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Behaviour of a concrete structure in a real compartment fire

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In July 2006, a full-scale compartment fire was set in an existing block of flats in Dalmarnock, Glasgow, Scotland. Prior to ignition, the structure was instrumented with deflection gauges, thermocouples and strain gauges. The growth of the fire was also carefully monitored. The resulting data provided a unique record of the behaviour of a concrete structure in fire through a heating–cooling cycle. The results show significant variation in structural temperatures within a relatively small compartment, demonstrating that the assumption of uniform temperature at any level within a fire compartment, which is implicit in many simple design fires, is incorrect. The response of the structure showed a corresponding complexity due to the combination of non-uniform spatial and temporal heating and the structural boundary conditions.

1. Introduction

This paper describes the structural aspects of a fire test on a full-scale, cast-in-situ concrete structure carried out at 4 Millerfield Place, Dalmarnock, Scotland on 25 July 2006. This paper is one of several reporting on a range of experiments carried out during July 2006 at Dalmarnock (Rein *et al.*, 2007; Stratford *et al.*, 2009).

The conventional, and widely used, method of testing structures for fire resistance is to subject single structural elements to a standard fire test (e.g. ISO 834 (ISO, 1975)) and thus obtain a fire resistance rating in the form of a time to failure. This approach has been severely criticised in a number of ways, such as those discussed by Drysdale (1998), but is still widely used. From a structural engineering point of view, one of the most serious criticisms is that the manner and time to failure of a single structural element in a furnace test bears little relation to the time to failure of a complete structural system. Furnace tests therefore do not provide the information needed to undertake performance-based designs of structures for resisting fire. Over the past 10 years, a considerable amount of work has aimed at understanding the global behaviour of steel and steel–concrete composite structures in fire (e.g. Bailey and Moore, 2000; Elghazouli *et al.*, 2000; Gillie *et al.*, 2001; Huang *et al.*, 2000). This work has included the gathering of experimental data and analysis of that information. As a result, the knowledge and computational tools available are now sufficient for performance-based designs of this kind of structure to be undertaken with confidence (e.g. Arup Fire, 2003).

By contrast, the understanding of the global behaviour of concrete structures has received relatively little attention. This is in part due to the perception that concrete structures perform well in fire, in part due to the lack of experimental data on complete concrete structures in fire and in part due to the difficulties associated with numerical modelling of concrete structures. Fire tests on complete concrete structures are infrequent. Prior to the Dalmarnock test discussed here, the most complete set of test data available on the structural behaviour of a heated concrete structure was that produced by a fire test on a reinforced concrete frame at Cardington, UK (Bailey, 2002; Canisius *et al.*, 2003). Unfortunately, this test suffered from instrumentation failure prior to its end and so the dataset is incomplete. More recently, a number of tests have been conducted on model-scale concrete slabs with the aim of verifying design methods for composite structures in fire (Bailey and Toh, 2006). While providing valuable data, these tests were still of a single-element type and the fire exposure was artificial rather than from a natural fire.

The test described in this paper is believed to be the first fire test on a concrete structure in which all of the following applied.

(*a*) The fire load was ‘real’, resulting from office furniture being burnt rather than from burning wooden cribs or gas.
(*b*) Both the fire and structural behaviour were monitored and well documented.
(*c*) The structure was a complete building rather than a structural element or set of elements.
(*d*) Data relating to the fire behaviour and structural behaviour were recorded during both the heating and cooling phases of the fire.

The test has therefore fulfilled two purposes. First, it has provided experimental evidence of the behaviour of a concrete structure subject to a realistic compartment fire. Second, and perhaps more importantly in the long term, it has produced a set of data that may be used to benchmark numerical and theoretical models of the behaviour of concrete structures in fire. To date, it has been difficult to verify models of the interaction between fires and structures with confidence. In particular, the nature of heat transfer into a concrete structure from a fire and the structural response of concrete structures to fire have been areas of considerable uncertainly due to the lack of experimental data. It is hoped that the results presented here provide the means to reduce uncertainties in these areas.

2. The test arrangement

2.1 The Dalmarnock structure

The structure in which the fire test was carried out was built in 1964 as a residential tower block. The 23-storey building was made of cast-in-situ reinforced concrete and each floor contained six flats. Figure 1 gives a general view of the structure. The fire test was conducted in a flat on the fourth floor and the fire compartment was in the living room of the flat. A plan view of the flat, based on an extract from the blueprints of the structure and indicating aspects of the test set-up, is shown in Figure 2. Details of the living room, including key dimensions, are indicated in Figure 3. The figure also shows the x–y coordinate system and north direction used throughout the tests. The floor to ceiling height was 2·4 m throughout the flat.

