Experimental behaviour of a steel structure under natural fire

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Abstract

Current design codes for fire resistance of structures are based on isolated member tests subjected to standard fire conditions. Such tests do not reflect the behaviour of a complete building under either normal temperature or fire conditions. Many aspects of behaviour occur due to the interaction between members and cannot be predicted or observed in tests of isolated elements. Performance of real structures subject to real fires is often much better than that predicted from standard tests due to structural continuity and the provision of alternative load paths.

This paper reports on the results of a collaborative research project (Tensile membrane action and robustness of structural steel joints under natural fire, European Community FP5 project HPRI—CV 5535) involving the following institutions: Czech Technical University (Czech Republic), University of Coimbra (Portugal), Slovak Technical University (Slovak Republic) and Building Research Establishment (United Kingdom). It consists of an experimental programme to investigate the global structural behaviour of a compartment on the 8-storey steel–concrete composite frame building at the Cardington laboratory during a BRE large-scale fire test, aimed at the examination of the temperature development within the various structural elements, the corresponding (dynamic) distribution of internal forces and the behaviour of the composite slab, beams, columns and connections.
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Keywords: Structural engineering; Steel and composite structures; Full-scale tests; Fire design; Structural integrity

1. Introduction

Significant developments have been made in analysing the behaviour of steel-framed structures under fire conditions in the last 10 years. Due to the high cost of full-scale fire tests and size limitations of existing furnaces, these studies are based on the observation of real fires and on tests performed on isolated elements subjected to standard fire regimes, which serve as reference heating, but do not model the natural fire. However, the failure of the World Trade Centre on 11th September 2001 and, in particular, of building WTC7, alerted the engineering profession to the possibility of connection failure under fire conditions. Many aspects of behaviour occur due to the interaction between members and cannot be predicted or observed from isolated tests, such as global or local failure of the structure, stresses and deformations due to the restraint to thermal expansion by the adjacent structure, redistribution of internal forces, etc. Moreover, isolated tests do not reflect the behaviour of a complete structure.

Unlike the standard fire curve a natural fire is characterized by three phases: a growing phase, a full developed phase and a decay phase. It is necessary to evaluate not only the effect on the structural resistance during the heating phase, but also the high cooling strains in the joint induced by distortional deformation of the heated elements during the decay phase.

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Table 1
Summary of full-scale fire tests

Authors	Year	Objectives
Pettersson et al. [2]	1976	Gas time–temperature in different fire compartments. Data in a form readily accessible to the practising engineer.
Witteveen et al. [3]	1977	The first reported test to assess structural behaviour under fire conditions. The stability of braced and unbraced frame at elevated temperature was studied.
Kruppa [4]	1981	Behaviour of external steel columns in a fire compartment.
Genes [5]	1982	Large-scale fire test on a compartment designed to simulate two floors of a 20-storey building, in order to assess the performance of protected beams.
Latham et al. [6]	1985	Steel time–temperature curves for unprotected structural steelwork for several sections exposed in a large fire compartment; variation of the fire load, the ventilation conditions and the thermal properties of the enclosing surfaces.
Rubert and Schaumann [7]	1986	Tested a series of quarter-to-half scale fire tests on steel sub-assemblies under fire in order to obtain the failure temperature of the heated steel members.
Anon [8]	1986	Fire behaviour of steel and composite construction.
Cooke and Latham [9]	1987	First test on a full-size loaded steel frame subjected to a natural fire using wooden cribs. This test showed that the performance of the frame was better than that of the individual elements as a result of the connection continuity that may be exploited to provide increased fire resistance of the beam.
Thomas et al. [10]	1992	Performance of light hazard sprinkler systems in real compartments. Structural behaviour of an unprotected composite structure.
Cardington Laboratory	1993–2003	See §2.3 of this paper.

In order to address these issues, and to improve the knowledge of the structural integrity of structures under fire conditions, it is the objective of this paper to report on the results of a collaborative research project (Tensile membrane action and robustness of structural steel joints under natural fire, European Community FP5 project HPRI—CV 5535) involving the following institutions: Czech Technical University (Czech Republic), University of Coimbra (Portugal), Slovak Technical University (Slovak Republic) and Building Research Establishment (United Kingdom). It consists of an experimental programme to investigate the global structural behaviour of a compartment on the 8-storey steel–concrete composite frame building at Cardington Laboratory during a BRE large-scale fire test on January 16, 2003, aimed at the examination of the temperature development within the various structural elements, the corresponding (dynamic) distribution of internal forces and the behaviour of the composite slab, beams, columns and connections.

2. Experimental research on the fire performance of structures

2.1. Full-scale fire tests

Over the years many isolated member tests have been carried out in several institutions. However, investigations involving full-scale tests under natural fire are limited. The development of the Cardington Laboratory of the Building Research Establishment (BRE) has provided the opportunity to carry out several research projects that included

full-scale fire tests. A complete state-of-art description of the experimental observations can be found in Wang [1], a brief summary being presented in Table 1.

2.2. The Cardington Laboratory

The Cardington Laboratory is a unique worldwide facility for the advancement of the understanding of whole-building performance. Most aspects of a building's lifecycle, from fabrication to fire resistance and explosions through to demolition, can be investigated on real buildings. This facility is located at Cardington, Bedfordshire, UK and consists of a former airship hangar with dimensions $48\,\text{m} \times 65\,\text{m} \times 250\,\text{m}$. It is used by industrial organizations, universities and research institutes, government departments and agencies. The BRE's Cardington Laboratory comprises three experimental buildings: a six-storey timber structure, a seven-storey concrete structure and an eight-storey steel structure.

The steel test structure was built in 1993. It is a steel framed construction using composite concrete slabs supported by steel decking in composite action with the steel beams. It has eight storeys (33 m) and is five bays wide ($5 \times 9 = 45$ m) by three bays deep ($6 + 9 + 6 = 21$ m) in plan, see Fig. 1. The structure was built as non-sway with a central lift shaft and two end staircases providing the necessary resistance against lateral wind loads. The main steel frame was designed for gravity loads, the connections consisting of flexible end plates for beam-to-column connections and fin plates for beam-to-beam connections were designed to transmit vertical shear loads. The building
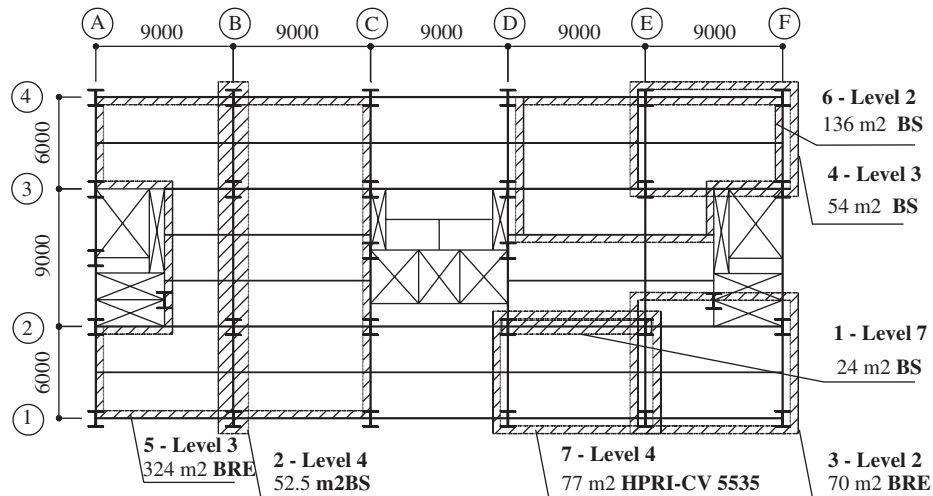
Fig. 1. The Cardington fire tests on steel structure.

