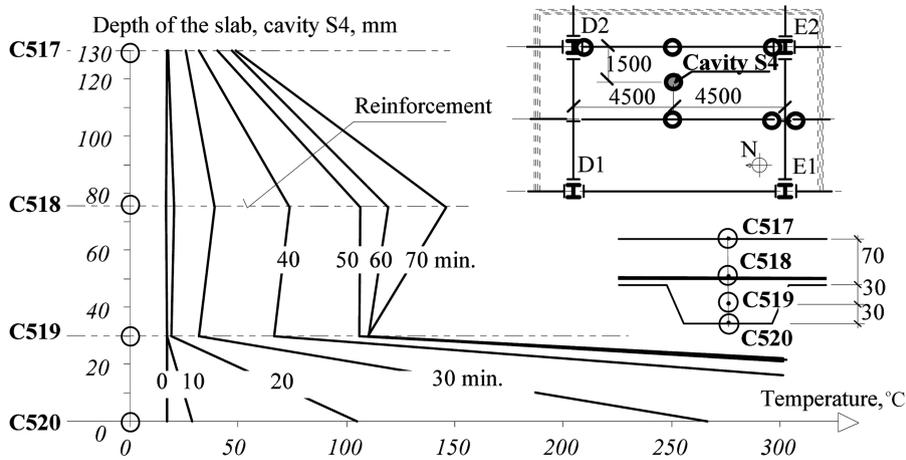


Fig. 17 Temperature variation within slab over the rib (cavity S4)

approximately 15 mm above the metal decking, in the concrete 35 mm above the metal decking and on the upper surface of the concrete 70 mm above the metal decking. The temperature of the reinforcement was measured in the ribs at locations S5 - S7.

Temperature measurements on the lower surfaces of the slab were limited because the thermocouples were connected to the metal sheeting, which debonded from the concrete in the first 20 to 30 min of the test. Maximum temperatures in the middle of the slab next to the rib (35 mm) and in the middle of the rib (30 mm) were very similar - up to 250°C in a 100 - 150 minutes, see Fig. 15. The temperatures of the reinforcement in the rib are different to those measured next to the rib, see Fig. 16. This is because of the different amounts of concrete cover. Fig. 17 shows that the temperature of the reinforcement over the rib is higher than the temperature of surrounding concrete. The temperatures of the upper surfaces of the concrete over the rib and the upper surface of the concrete next to the rib are similar with maximum temperatures of approximately 110°C, see Fig. 18.

A summary of the temperatures recorded in the slab at location S4 is presented in Fig. 17. It shows that the temperatures of the reinforcement over the rib were less than 150°C.

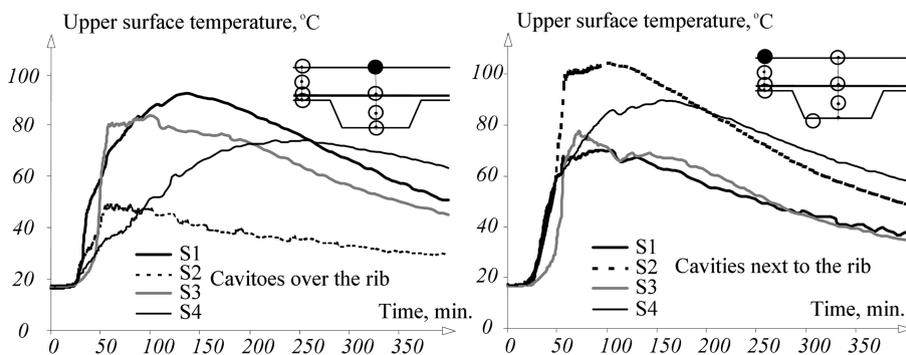


Fig. 18 Temperatures of the upper surface over the rib and next to the rib

The calculation of temperature in the concrete slab is complex compared to the procedure for calculating the steel temperatures. Because of the massive sections of concrete (compared with steel) and the thermal properties of concrete it is not possible to calculate the temperatures by using a simple analytical equation. However, the temperatures in concrete could be calculated by FEM or by using a differential method. Table D.5 of EN 1994-1-2: 2004 contains the temperatures for normal weight concrete subject to a standard time-temperature curve temperature for fire duration from 30 to 240 minutes. This table can also be used for lightweight concrete. For the preliminary prediction of the slab temperatures in this test the differential method according to Karpaš and Zoufal (1989) was used (see Annex C of this paper). The temperatures can be calculated using a spreadsheet.