Figure 1. 4 Millerfield Place, Dalmarnock, the location of the fire test

The living room was located in one corner of the structure and so had two external walls. The shorter (west) external wall was load bearing. The second (north) wall, however, was load bearing along only part of its length, but contained two structural columns. Of the internal walls, the southern wall was structural, but the eastern wall was a lightweight partition wall dividing the fire compartment from the kitchen of the flat; this wall performed no structural function.

The performance of the ceiling slab of the fire compartment was of particular structural interest during the tests. The details of the support conditions of this slab (which also formed the floor of the flat above) were quite complex, as can be appreciated from the above discussion and Figure 3.

The nominal thickness of the ceiling slab was 150 mm. A ground-penetrating radar (GPR) survey of the slab was conducted prior to the fire. This indicated continuous 8 mm diameter mesh smooth reinforcement on the lower surface over the entire slab area, additional 14 mm diameter twisted bar reinforcement near the top surface adjacent to the external support and further 14 mm diameter twisted bar reinforcement near the top and bottom surfaces adjacent to the internal support. The reinforcement diameters were confirmed by coring the slab after the fire. Figure 4 shows the reinforcement along a north–south section (marked A–B in Figure 3). The reinforcement was continuous between floor slabs and the vertical load-carrying members.

Cores were taken from floor slabs in two adjacent parts of the structure and tested in compression using standard test cylinder test methods given in BS EN 12390-3 (BSI, 2002a). Prior to the fire test, 70 mm diameter cores were taken from an adjacent flat on the same floor, but these gave inconsistent and low strengths due to problems associated with removing a complete core. After the test, 90 mm diameter cores were taken from an adjacent room in the same flat, which was not heated. These gave a compressive cylinder strength of 30 N/mm^2, in line with what would be expected from concrete in a 40-year-old residential building.

2.2 Fire load, ventilation and fire behaviour

The fire load in the compartment consisted of office furnishing, with the main fire load coming from a sofa placed in the middle of the compartment. Other items of furniture included bookshelves filled with paper, a desk, carpet, a computer and a chair. Figure 5 gives an overview of the compartment furnishings, with the sofa partly visible in the bottom right-hand side of the photograph. The fire was started by the ignition of a waste paper basket next to the sofa. Further details of the fire load can be found in Rein *et al.* (2007).

Ventilation to the fire compartment was provided by several sources that varied during the test. At ignition, the doors to the rest of the flat and the kitchen were open (Figure 2) while the windows in the fire compartment and kitchen were both closed. The intention was that the doors to all the other flats on the floor