Table 2
Fire test on steel structure in Cardington Laboratory [15]

No.	Test	Fire compartment		Load	
		Size, (m)	Area (m^2)	Fire	Mechanical (%)
1	One beam heated by gas	8×3	24	Gas	30
2	One frame heated by gas	21×2.5	53	Gas	30
3	Corner compartment	9×6	54	40 kg/m^2 of wood cribs	30
4	Corner compartment	10×7	70	45 kg/m^2 of wood cribs	30
5	Large compartment	21×18	342	40 kg/m^2 of wood cribs	30
6	Office—Demonstration	18×9	136	45 kg/m^2 of wood cribs	30
7	Structural integrity	11 x7	77	40 kg/m^2 of wood cribs	56

simulates a real commercial office in the Bedford area and all the elements were verified according to British Standards and checked for compliance with the provisions of the Structural Eurocodes.

The building was designed for a dead load of 3.65 kN/m^2 and an imposed load of 3.5 kN/m^2, see [11]. The floor construction is of steel deck and light-weight in situ concrete composite floor, incorporating an anti-crack mesh of 142 mm^2/m in both directions. The floor slab has an overall depth of 130 mm and the steel decking has a trough depth of 60 mm.

2.3. Fire tests at Cardington Laboratory

Seven large-scale fire tests at various positions within the experimental building were conducted; see Fig. 1 and Table 2 [12]. The main objective of the compartment fire tests was to assess the behaviour of structural elements with real restraint under a natural fire.

The first test performed in Cardington was a restrained beam test involving a single $305 \times 165 \times 40$UB composite beam section supporting the seventh floor of the building [13]. A gas-fired furnace was used to heat the beam to approximately 900 °C. The second test, a plane frame test, involved heating a series of beams and columns across the full width of the building. Again, a gas-fired furnace was used to heat the steelwork to approximately 800 °C. The third test, the British Steel corner compartment test was the first natural fire carried out in the Cardington Laboratory, representing a typical office fire (timber cribs were used to provide a fire load of 45 kg/m^2). In this test, both the perimeter beams and the columns were fire protected with the internal beam unprotected. Load bearing concrete blocks were used for the compartment walls. The fourth test, the BRE corner compartment test also used timber cribs to provide a fire load of 40 kg/m^2. The compartment walls were constructed using fire-resistant board and the northern boundary was formed by constructing double-glazed aluminium screens. All columns were protected up to and including the connections. It was observed that the fire development was largely influenced by the lack of oxygen in the compartment [14]. The fifth test was the largest compartment test in the world. The compartment was designed to represent a modern open-plan office (18 m × 21 m). The compartment was bounded by fire-resistant walls. The main aim of this test was to investigate the ability of a large area of composite slab to support the applied load once the main beams had failed.

Table 3
Summary of results from major fire tests in the Cardington laboratory [16]

No.	Org.	Floor	Time (min) to max. atmosphere temp.	Maximum temperature (°C)		Measured deformations (mm)	
				Gas	Steel	Maximal	Residual
1	BS[a]	7	170	913	875	232	113
2	BS	4	125	820	800	445	265
3	BS	3	114	1000	903	269	160
4	BRE[b]	2	75	1020	950	325	425
5	BRE	3	70	—	691	557	481
6	BS	2	40	1150	1060	610	—
7	ČVUT[c]	4	55	1108	1088	>1000	925

[a]BS, British Steel (now Corus).
[b]BRE, Building Research Establishment.
[c]ČVUT, collaborative research proposed by Czech Technical University.
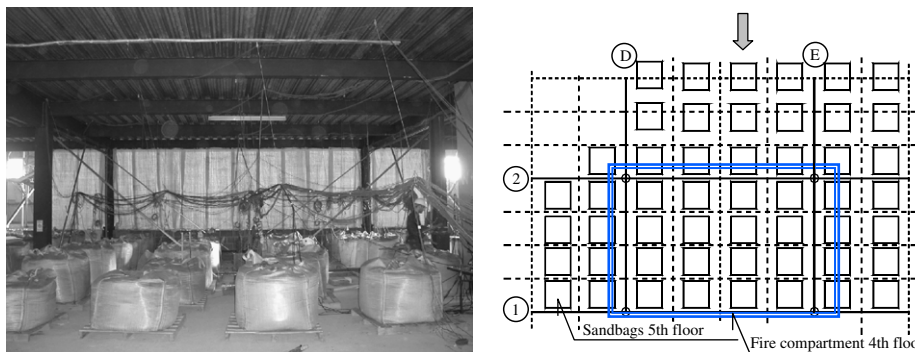
Fig. 2. Sandbags on the 5th floor.

Consequently, all the beams had no fire protection and all columns were fire protected. Again, the ventilation conditions governed the fire severity. In the demonstration test (sixth test), unlike in the previous tests, real furniture (desks, chairs, filling cabinets, computer terminals, etc.) were used to provide the fire load. The ventilation was provided by windows and blank openings. The beams were unprotected while the columns were protected. This test was characterized by a rapid rise in temperature representing a severe fire scenario. The principal results of these tests are summarized in Table 3 [16], where, for ease of comparison, the 7th test was also included.

3. Structural integrity test program

3.1. Fire compartment

The fire test was carried out in a centrally located compartment of the building (Fig. 1), enclosing a plan area of $11 \, m \times 7 \, m$ on the 4th floor [16], after 4 months of preparation. The identification of the compartment is illustrated in Fig. 2.

The mechanical load was simulated using sandbags, each weighing 1100 kg. Fig. 2 illustrates the loaded area on the 5th floor. In addition to the self-weight of the structure, sandbags were used to represent the following mechanical loadings: the remaining permanent actions, 100% of

variable permanent actions and 56% of live actions. The mechanical load was designed to reach the local collapse of the floor at the peak temperature, based on analytical and FE simulations [17].

The fire load was provided by $40 \, kg/m^2$ of wooden cribs (moisture contents < 14%) covering the compartment floor area (Fig. 3). The fire compartment was bounded with three layers of plasterboard $(15 \, mm + 12.5 \, mm + 15 \, mm)$ with a thermal conductivity around 0.19–0.24 W/mK. In the external wall (gridline 1) the plasterboard was fixed to a 1.5 m high brick wall. An opening 1.27 m high and 9 m long simulated an open window to ventilate the compartment and to allow the observation of the behaviour of the various elements. Preliminary calculations of the complex simulation of fire development predicted a short and hot fire [16].