The results from the differential method depend on several parameters. One of the parameters that have a significant effect is the moisture content of the concrete. The moisture content of the concrete

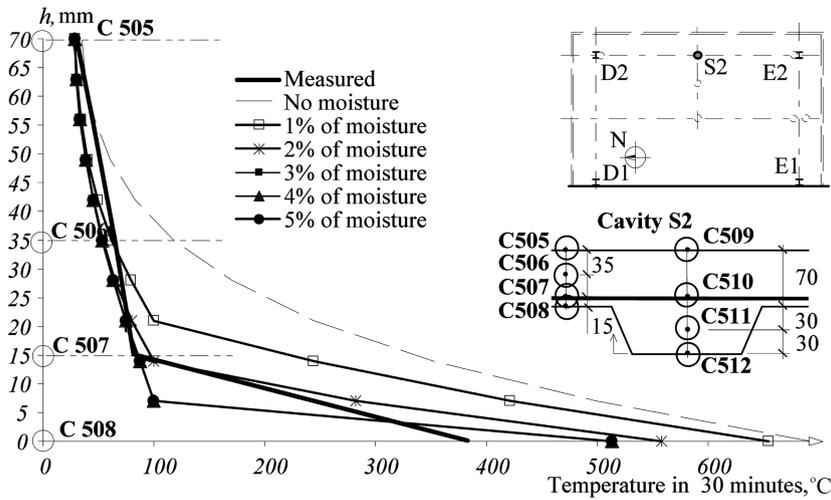


Fig. 19 Influence of the concrete moisture on slab temperatures across the height at $t = 30$ min (cavity S2)

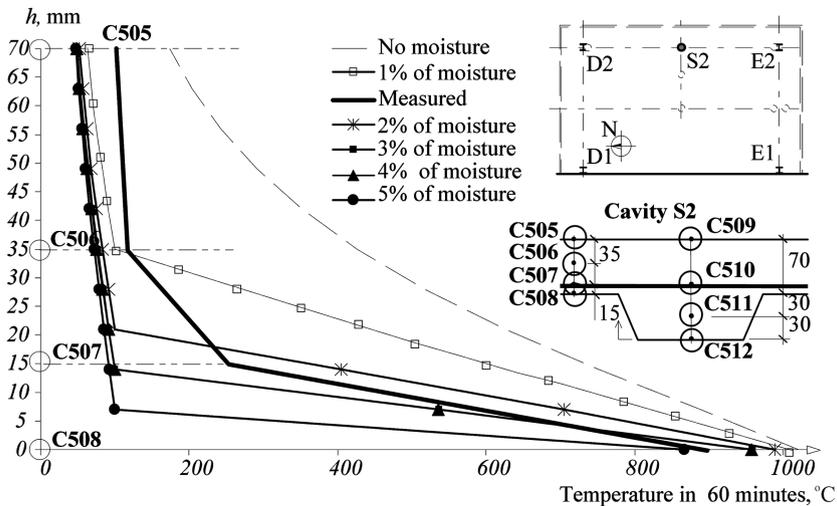


Fig. 20 Influence of the concrete moisture on slab temperatures across the height at $t = 60$ min (cavity S2)

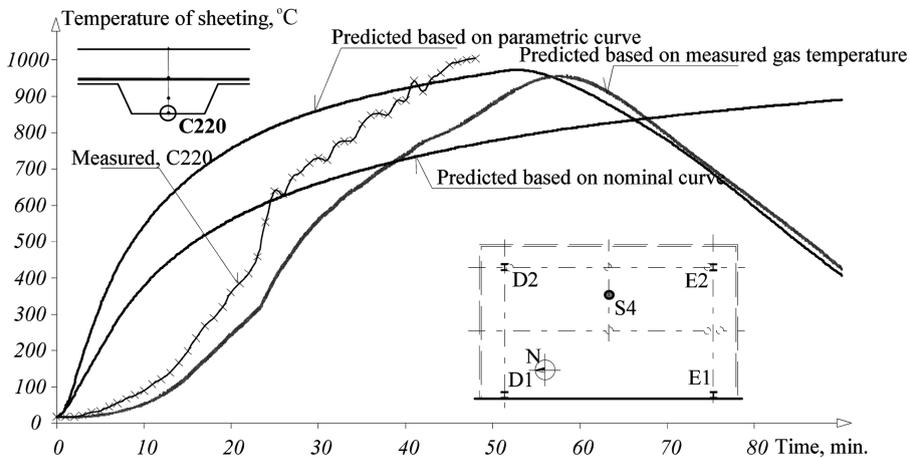


Fig. 21 Comparison between predicted and measured temperature at the cavity S4 (bottom of the rib)