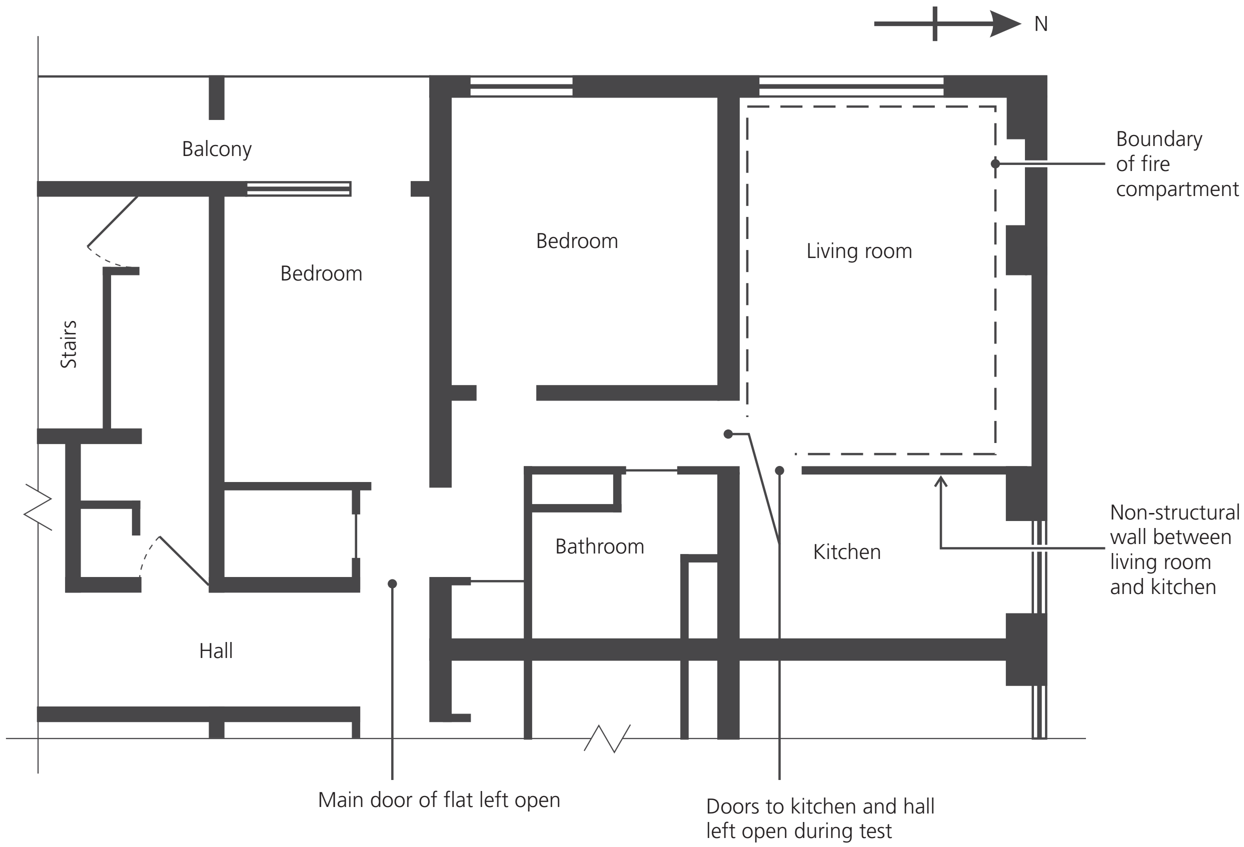


Figure 2. Plan of the Dalmarnock tower showing the layout of the flat in which the fire test took place (traced from the blueprints) (approximately to scale)

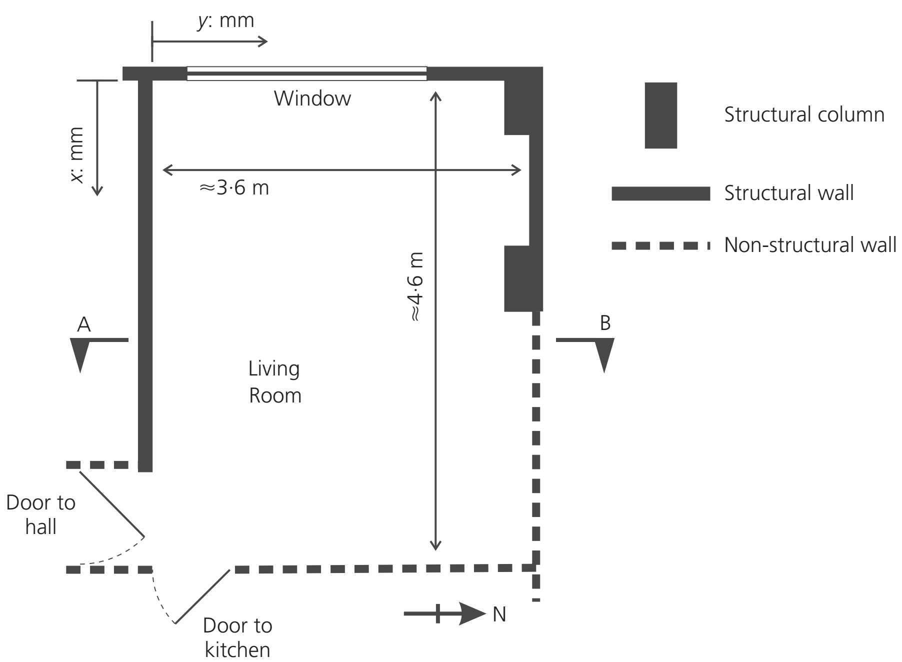
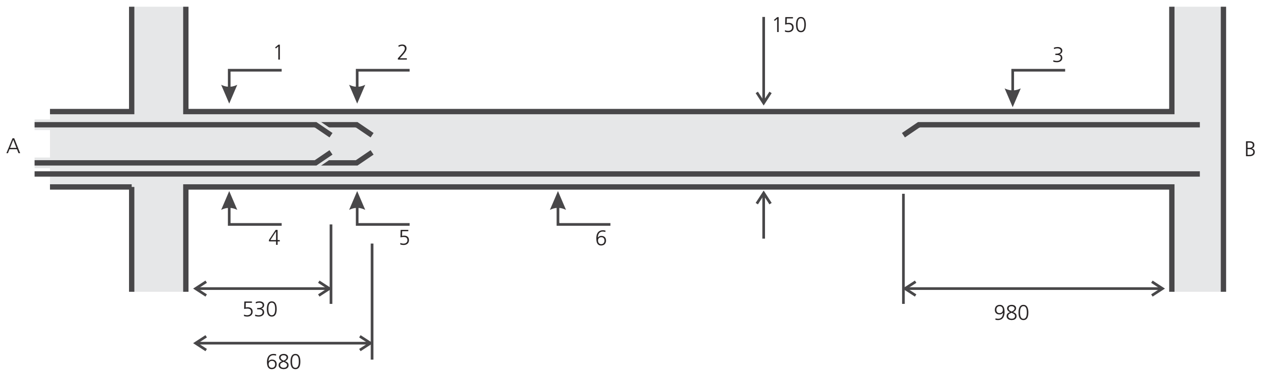