The columns, external joints and connected beam (1.0 m from the joints only) were fire protected to prevent global structural instability (Fig. 4). The material protection used was 15 mm of Cafco300 vermiculite-cement spray, with a thermal conductivity of 0.078 W/mK at room temperature.

3.2. Structural arrangement

The steel structure exposed to fire consists of two beam sections ($356 \times 171 \times 51$ UB for the edge beams and the 6 m
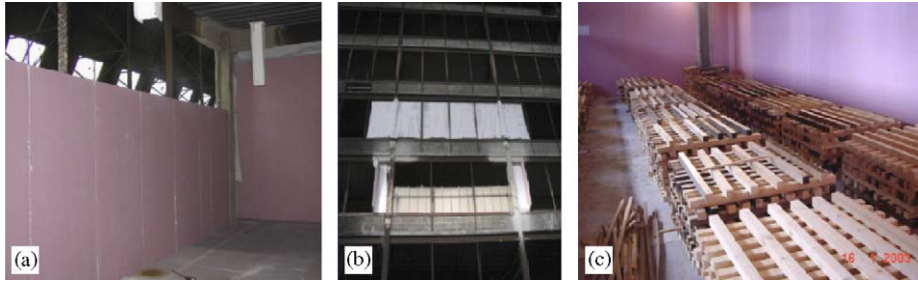
Fig. 3. (a) internal view of the compartment; (b) external view of the compartment and (c) fire loading.
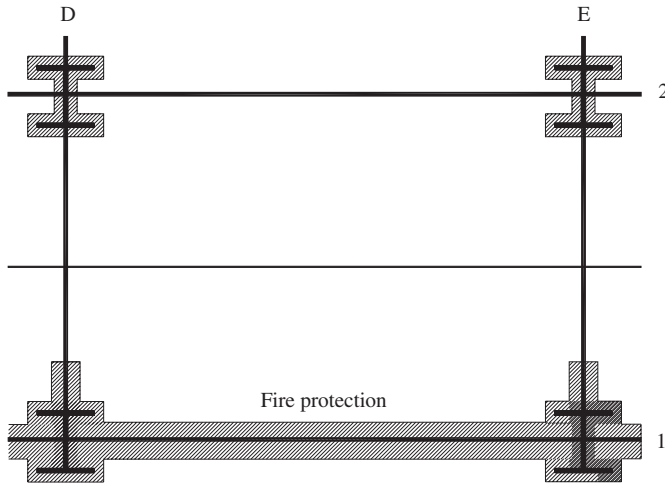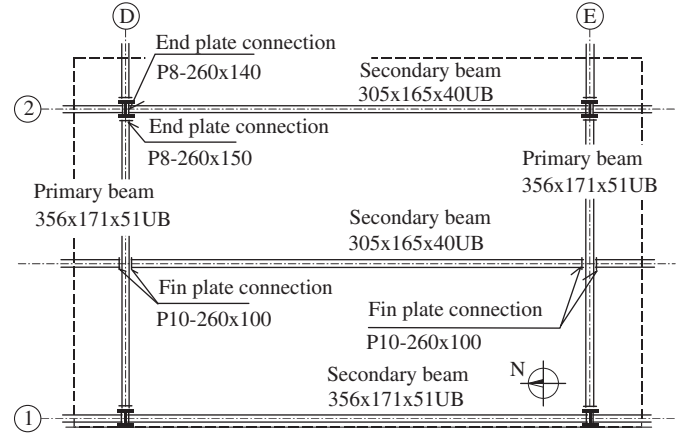

Fig. 4. Fire protection arrangement.

Fig. 5. Arrangement of members in selected fire compartment.

primary beams and $305 \times 165 \times 40$ UB for the internal secondary beams) and two columns ($305 \times 305 \times 198$ UC and $305 \times 305 \times 137$ UC) as shown in Fig. 5. The joints were a cruciform arrangement of a single column with either three or four beams connected, respectively, to the column flange or web. The composite behaviour was achieved by a concrete slab over the beams cast on shear studs. The measured sections geometry is presented in Fig. 6 and Table 4.

3.3. Laboratory equipment and instrumentation

The main requirements of the instrumentation were to measure the temperature, the distribution of internal forces and the deflected shape of the floor and of the main structural elements. The instrumentation included thermocouples, strain gauges and displacement transducers. A total of 133 thermocouples were used to monitor the temperature in the connections and beams within the compartment, the temperature distribution through the slab and the atmosphere temperature within the compartment. An additional 14 additional thermocouples were used to monitor the temperature of the protected columns. In order to measure the elements stresses and deformations, two different types of gauge were used: high temperature ones and ambient temperature ones. In the exposed and un-protected elements high-temperature strain
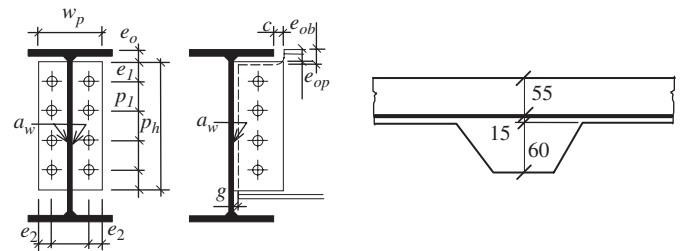

Fig. 6. Connection and slab geometry.

gauges were used. The number of these instruments (9) was limited because of economic reasons, so these strain gauges were used in the connections (fin plate and end plate—minor axis) only (Fig. 7). In the protected columns and unexposed elements a total of 47 ambient strain gauges were installed. Twenty five vertical displacement transducers were installed directly above the 5th floor, in a square mesh, to measure the deformation of the concrete slab, see Fig. 8. Twelve additional transducers were used to measure the horizontal movement of the columns and the Laboratory 10 video cameras and two thermo imaging cameras recorded the fire and smoke development, the structural deformations and the temperature distribution with time.

Table 4
Connections measured geometry

Connection	Grid number	h_p (mm)	w_p (mm)	t_p (mm)	e_0 (mm)		e_1 (mm)	p_1 (mm)	e_2 (mm)	g (mm)	c (mm)	a_w (mm)
Fin plate	D1.5	259	100	10	22	27	40	60	50	10	20	6
	E1.5	260	100	10	25	25	40	60	50	9	20	7
End plate	D2 maj	260	150	8	20		40	60	30	—	—	6
	D2 min	262	140	8	25		40	60	30	—	—	6
	E2 maj	259	150	8	22		40	60	30	—	—	6
	E2 min	259	140	8	25		40	60	30	—	—	6
Bolts	M20											