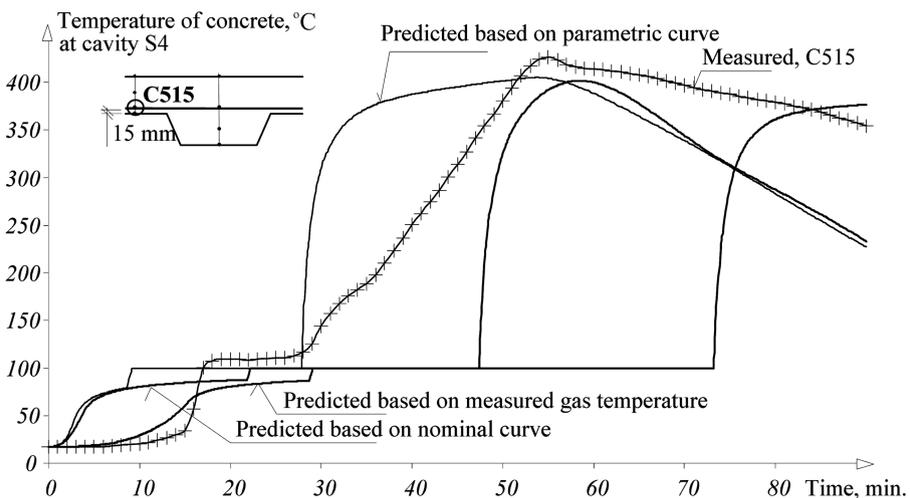


Fig. 22 Comparison between measured and predicted temperature on the cavity S4 (next to rib, 15 mm from bottom)

causes a plateau in the heating curve when temperatures of 100°C are reached (Figs. 15 and 16). Figs. 19 and 20 show the temperatures of the concrete slab next to the rib as a function of moisture content and are compared with the measured values. From measurements it is reported that the moisture content in the concrete of floors in the Cardington frame is approximately 3%.

The predicted temperatures of the concrete slab are based on a parametric time-temperature curve (Wald *et al.* 2003) and are compared with the measured values. The results of calculations based on the temperatures obtained from nominal and parametric fire curves and those obtained from the measured gas temperatures are shown in Fig. 21 and Fig. 22. The convection and radiation components of heat transfer coefficients (Annex C, Table C.1) have a significant influence on modelling.

8. Conclusions

On the 16 January 2003 a full-scale fire test was carried out at the Building Research Establishment's Cardington laboratory. One of the main aims of this fire test was to collect high quality data on the distribution of temperatures within the main structural members. This paper presents an overview of the Cardington facility together with a description of the fire test. It also presents in detail the measured temperatures in the composite steel/concrete slab, the supporting steel beam and columns and in the beam-to-column and beam-to-beam connections. Comparison is also made with the analytical methods given in EN 1993-1-2: 2004 for calculating the temperature and temperature distributions in the structural steel members. From these comparisons it can be concluded that:

1) The methods for calculating the compartment temperature given in prEN 1991-1-2: 2003 compare well with the measured data (Wald *et al.* 2005). The incremental analytical models predict the temperatures of the unprotected beams with a good accuracy. The column temperature may be predicted from the gas temperature during the heating phase, for the first 60 minutes of fire, by 2D incremental analytical models which also apply to the columns with the unprotected joint area.

2) Calculations of the connections temperature, using the measured gas temperature in the fire compartment (based on the mass of the connection parts) during the heating phase, are conservative, see Figs. 11 and 12, but a calculation based on the bottom flange temperature of the supported beam is less conservative. The analytical prediction of the temperature of the structure during its cooling will help in the next revision of the standard EN 1993-1-2: 2004 to apply the available knowledge with higher accuracy bringing high safety and economy.

3) The temperatures of the concrete slab are lower than the temperatures of the supporting steel members. The accuracy of the methods for calculating the temperature of the concrete slab is sensitive to the moisture content of the concrete.