Figure 3. Plan of the fire compartment, in the living room of the flat



Bar mark	Reinforcement	Cover: mm	Spacing: mm
1	14 mm dia. twisted bar	40	300
2	14 mm dia. twisted bar	40	300
3	14 mm dia. twisted bar	40	300
4	14 mm dia. twisted bar	40	300
5	14 mm dia. twisted bar	40	300
6	8 mm dia. smooth bar	6	80

Figure 4. Section through the ceiling slab showing the reinforcement layout (the location of the section is marked A–B on Figure 3) (dimensions in mm)

Figure 5. Photograph of the fire compartment showing the fuel loading (looking north–west)

where the fire took place would be closed. However, due to an oversight, this was not the case: doors to other flats on the floor were open at ignition and provided ventilation to the fire via the hall. As a consequence, when flashover occurred it occurred 'into' the fire compartment without any window breakage. As the intention had been for the fire to flashover 'out' of the compartment, a stone was thrown to break the living room window and thus provide ventilation from this source. The window in the kitchen broke of its own accord during the fire. Key events during the fire and the times at which they occurred are listed in Table 1.

Laboratory tests of a sofa identical to that in the fire compartment suggest that, prior to flashover, the fire burnt in a t^2 manner and produced a peak heat release rate of 800 kW (Rein *et al.*, 2007).

The ceiling slab above the fire compartment was loaded only by its own self-weight. Due the thickness of the floor slab, in a design scenario, a large proportion of the total loading on the floor at the fire limit state would be dead load so additional live loading was omitted during testing to allow instrumentation to be installed more easily. The deflections recorded during the test were due to the interaction of thermal expansion and the restraining action of the surrounding structure.

2.3 Instrumentation

Data recorded for the structural aspects of the test consisted of deflections of the ceiling and one wall of the fire compartment:

Clock time	Time after ignition (min:s)	Event
12:23:00	0	Ignition
12:23:09	0:09	Cushions ignite
12:23:15	0:15	Researchers and firemen leave compartment; front door closed
12:26:06	3:06	Smoke visible in main corridor
12:27:35	4:35	Bookcase ignites
12:28:00	5:00	Aerosol can explodes
12:28:00–12:28:30	5:00–5:30	Flashover
12:28:15	5:15	Flames project to flat corridor ceiling, low visibility in main corridor
12:28:23	5:23	Ignition of paper lamp and table papers
12:36:21	13:21	Window breakage
12:41:00	18:00	External flaming
12:42:00	19:00	Firemen enter and tackle fire
12:45:00	22:00	Mostly smouldering

Table 1. The timing of key events during the fire test

strains on the upper surface of the ceiling of the compartment and temperatures within the ceiling of the compartment. The majority of the instrumentation was on the upper surface of the ceiling slab, connected to a datalogger in the kitchen of the flat above the fire compartment. The windows and all openings made in the slab were sealed to ensure that hot gases did not enter the flat above the fire compartment and affect the instrumentation.

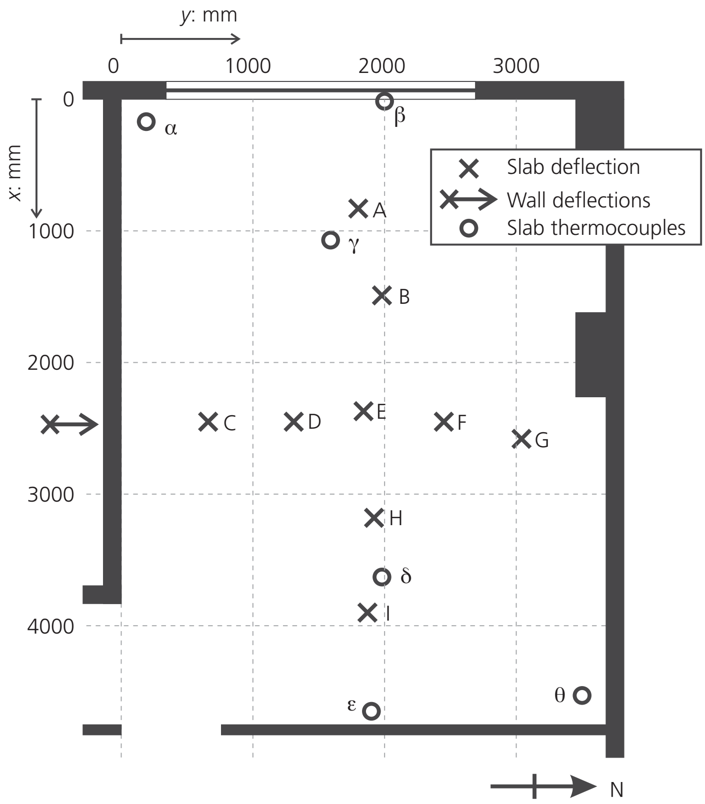
Slab temperature measurements were taken by means of thermocouples at six locations in the ceiling above the fire compartment, shown in Figure 6, where each group of thermocouples is assigned a Greek letter for identification. At each location, four thermocouples were placed at 10, 50, 100 and 150 mm from the bottom surface of the slab. The thermocouples were inserted into the slab by drilling an 18 mm diameter hole through its entire depth, inserting the thermocouples and then filling the remaining space with cementitious grout. Care was taken to ensure that a layer of grout was present between the lowest thermocouple and the fire compartment so that the temperature measurements were those of the slab and not the hot gases.