Being: h_p the height of the plate, w_p the width of the plate, t_p the thickness of the plate, e_0 distance between the upper flange and the plate., e_1 the end distance from the centre of the fastener hole to the adjacent end of any part, measured in the direction of the load transfer, p_1 the spacing between centres of fasteners in a line in the direction of load transfer, e_2 the edge distance from the centre of the fastener hole to the adjacent edge of any part, measured at right angles to the direction of the load transfer, g gap between the web of the primary beam and secondary beam, c distance between the upper flange of primary beam and secondary beam, a_w the thickness of the weld on the beam web.
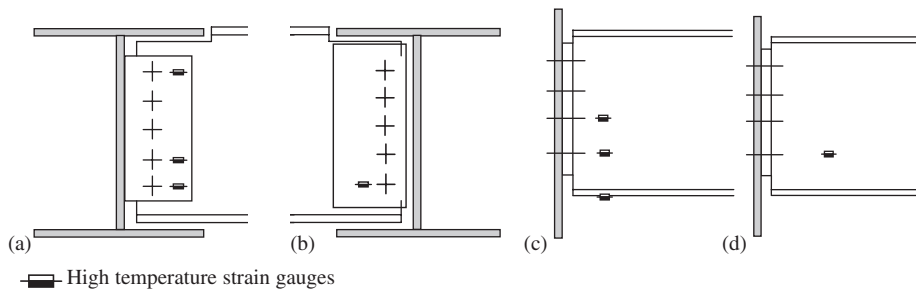
Fig. 7. Location of the high-temperature strain gauges: (a) D1.5; (b) E1.5; (c) D2; (d) E2.
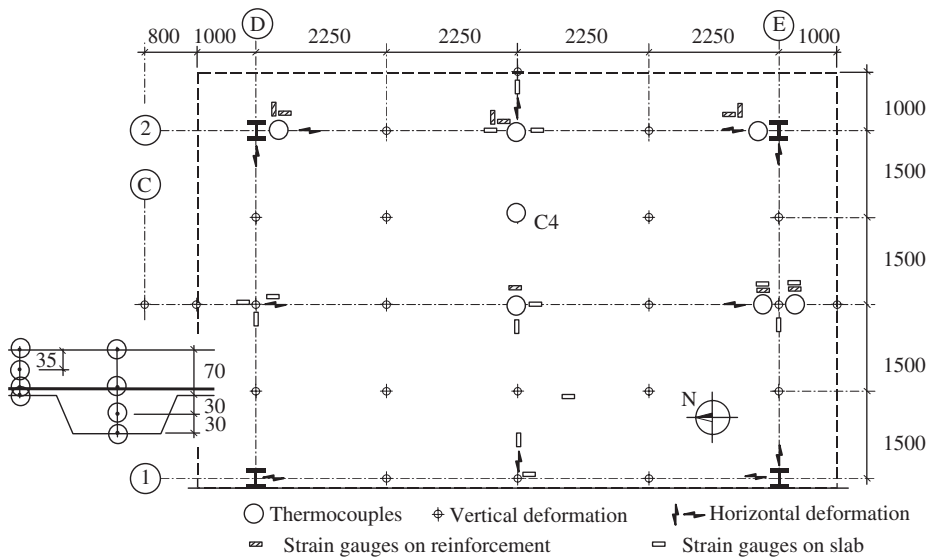

Fig. 8. Location of measurements on the slab, at 5th floor level.

3.4. Mechanical properties of structure

Table 5 reproduces the material properties at ambient temperature for the steel and concrete [11]. The actual concrete strength was assessed during the experimental preparation of the integrity test using non-destructive testing [16], while the measured steel properties were obtained during building construction.

Table 5
Material properties of tested elements, [11] and [16]

Material	Ultimate Stress (MPa)		Yield Stress (MPa)		Compressive Strength (MPa)		Tensile Strength (MPa)	
	Nominal	Measur	Nominal	Measur	Nominal $f_{c,k}$	Measur $f_{ck,\,cube}$	Nominal	Measur
Steel								
S275	430	469	275	303	—	—	—	—
S355	510	544	355	396	—	—	—	—
Plate								
Grade 43	430	—	275	—	—	—	—	—
Bolts								
8.8	800	869	640	—	—	—	—	—
Concrete								
LW 35/40	—	—	—	—	35	39,39 ±0,48	3.2	—
Reinforc.								
A142 mesh: T6@200 mm	—	—	—	—	—	—	—	—

Fig. 9. Compartment after fire, residual deformation 925 mm, no local collapse of structure.
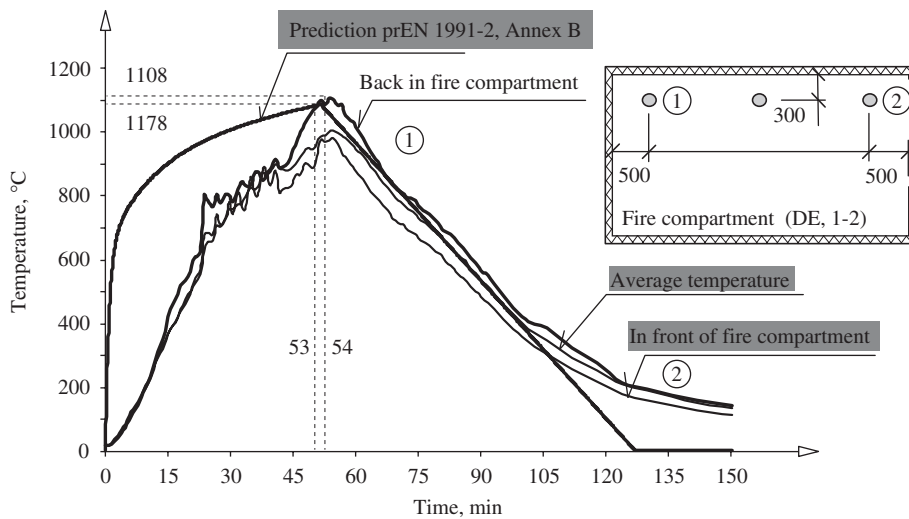
Fig. 10. Compartment temperature.