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Annex A: measured temperatures

Table A1. Maximum gas temperatures in time intervals (°C), thermocouples 300 mm under ceiling, see Fig. 1

Thermocouple Time interval, min.	C 521	C 522	C 523	C 524	C 525	C 526	C 527	C 528	average
10 – 15	356.4	321.0	349.5	370.4	399.0	422.8	386.0	358.2	373.0
25 – 30	687.6	660.1	698.3	762.6	806.8	838.0	827.6	782.4	805.0
40 – 45	810.5	777.3	834.8	851.1	935.0	971.6	964.5	885.9	966.0
0 – 180	1 015.3	1 016.1	1 007.3	990.5	1 107.8	1 096.3	1 063.1	979.8	1 074.0
75 – 80	769.6	796.2	730.5	697.2	762.6	754.5	735.0	662.2	761.0
90 – 95	567.1	579.7	576.9	528.7	560.3	535.0	555.1	475.1	555.0

Table A2. Steel beam temperatures (°C), thermocouples numbers see Fig. 1

Thermocouple Time, min.	C 480	C 481	C 482	C 483	C 484	C 485	C 486	C 487	C 488
15	65.7	115.0	115.6	102.4	137.8	156.0	115.7	153.5	129.4
30	390.4	541.5	539.7	503.0	696.2	720.7	556.3	709.0	694.6
45	708.5	756.8	775.5	833.2	966.1	995.6	832.8	923.0	942.9
60	792.1	776.9	792.7	958.6	966.5	995.1	1 007.4	1 007.2	1 037.8
Max.	798.4	810.9	824.5	981.7	1 032.4	1 057.4	1 025.7	1 057.6	1 087.5
75	681.5	636.9	658.0	795.0	770.5	797.2	835.5	801.0	813.3
90	544.3	489.4	505.6	683.1	633.7	662.2	709.0	661.9	658.1
106	419.9	362.6	368.4	533.7	468.5	485.1	559.5	495.1	484.8
130	286.7	230.4	227.7	359.4	296.9	297.6	364.3	310.5	179.1
Position	Upper flange	Beam web	Lower flange	Upper flange	Beam web	Lower flange	Upper flange	Beam web	Lower flange

Table A3. Temperatures (°C), header plate connection D2-C2, minor axis, thermocouples numbers - see Fig. 1

Thermocouple Time, min.	C 454	C 455	C 456	C 457	C 459	C 460	C 461	C 462
15	67.0	48.5	58.5	62.1	55.1	66.9	61.3	63.1
30	233.0	187.4	220.9	231.2	216.7	270.9	273.0	281.3
45	422.0	386.5	447.7	410.1	446.2	439.0	491.8	545.0
60	601.5	623.5	672.7	589.3	673.9	608.7	706.3	774.0
75	726.4	743.2	779.1	708.9	772.2	713.4	779.4	816.3
Max.	728.0	745.3	779.1	711.1	772.2	714.4	780.9	846.7
90	699.6	719.0	735.2	687.5	731.6	679.1	726.3	725.9
106	596.2	620.1	635.4	591.8	637.2	564.1	616.5	583.6
130	431.5	440.8	450.4	429.0	451.0	401.7	429.4	383.9
160	297.1	301.2	305.2	296.8	306.4	277.8	288.3	253.5
Position	1 st bolt	2 nd bolt	4 th bolt	Plate 1 st row	Plate 4 th row	Upper flange	Web	Lower flange

Table A4. Temperatures (°C) at header plate connection D2-D1, major axis, thermocouples numbers - see Fig. 1

Thermocouple Time, min.	C 466	C 467	C 468	C 469	C 470	C 471	C 463	C 464	C 417
15	67.4	60.8	61.4	74.1	68.2	67.3	64.0	100.1	89.4
30	241.2	270.4	274.0	319.5	324.0	323.9	334.4	470.1	422.8
45	476.8	512.8	519.4	516.7	567.9	572.5	553.5	654.2	636.5
60	655.3	713.4	735.7	717.6	785.9	800.9	724.0	881.0	870.5
75	733.2	797.9	804.3	758.8	808.2	808.3	747.7	798.8	818.8
Max.	733.8	800.0	811.7	765.8	831.3	838.6	762.0	905.5	916.0
90	692.7	734.7	734.8	691.5	730.3	723.5	679.4	687.9	709.4
106	581.3	631.6	628.5	583.3	619.1	608.1	567.4	540.1	552.1
130	412.1	433.2	435.8	415.6	427.0	423.6	408.7	365.6	354.5
160	284.9	299.8	305.2	290.0	297.7	296.2	289.9	253.4	233.8
Position	1 st bolt	3 rd bolt	4 th bolt	Plate 1 st row	Plate 3 rd row	Plate 4 th row	Upper flange	Web	Lower flange