An array of nine deflection transducers was used to record the deflection of the compartment's ceiling slab, as shown in Figure 6, which also lists the coordinates and letter identifier at each position. The deflection transducers were mounted on two crossing scaffold bars, supported on the edge of the floor slab so that the deflections recorded are changes relative to the edges of the room. Any overall change in the height of floor was not captured and, as noted previously, the east wall adjacent to the kitchen was non-structural and hence could deflect vertically (Figure 3).

The horizontal deflection of the internal structural wall of the fire compartment was monitored using three deflection transducers (Figure 6) positioned a quarter, half and three-quarters of the way up the wall. These transducers were located in the room adjacent to the fire, which filled with hot gases in the later stages of the test. Consequently, although the transducers were protected by insulation material, their readings may not be entirely accurate. The results presented later, however, show that the recorded deflections were negligibly small.

Electrical resistance strain gauges were bonded to the slab at the 22 locations shown in Figure 7. Strain measurement is far from straightforward during a fire test, due to the effects of temperature upon the strain gauges, which by necessity must be at the same temperature as the slab being tested. Gauges 50 mm in length intended for temperatures up to 80°C were used, and the strain results were later corrected for temperature using the manufacturer's calibration data. Although other strain measurement technologies are available, they were impractical within the time frame and budget of the tests.

A large quantity of instrumentation was also installed within the fire compartment to record the progress of the fire (Rein *et al.*, 2007). Gas temperatures in the fire compartment and just outside the window were instrumented with 270 thermocouples, arranged to record the 3D temperature field. Heat flux gauges and air velocity probes were also used to characterise the fire environment. Additionally, to investigate their fire resistance, six strips of fibre-reinforced polymer (FRP) strengthening were bonded to the underside of the ceiling slab, along the lines of strain gauges 2–7 (Figure 7). The FRP strengthening was instrumented with thermocouples and strain gauges, in locations corresponding to those on the top surface of the slab. The FRP results are not given here as the FRP did not act compositely with the concrete slab due to the high-temperature behaviour of the bonding adhesive. Further details of these tests can be found in Stratford *et al.* (2009).



Deflection gauge positions: mm

	x	y
A	1800	830
B	1980	1490
C	660	2450
D	1310	2450
E	1840	2370
F	2450	2450
G	3040	2580
H	1920	3180
I	1870	3900

Thermocouple positions: mm

	x	y
α	170	190
β	15	2000
γ	1070	1590
δ	3630	1980
ε	4650	1900
θ	4530	3500

Figure 6. Locations of the thermocouples within the heated concrete slab and the deflection gauges above it

3. Results

3.1 Gas temperatures

Figure 8 shows the evolution of gas phase temperatures measured within the fire compartment. It plots the mean temperature from 240 thermocouples distributed throughout the volume of the compartment, with the variation in temperature across the compartment indicated by the mean plus/minus one standard deviation curves. The ISO 834 time–temperature curve is included as a benchmark against which the experimental results can be compared. The three main phases of the fire are shown in Figure 8 (corresponding to the events listed in Table 1)

(*a*) the growth period, ending with flashover 5 min after ignition
(*b*) the fully developed fire, during which the ventilation regime changed at around 13 min when the window was broken, and external flaming beginning at 18 min
(*c*) fire extinguished at 19 min by firefighters, after which temperatures decayed.

There was considerable spatial variation in gas temperatures and incident heat fluxes across the surface of the ceiling slab. Broadly, the fire was initially concentrated towards the eastern side of the compartment, due to the sofa and bookcases located in the north-east corner. The fire spread to the front of the compartment after the ventilation changed when the window was broken. Further details can be found in Rein *et al.* (2007).

3.2 Post-fire inspection of the structure

The structure was inspected after the test to give an overview of its performance. Despite the firefighting operations, the fire compartment itself had almost entirely burnt out with little combustible material remaining. The lower (directly heated) surface of the ceiling was found to have shed all of the covering of plaster that was initially present; however, there was no spalling of any kind of the structural concrete. The lack of spalling is likely due to a combination of the concrete being old (and therefore dry), of relatively low strength and only being heated to relatively low temperatures.