4. Experimental observations

4.1. Fire development and compartment temperature

During the test the predicted local collapse of the structure was not reached, see Fig. 9. Fig. 10 compares the temperatures recorded in the compartment with the parametric curve presented in Eurocode 1, Annex A [19]. The quantity of thermal load and the dimensions of the opening on the facade wall were designed to achieve a representative fire in the office building. The maximum recorded compartment temperature near the wall (2 250 mm from D2) was 1107.8 °C after 54 min,
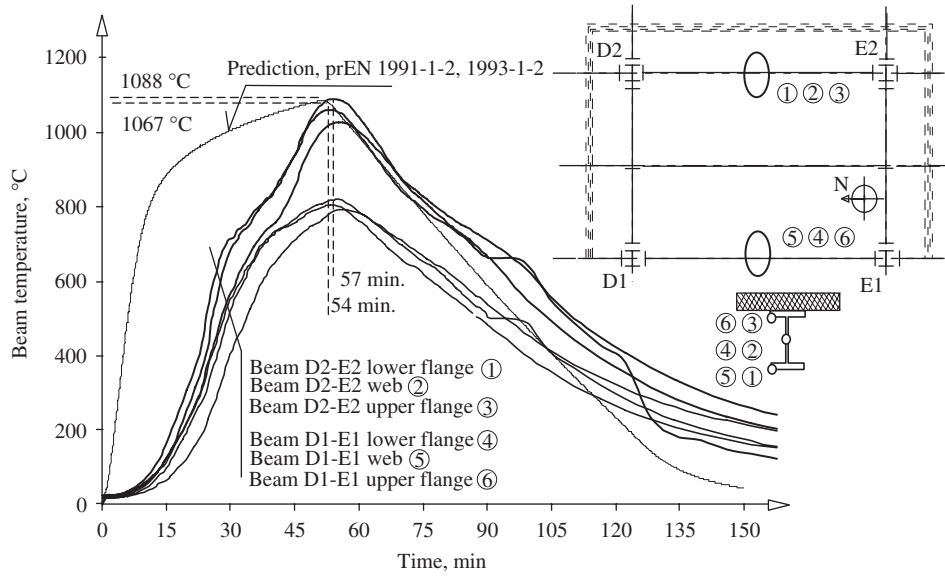
Fig. 11. Differential relative temperature variation within the beams D1–E1; D2–E2.
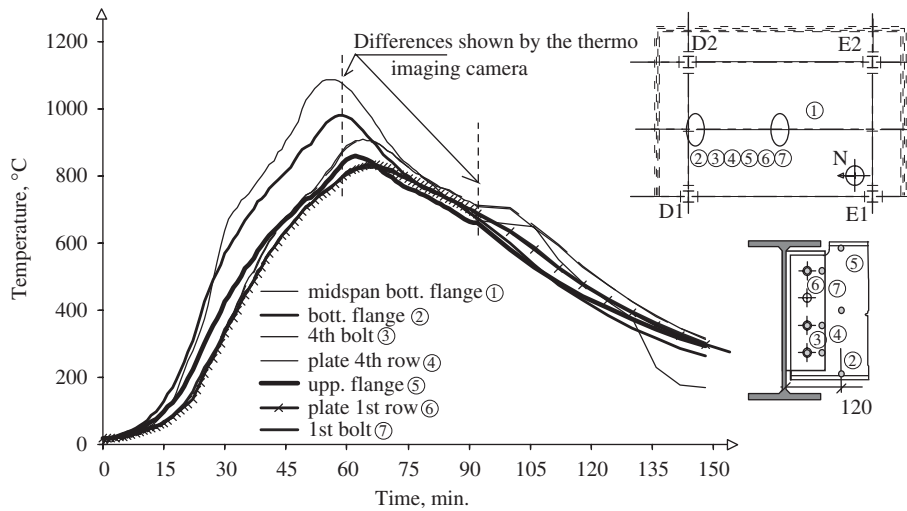
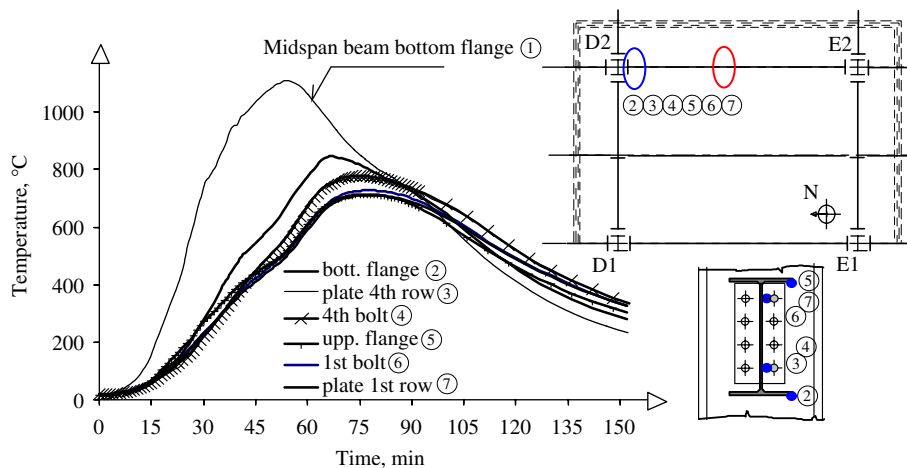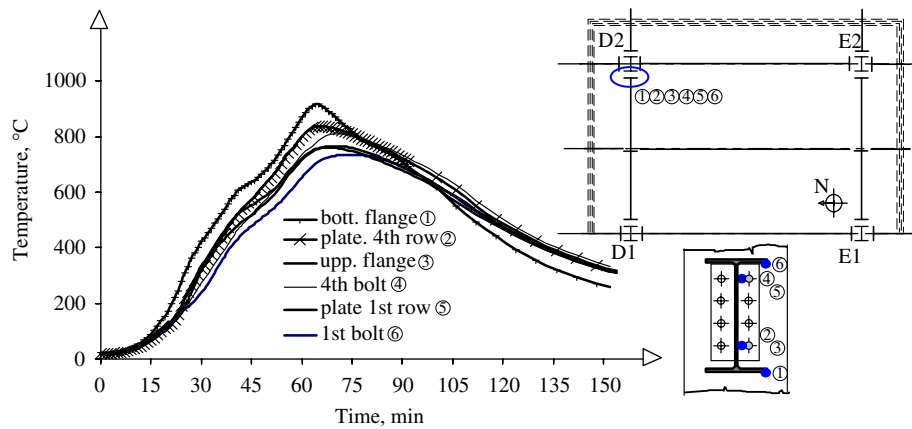Fig. 12. Differential relative temperature variation within the beam-to-beam fin plate connection D1/2–E1/2.

Fig. 13. Differential relative temperature variation within the beam-to-column minor axes end plate connection D2–E2.

while the predicted temperature was 1078 °C in 53 min, see [16].

4.2. Temperature evolution in the structure

Measurements of the temperature in the mid-span beams were taken on the bottom flange, on the web and on the upper flange. A summary of the temperatures recorded in the beams is presented in Fig. 11. The maximum recorded steel temperature of 1087.5 °C occurred after 57 min of fire, on the bottom flange of beam DE2 in the middle of the section. By calculation, using a step by step procedure for

the heat transfer into the unprotected steel structure, see [20, (Eq. 4.24)] and with the help of the time–temperature curve measured on the compartment, a maximum steel temperature of 1067 °C in 54 min was predicted.

Measurements of the temperature in the connections were taken on the beam adjacent to the connection, in the plate and in the bolts. A summary of the temperatures recorded in the connections is presented in Figs. 12–14.