Table A5. Temperatures (°C) at fin plate connection D1.2-E1.2, thermocouples numbers - see Fig. 1.

Thermocouple Time, min.	C 441	C 442	C 443	C 444	C 446	C 447	C 448	C 449
15	68.5	66.4	70.2	65.6	70.5	98.2	85.9	129.5
30	343.0	350.1	367.6	331.1	368.8	424.5	425.5	570.0
45	636.3	671.5	686.9	635.8	691.6	671.2	726.5	812.4
60	805.3	862.9	894.5	810.3	899.1	848.6	912.9	975.5
Max.	825.6	881.4	907.2	834.3	908.3	859.1	913.8	981.6
75	789.1	810.8	817.4	792.9	816.4	764.0	784.7	798.3
90	703.6	717.0	718.8	702.0	716.7	663.9	686.7	692.0
106	587.0	598.4	597.1	580.7	591.1	527.5	542.4	534.7
130	396.2	391.4	382.9	390.6	383.9	373.6	362.1	346.9
160	257.1	249.2	242.1	254.1	244.1	257.9	236.8	225.5
Position	1 st bolt	3 rd bolt	4 th bolt	Plate. 1 st row	Plate. 4 th row	Upper flange	Web	Lower flange

Table A6. Temperatures (°C) in slab - cavities S2 and S4, thermocouples numbers - see Figs. 1 and 17

Time, min	Cavity S4, next to the rib			Cavity S4, across the rib				Cavity S2, across the rib			
	C 513	C 514	C 515	C 517	C 518	C 519	C 520	C 509	C 510	C 511	C 512
15	17.3	19.0	34.5	17.5	17.1	21.1	199.3	17.5	17.9	17.6	52.6
30	27.5	94.3	144.1	21.6	25.3	54.2	731.5	25.7	39.2	32.2	266.5
45	53.6	117.9	313.7	30.0	50.8	102.8	986.6	36.9	84.7	100.9	661.8
60	64.3	185.1	413.9	36.2	81.1	142.5	*	48.4	109.6	109.1	936.5
75	73.3	233.6	387.6	38.8	108.7	182.8	*	47.5	134.6	110.1	776.3
90	80.3	245.1	354.2	45.5	113.1	230.7	*	46.4	162.4	113.7	667.1
Max	89.7	245.3	426.7	74.0	140.4	257.9	1022.8	48.8	192.1	228.9	1040.6
106	86.0	237.4	307.2	52.0	118.9	255.5	*	45.1	185.8	163.6	*
130	87.0	209.9	237.7	59.9	119.9	253.8	*	41.7	191.9	222.4	*
160	89.7	179.9	192.5	67.2	137.4	225.5	*	39.1	183.6	222.0	*
184	87.7	161.1	168.7	70.9	140.3	201.7	*	37.6	166.4	200.3	*
Position	70 mm	35 mm	15 mm	130mm	75 mm	30 mm	0 mm	130 mm	75 mm	30 mm	0 mm

*Connection to the thermocouple was lost.

Annex B: design models

EN 1993-1-2: 2004 enables to predict the transfer of heat from the fire compartments to unprotected as well as protected steelwork. For an equivalent uniform temperature distribution in the cross-section, the increase of temperature $\Delta\theta_{a,t}$ in an unprotected steel member during a time interval Δt should be determined from clause 4.2.5.1 as:

$$\Delta\theta_{a,t} = k_{sh} \frac{A_m/V}{c_a \rho_a} \dot{h}_{net,d} \Delta t \quad (\text{B.1})$$

where:

- k_{sh} is correction factor for the shadow effect. It is used in case of nominal standard time temperature curves. This factor was not taken into account in prediction.
- A_m/V is the section factor for unprotected steel members.
- A_m is the surface area of the member per unit length.
- V is the volume of the member per unit length.
- c_a is the specific heat of steel.
- $\dot{h}_{net,d}$ is the design value of the net heat flux per unit area.
- Δt is the time interval, taken as 5 seconds
- ρ_a is the unit mass of steel.