There was no evidence of any hot gases entering the room above the fire compartment. The upper surface of the slab contained visible cracks at the locations shown in Figure 9. These cracks generally coincided with the lines at which reinforcement in the upper part of the slab was curtailed (Figure 4). This is unsurprising, due to the step change in flexural capacity at these locations.

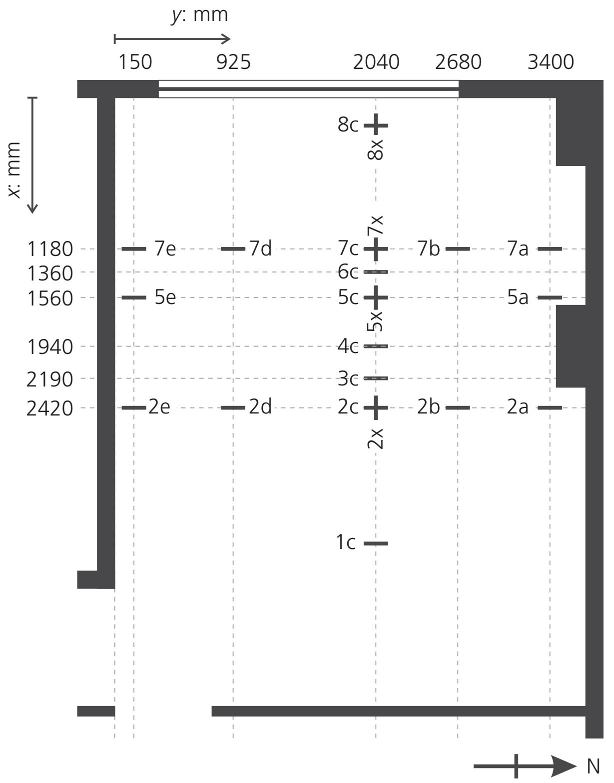


Figure 7. Locations of the strain gauges on the upper surface of the heated concrete slab

3.3 Slab temperatures

Thermocouple readings from within the ceiling slab are plotted in Figure 10, which also shows the position of the individual thermocouples through the depth of the slab at each of the locations in Figure 6. It is clear from these plots that the locations at which highest temperatures occurred within the area of the slab were very localised and that a steep thermal gradient was produced in the slab.

The highest temperatures were recorded at locations δ, ε and θ, away from the compartment window. This was because ventilation to the fire was initially supplied via the doorway to the hall (Figure 2). Of these three locations, δ was above the sofa and hence recorded the highest temperatures. Although θ was above the second largest fuel source (a bookcase heavily laden with combustible material), its location in the north-east corner of the room and adjacent to the cool slab in the kitchen meant that lower temperatures were recorded here than at locations δ or ε.

After the window of the compartment was broken, ventilation to the compartment changed, and there was an increase in the temperatures recorded at location γ. Locations α (in the south-east corner of the room) and β (near the window) remained relatively cool throughout the tests.

The lower surface of the slab was heated much more rapidly and to higher temperatures than the internal part due to the high thermal capacity of the concrete. It is also noticeable that the

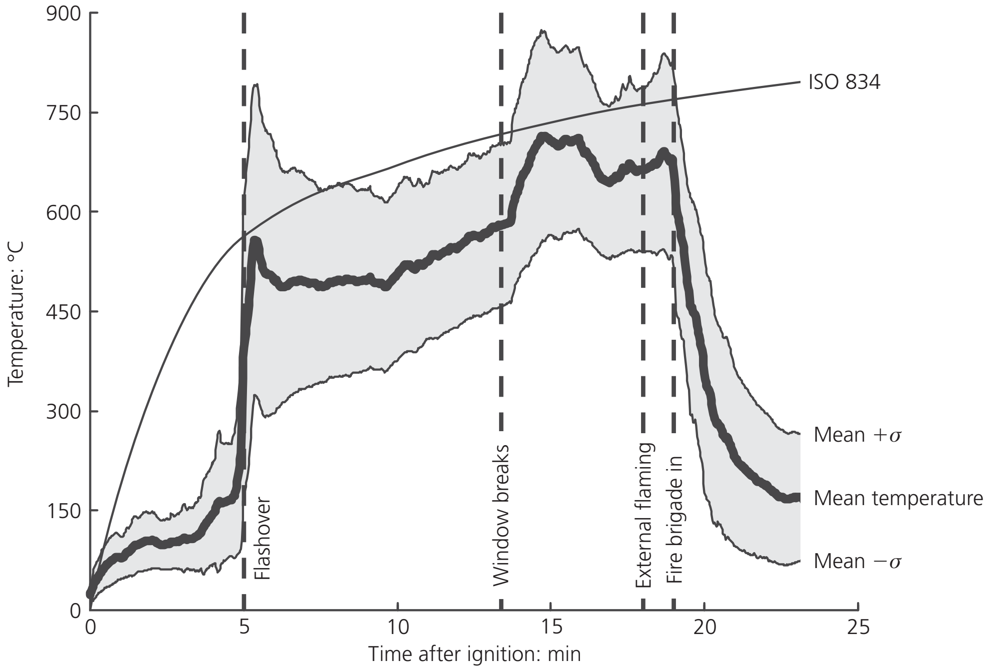


Figure 8. Gas phase temperatures recorded within the fire compartment (mean ±1 standard deviation)