In the heating phase, the joint temperature is significantly lower than the remote bottom flange, which is usually the critical element that defines the limiting temperature of the beam; in contrast, the cooling down

Fig. 14. Differential relative temperature variation within the beam-to-column major axes end plate connection D2–D1.
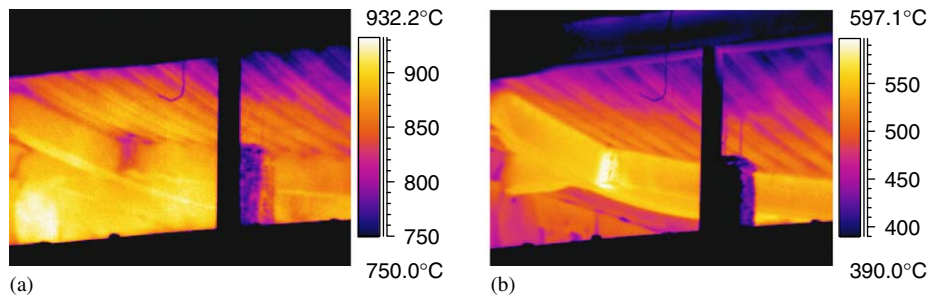

Fig. 15. Fin plate connection D1–2 recorded by thermo imaging camera (a) during heating after 58 min of fire; (b) during cooling after 92 min.
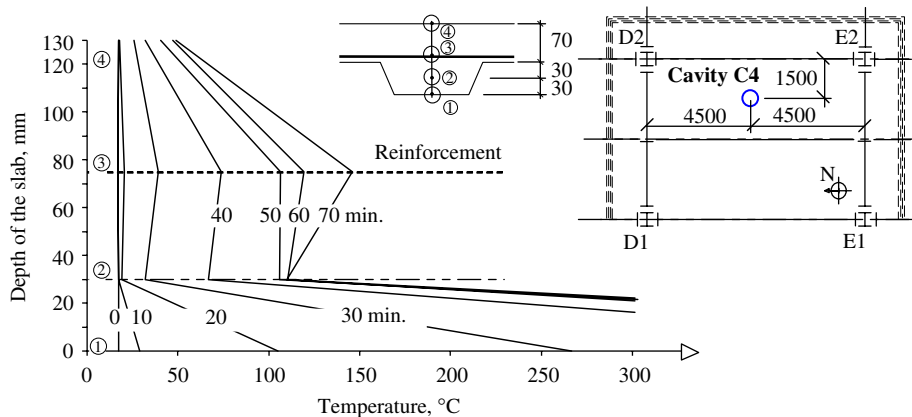

Fig. 16. Differential relative temperature variation within slab over the rib, cavity C4.

in the joints was slower. Using the thermal cameras it was possible to observe this effect, see Fig. 15.

The maximum joint temperature was around 200 °C lower than the maximum temperature of the beam. The first bolt row from the top was significantly cooler than the lower bolts, because of shielding by the adjacent slab and

column. The end-plate was hotter than the bolts at the same level due to the ratio of the bolt diameter to the end-plate thickness (20 mm).

A summary of the temperatures recorded in the slab is presented in Fig. 16 for temperatures in the reinforcement over the rib.
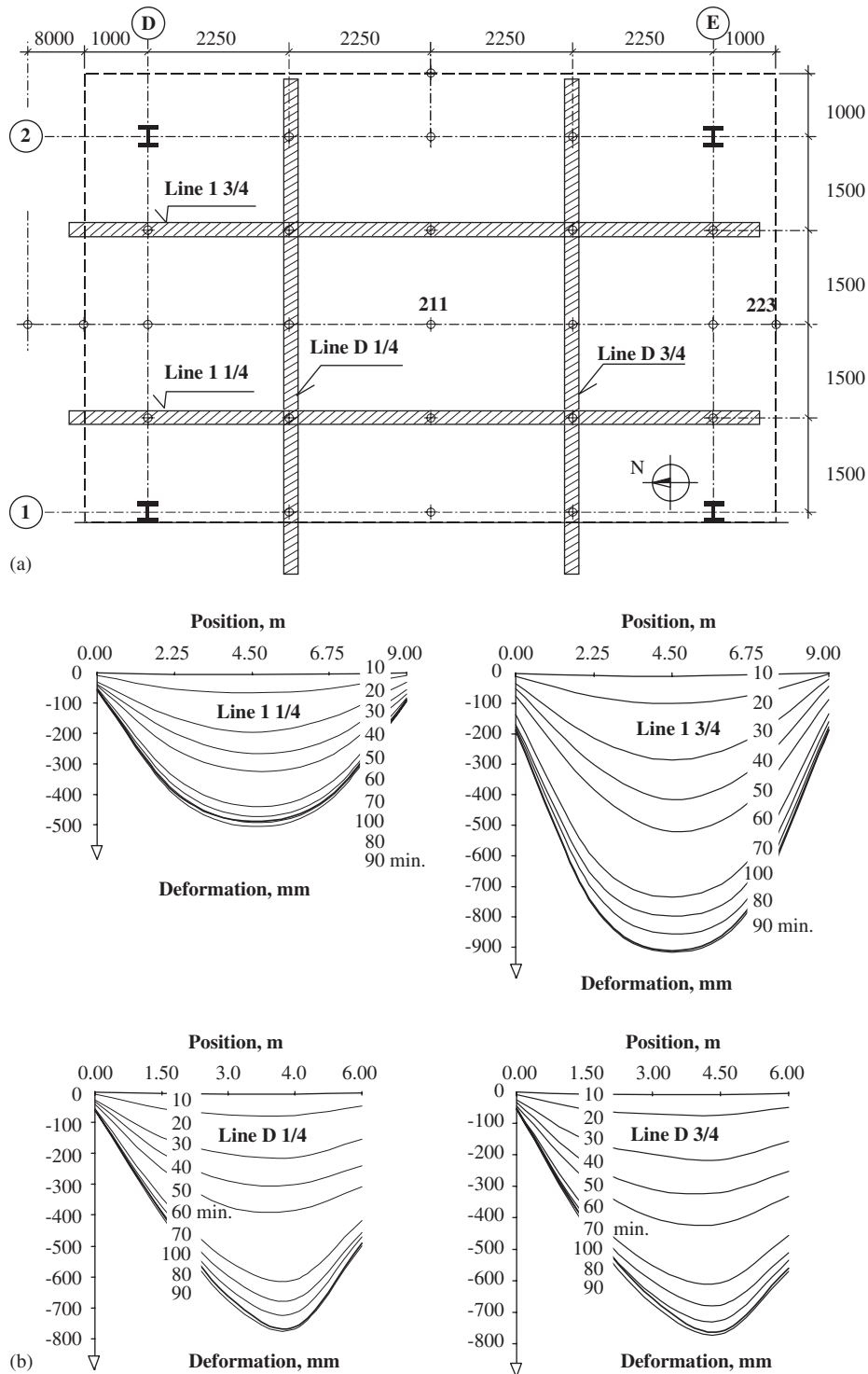
Fig. 17. Vertical deformations; (a) measured points, (b) reached values at drawn lines, * deflection more then 900 mm may be affected by the limited record in transducers.

4.3. Behaviour of the structural members

The maximum deflections were not recorded by the displacement transducers because its range was limited to 1000 mm minus the initial offset. From the video cameras on the 5th floor it is possible to recalculate a maximum vertical displacement of about 1200 mm. Fig. 17 shows the vertical displacement recorded in beams D1–E1, D1/2–E1/2, D2–E2. Comparing the different secondary beams, it is observed that during the heating phase, the beam with a lower displacement is the beam near the window, because of lower temperatures, while the beam near the internal wall shows the biggest displacement. In the cooling phase, both these beams partially recovered.

Local buckling of the beam lower flange was one of the main failure mechanisms. It is observed in the lower beam flange and web adjacent to the joints, see Fig. 18, the concrete slab having restrained the upper flange. This local buckling occurs during the heating phase after about 23 min of fire (observed by thermo imaging camera), due to the restraint to thermal elongation provided by the adjacent cooler structure and the structural continuity of the test frame. The heated lower flange of the beam is unable to transmit the high normal forces generated in the lower flange of the beam to the adjacent beams/columns after closure of the gap in the lower part of the connections. The beam could be assumed to behave as 'simply supported', allowing larger mid-span deflections to develop. This phenomenon was numerically predicted by Lamont and Usmani [21].