The value of $\dot{h}_{net,d}$ should be obtained from EN 1991-1-2 using $\varepsilon_f = 1.0$ and $\varepsilon_m = 0.7$ where ε_f , ε_m are as defined in EN 1991-1-2.

For a uniform temperature distribution in a cross-section, the temperature increase $\Delta\theta_{a,t}$ of an insulated steel member during a time interval Δt should be obtained from EN 1993-1-2: 2004 par. 4.2.5.2 as

$$\Delta\theta_{a,t} = \frac{\lambda_p A_p / V (\theta_{g,t} - \theta_{a,t})}{d_p c_a \rho_a (1 + \phi / 3)} \Delta t - (e^{\phi/10} - 1) \Delta\theta_{g,t} \text{ (but } \Delta\theta_{a,t} \geq 0 \text{ if } \Delta\theta_{g,t} > 0) \quad (\text{B.2})$$

with:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p / V$$

where:

- A_p / V is the section factor for steel members insulated by fire protection material;
- A_p is the appropriate area of fire protection material per unit length of the member, which should generally be taken as the area of its inner surface.
- V is the volume of the member per unit length.
- c_a is the temperature dependant specific heat of steel.
- c_p is the temperature independent specific heat of the fire protection material.
- d_p is the thickness of the fire protection material.
- Δt is the time interval taken as 30 seconds.
- $\theta_{a,t}$ is the steel temperature at time t .
- $\theta_{g,t}$ is the ambient gas temperature at time t .
- $\Delta\theta_{g,t}$ is the increase of the ambient gas temperature during the time interval Δt .
- λ_p is the thermal conductivity of the fire protection system;
- ρ_a is the unit mass of steel.
- ρ_p is the unit mass of the fire protection material.

For beam to column and beam to beam connections, where the beams are supporting any type of concrete floor, the temperature for the connection may be obtained from the temperature of the bottom flange at mid span. The connection temperature may be predicted, if the depth of the beam is less than 400 mm. see EN 1993-1-2: 2004 Annex D 3.1. as:

$$\theta_h = 0.88 \theta_o (1 - 0.3 h/D) \quad (\text{B.3})$$

where

- θ_a is the temperature at height h of the steel beam.
- θ_o is the bottom flange temperature of the steel beam remote from the connection.
- h is the height of the component being considered above the bottom of the beam.
- D is the depth of the beam.

Annex C: differential method for slab temperature calculation

Heating of the member depends on the heat transfer between the surrounding environment and the heat conduction within the member. This is expressed by Fourier heat transfer equation for non-steady heat conduction inside the member

$$\frac{\partial}{\partial x} \left(\lambda_x \frac{\partial \theta}{\partial x} \right) + \frac{\partial}{\partial y} \left(\lambda_y \frac{\partial \theta}{\partial y} \right) + \frac{\partial}{\partial z} \left(\lambda_z \frac{\partial \theta}{\partial z} \right) + \bar{Q} = \rho c \frac{\partial \theta}{\partial t} \quad (\text{C.1})$$

where

- $\lambda_x, \lambda_y, \lambda_z$ are thermal conductivities.
- ρ is density.
- c is specific heat capacity.
- q is temperature.
- \bar{Q} is internally generated heat.

In preliminary calculations of the slab temperatures the simplification into one-dimensional problem is possible

$$\frac{\partial}{\partial x} \left(\lambda_x \frac{\partial \theta}{\partial x} \right) = \rho c \frac{\partial \theta}{\partial t} \quad (\text{C.2})$$

Boundary conditions are defined by time-temperature curve and by heat transfer which is characterised by heat transfer coefficients - convective and radiative. Dominant at high temperatures is radiation component which can be estimated as

$$\alpha_r = \frac{5,77 \varepsilon_r}{\theta_g - \theta_k} \left[\frac{(\theta_g + 273)^4}{100^4} - \frac{(\theta_k + 273)^4}{100^4} \right] \quad (\text{C.3})$$

where

- ε_r is resultant emissivity.
- θ_g is gas temperature.
- θ_k is member surface temperature.