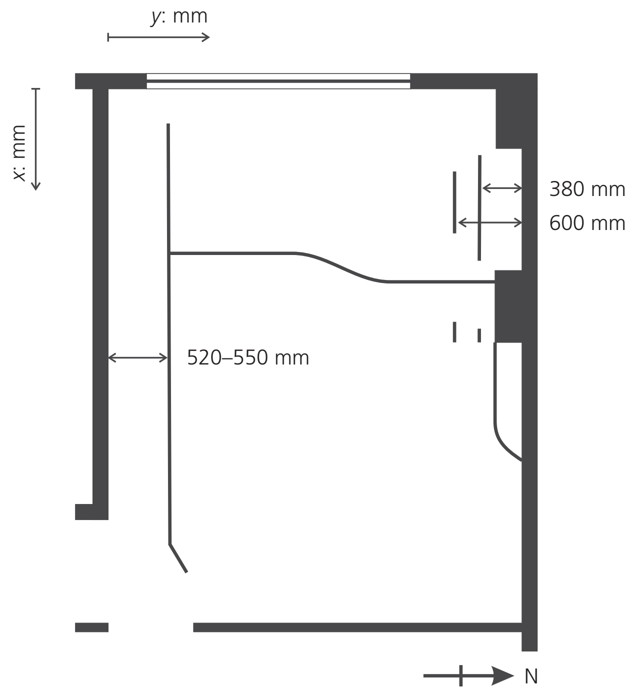


Figure 9. Crack pattern on the upper surface of the ceiling slab after the fire

temperature of the lower surface of the slab dropped rapidly at all locations from around 20 min; this is because firefighters sprayed cold water onto the slab while extinguishing the fire. The internal portions of the slab maintained their temperatures despite fire-fighting activities; indeed, the upper layers of the slab continued to increase in temperature even after the fire was completely extinguished. For example, Figure 11 shows the temperature profile through the depth of the slab at location γ.

The localised nature of the heating is significant as it is almost always assumed when performing design calculations to determine the behaviour of heated structural elements that there is no variation of temperature within a compartment. For example, both the ISO standard fire curve given in ISO 834 (ISO, 1975) and Eurocode parametric curves given in BS EN 1991-1-2 (BSI, 2002b) assume uniform compartment temperatures. The results in Figure 10 show that such an assumption does not hold, even for small compartments.

3.4 Slab and wall deflections

Figures 12 and 13 show the vertical deflections of the ceiling slab. Figure 12 plots the deflections along a north–south strip across the slab and Figure 13 plots deflections along an east–west strip across the slab (Figure 6). Each figure shows the changes in deflection with time and the deflection profiles across the slab at significant times during the test. The poor deflection resolution of some of the traces (e.g. trace D) is due to the use of long-travel deflection gauges.

The peak deflection at the centre of the slab was around 10 mm, with 4 mm of this deflection recovered on cooling. Since the fire duration was short and high temperatures only penetrated a short distance into the concrete slab, the majority of the deflections recorded will have resulted from thermal curvature rather than from loss of material stiffness or strength. Across the north–south section (Figure 12), the deflections reduce towards the edges of the slab, as expected. It is notable that gauge C recorded negative (upward) deflections from around 18 min. This corresponds to the formation of a hogging crack in the slab that effectively moved the support from the wall to slightly inside the fire compartment. The crack formed slightly to the south of gauge C (Figure 9). As the fire cooled (Figure 8) but the upper layers of the slab continued to heat up (Figure 11), the deflection of the slab reduced but the hogging crack opened, as shown by the deflection profile at 60 min (Figure 12).

Similar behaviour is seen along the east–west section (Figure 13), where a kink in the deflection profile around gauge B indicates the formation of another hogging crack (Figure 9). As previously noted, the eastern end ($x = 4{\cdot}6$ m) of the scaffold tube used to support these deflection gauges was not a fixed point on the slab and so may itself have deflected.

The very small horizontal deflections recorded for the internal wall of the compartment (Figure 14) show that the heating of this wall was small in comparison with that of the compartment ceiling.