As temperature and the associated deformations increase, shear buckling of the beam web was observed, see Fig. 19.
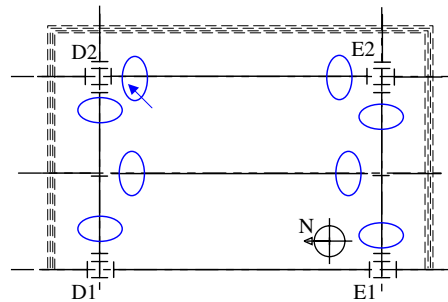

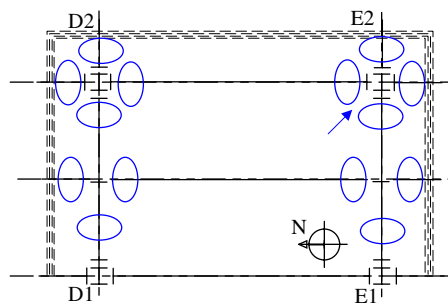Fig. 18. Local buckling of beam lower flange.

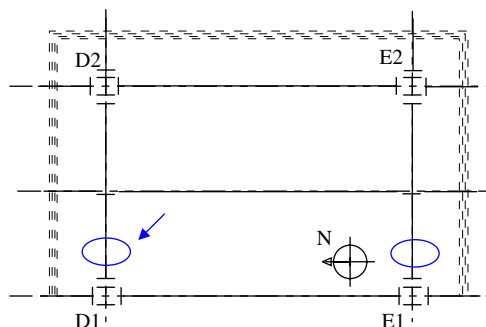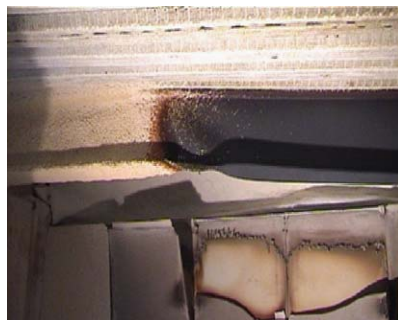

Fig. 19. Beam web in shear.

Fig. 20. Plastic hinge in the unprotected beam cross-section close to the end of the fire protection.

The formation of a plastic hinge in the beam cross-section next to the protected zone was one of the main observed mechanisms in the main beam D1/2, see Fig. 20. This hinge is induced by distortional buckling during the first stage of the heating phase, due to the restraint to thermal elongation provided by the adjacent protected section. This behaviour is associated with the local loss of stability in the bottom flange. Subsequently, during the second stage of the heating phase, the beam rotates around this point due to the large mid-span deflection.
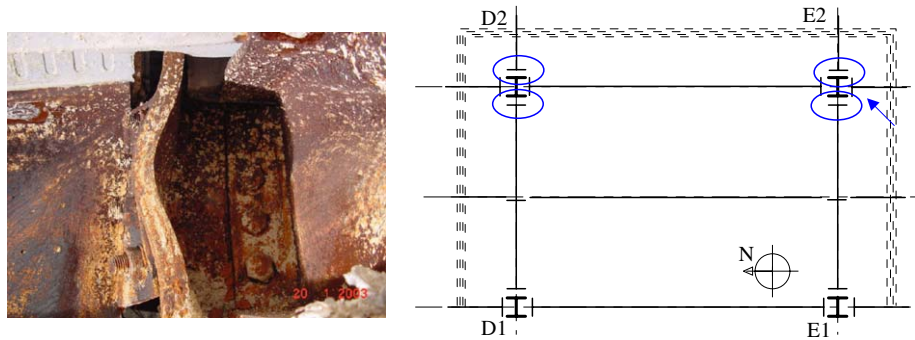

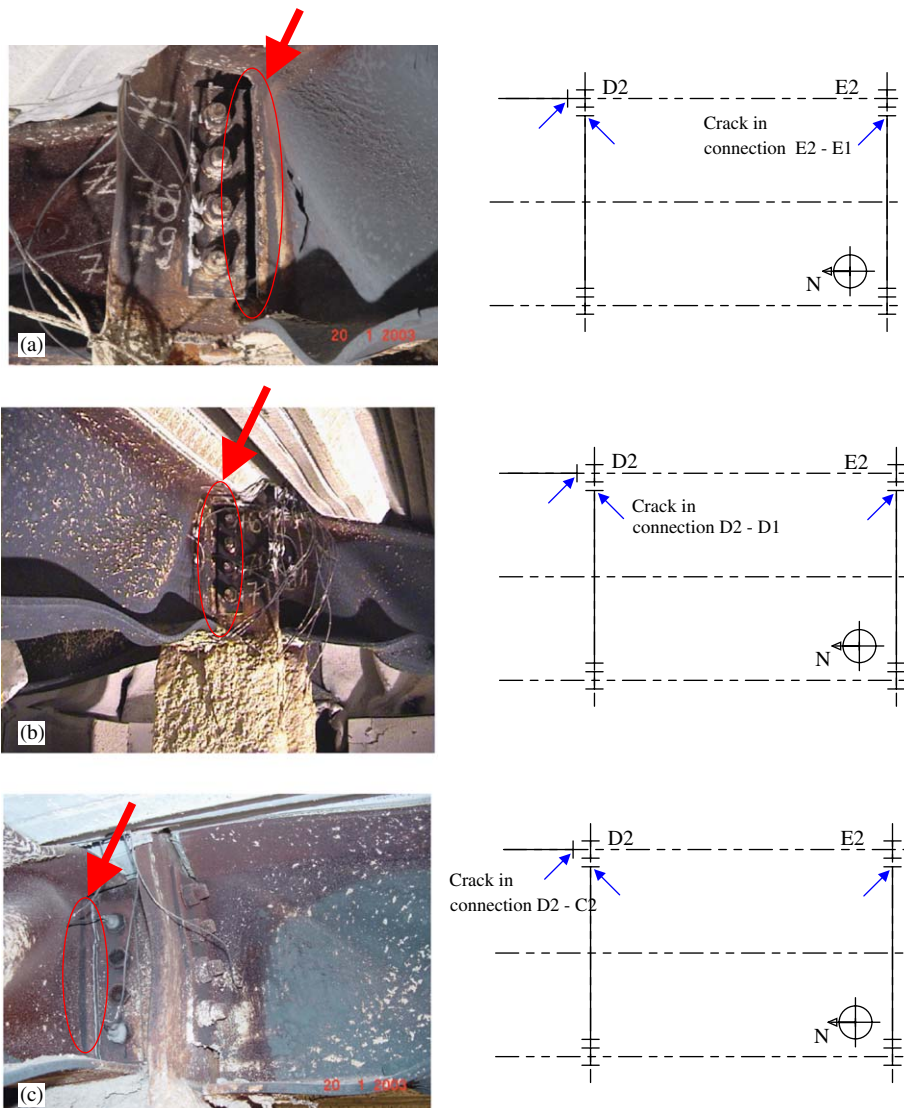Fig. 21. Buckling of column flange in compression, column E2.