Table C.1. Heat transfer coefficients

Literature		α_r	α_c
(Karpaš and Zoufal, 1989)	exposed conc. surface	$\alpha_r = \frac{3,75}{\theta_g - \theta_k} \left[\frac{(\theta_g + 273)^4}{100^4} - \frac{(\theta_k + 273)^4}{100^4} \right]$	16.7
	not exposed conc. surface	$\alpha_r = \frac{4,64}{\theta_g - \theta_k} \left[\frac{(\theta_g + 273)^4}{100^4} - \frac{(\theta_k + 273)^4}{100^4} \right]$	11.4
(Petterson <i>et al.</i> , 1976)	exposed surface	$\alpha_r = \frac{5,77 \varepsilon_r}{\theta_g - \theta_k} \left[\frac{(\theta_g + 273)^4}{100^4} - \frac{(\theta_k + 273)^4}{100^4} \right]$ $\varepsilon_r = \frac{1}{1/\varepsilon_m + 1/\varepsilon_f - 1}$	23
	not expos. surf	0.033 θ_u ; where θ_u is temperature of non-exposed surface	8.7
(ENV 1991-1-2; 1994)	exposed surface	$\alpha_r = \frac{\Phi 5,67 \varepsilon_r}{\theta_g - \theta_k} \left[\frac{(\theta_g + 273)^4}{100^4} - \frac{(\theta_k + 273)^4}{100^4} \right]$ $\Phi = 1.0$ $\varepsilon_m * \varepsilon_f = 0.7 * 0.8 = 0.56$	25
	not expos. surf	neglected	9
(ENV 1991-1-2)	exposed surface	$\alpha_r = \frac{\Phi 5,67 \varepsilon_r}{\theta_g - \theta_k} \left[\frac{(\theta_g + 273)^4}{100^4} - \frac{(\theta_k + 273)^4}{100^4} \right]$ $\Phi = 1.0$ $\varepsilon_m * \varepsilon_f = 0.8 * 1.0 = 0.8$	35
	not expos. surf	9	

ε_f is the emissivity of flames and ε_m the emissivity of the surface.

Different heat transfer coefficients according to different authors are shown in Table C.1.

In differential method in one-dimensional heat transfer the Fourier Eq. (C.2) is simplified in the form

$$\frac{\Delta\theta}{\Delta t} = \frac{\lambda}{c\rho\Delta x^2} \Delta^2\theta \quad (\text{C.4})$$

The slab is divided into layers. The layer thickness Δx cannot be too big. for $1000 < \rho < 2000$ the recommended maximum layer thickness is 20 mm. Temperatures are calculated in time intervals

$$\Delta t \leq \frac{\Delta x^2}{2a} \quad \text{with} \quad a = \frac{\lambda}{c\rho} \quad (\text{C.5})$$

Temperature of the surface layer is calculated as

$$\theta_{1,t+1} = C_1 \cdot \theta_{N,t} + C_2 \cdot \theta_{1,t} + C_3 \cdot \theta_{2,t} \quad (\text{C.6})$$

Where :

$\theta_{N,t}$ is the surface temperature.

$\theta_{1,t}$ and $\theta_{2,t}$ are temperatures of inner layers.

C_1 , C_2 and C_3 are coefficients as a function of material properties λ , c , ρ and heat transfer coefficient $\alpha = \alpha_r + \alpha_c$.

Temperature of the internal layer is calculated as

$$\theta_{m,t+1} = C_4 \cdot \theta_{m-1,t} + C_5 \cdot \theta_{m,t} + C_4 \cdot \theta_{m+1,t} \quad (\text{C.7})$$

Where:

$\theta_{m,t}$ is the internal layer temperature.

C_4 and C_5 are coefficients as a function of material properties λ , c and ρ .

Influence of the moisture content is taken into account by the temperature increment which expresses the amount of heat necessary to evaporation of water.

$$\Delta\theta_m = \frac{\bar{E}}{100} 2,26 \cdot 10^6 \frac{v}{100c} \quad (\text{C.8})$$

Where:

\bar{E} is evaporating water in % (for members heated from one side = 40%).

$2,26 \cdot 10^6$ is the heat necessary for water evaporation.

v is the moisture content.

When the temperature of 100°C is reached, all other temperatures will be 100°C till the moment when temperature increment is bigger than $\Delta\theta_m$. After this moment the calculations continue in normal way.

CC