Fig. 22. Fracture of the end-plate along the welds, connections D2–D1; E2–E1, D2–C2.
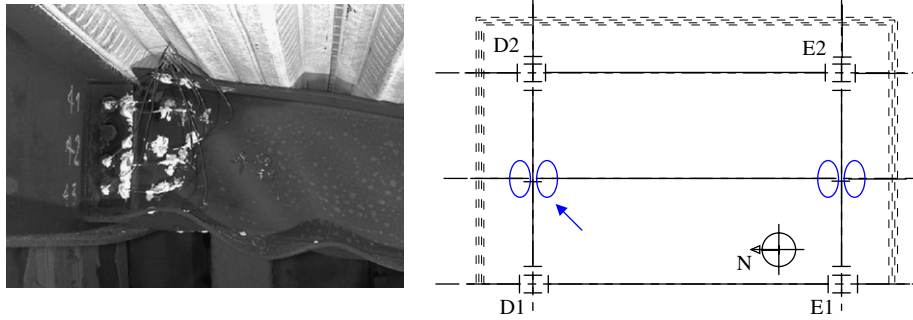
Fig. 23. Elongation of holes in the beam web in fin plate connection.

Due to the end-plate loading, bending in the column flange was observed in one of the major axis beam-to-column joints, see Fig. 21. This behaviour results from the small column flange thickness ($t = 21.4$ mm) and the small distance between the bolts, the bolted end-plate behaving as a welded joint. This behaviour was observed in both columns flanges of the two beam-to-column joints (D2; E2).

4.4. Behaviour of the connections

Fracture of the end-plate along the welds was observed, caused by the horizontal tensile forces during cooling of the connected beam under large rotations associated with flexible end-plate joints, see Fig. 22. The fracture occurred along one side of the connection only, while the other side remained intact. After one side has fractured, the increased flexibility allowed larger deformations without further fracture. This behaviour was observed in both the major axis beam-to-column joints (D2–D1; E2–E1) and the minor axis beam-to-column joints (D2–C2).

The elongation of the holes in the beam web in the tension/compression part of the fin plate connection is due to the associated large rotations, see Fig. 23. The elongation of the holes occurred on the web of connected beam, while the fin-plate remained intact: the beam web thickness (6 mm) is smaller than the fin-plate (10 mm); again, the elongation of the holes of the beam web leads to increased joint flexibility, allowing larger deformations without further fracture.

4.5. Composite slab behaviour

Fracture in the concrete slab was observed, a large crack propagating from the face of the column flange parallel to the beam (D2–E2), see Fig. 24. This crack developed due to the tension in the concrete slab, along the weak zone in the composite beam–flange extremity. After the concrete cracked, the joint stiffness gradually decreased. Secondary cracks occurred perpendicular to, and continuous across, the connections on both sides of the slab, see Fig. 25. The maximum vertical displacement occurred along a line of mesh reinforcement overlap (without adequate attachment). This led to slippage of the mesh and the
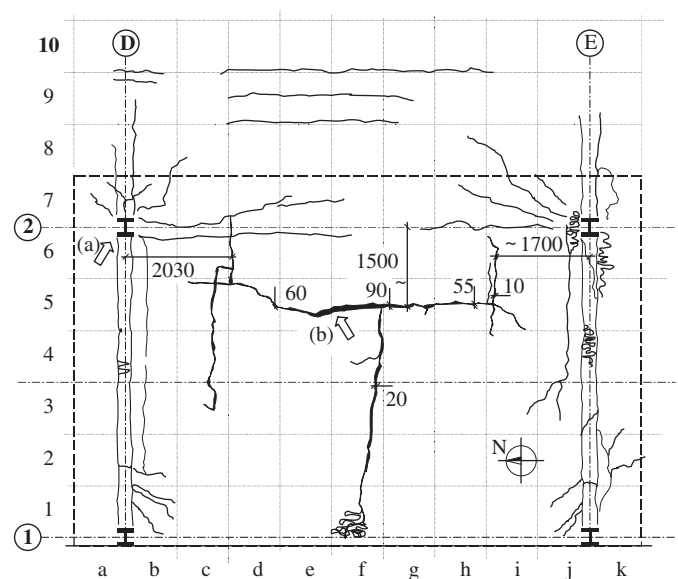

Fig. 24. Mesh of cracks in concrete slab.

corresponding behaviour of the slab was "de facto" without reinforcement.

The cameras above the fire compartment recorded loss of the integrity limit state of the concrete slab after 54 min. The opening of the composite slab took place around column E2 by a punching mechanism due to the tension in the concrete slab in the edge compartment. Furthermore, many tiny cracks were observed in different areas of the concrete slab.

5. Conclusions

Collapse of structure was not reached, thus allowing to demonstrate that the structure has adequate fire safety for the fire load of 40 kg/m^2 of wood cribs, which represents a design fire load in a typical office building, coupled with a mechanical load in excess of typical serviceability conditions. The test results fully supported the concept of unprotected beams and connections with protected columns as a viable system for composite floors [18].

The local buckling of the lower flanges of beams was observed after 23 min of fire. Fracture of the end plates
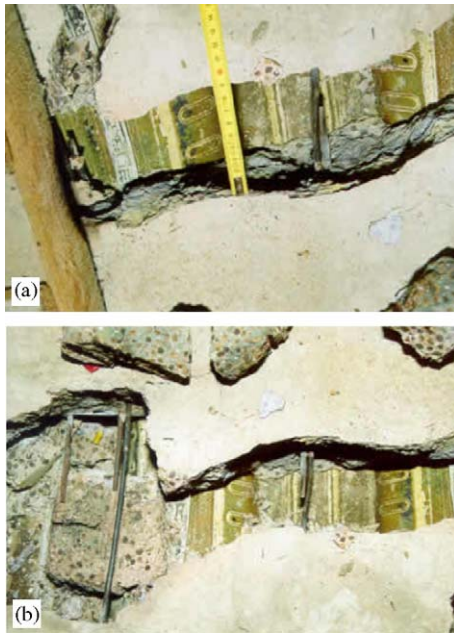
Fig. 25. Reinforcement mesh (a) slippage and cracking near to the column D2, (b) largest crack/opening in the mid-span.

occurred under cooling in the heat affected zones of welds without losing the shear capacity of the connections. The fin plate connections behaved in a ductile fashion due to elongation of holes in bearing.

This test shows the conservatism of the Eurocode fire design [19,20]. The calculated values show good and conservative predictions of the temperature in the fire compartment, the transfer of heat into the structure and connections and the prediction of structural behaviour.

The detailed behaviour of the composite slab, connections and columns is currently being investigated to refine the analytical and numerical prediction models, preliminary published results being available in [22,23]. Further experimental tests on subassemblies using the boundary conditions measured on the Cardington frame test are currently being prepared to further explore the behaviour of connections, columns and beams.

Acknowledgement

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