3.5 Slab strains

Figure 15 shows the variation in recorded strains with time. The strain gauges are grouped into five lines of gauges, corresponding to the locations shown in Figure 7. Lines 2, 5 and 7 (Figures 15(a), 15(b) and 15(c)) run north–south, and the gauges are oriented in the same direction. The other two plots in Figure 15 show strain gauges on an east–west strip of the slab, along the centre line of the roof, with the gauges orientated east–west in Figure 15(d) and north–south in Figure 15(e). The figure plots the total strains in the concrete due to the combination of deformation and thermal expansion. To obtain the total strain, the strain gauge data were corrected for temperature effects, to remove

(*a*) the apparent strain due to differential thermal expansion of the strain gauge components and temperature-dependent electrical effects (these corrections were based on the manufacturer's calibration chart, which was valid up to 80°C; Figure 10 shows that this temperature was not reached on the top surface of the slab during the test)
(*b*) the temperature variation of the gauge factor, given by the manufacturer
(*c*) differential thermal expansion between the strain gauge (which had a coefficient of thermal expansion matched to steel) and the concrete (assumed to have a coefficient of thermal expansion of 10 microstrain/°C).

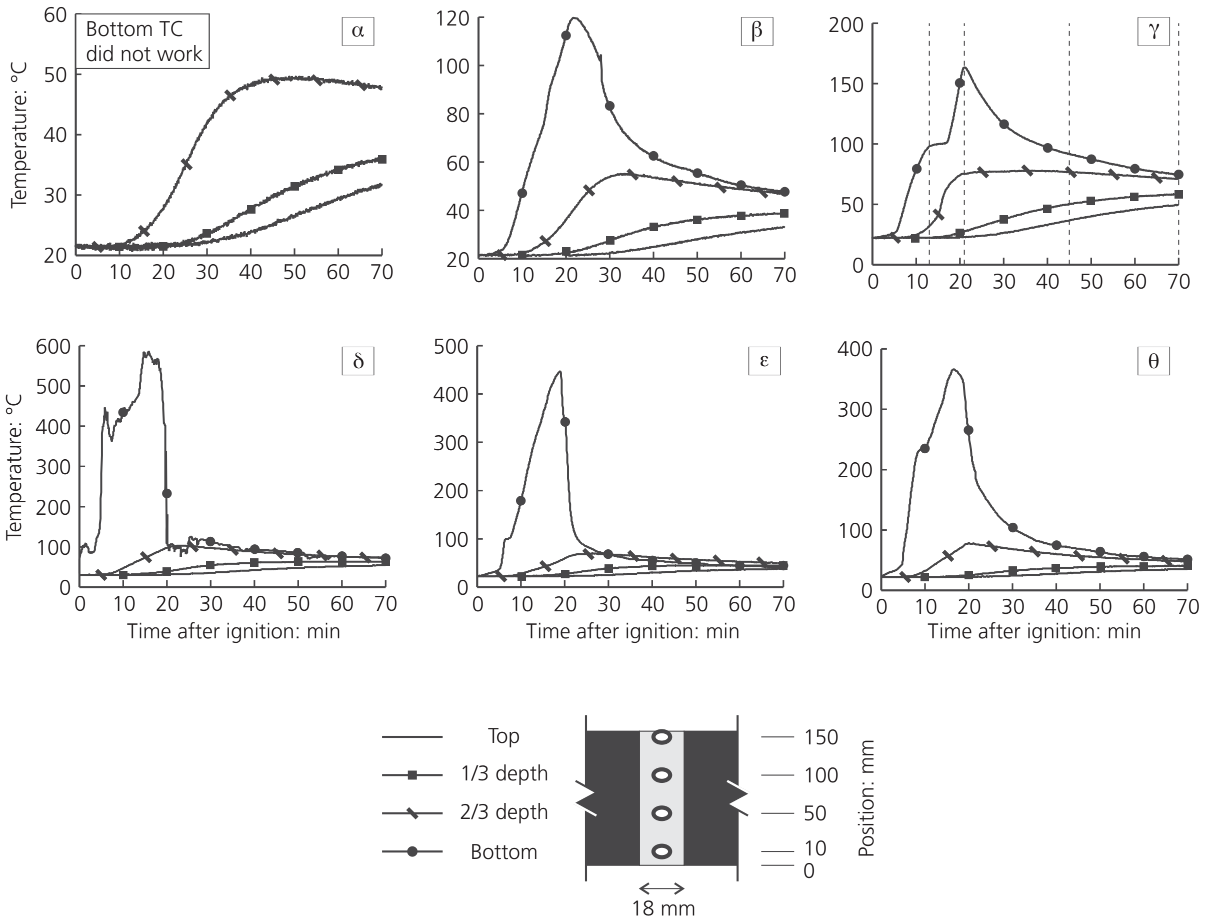


Figure 10. Temperature data recorded at the locations shown in Figure 6 (note the different temperature scales used at each location)

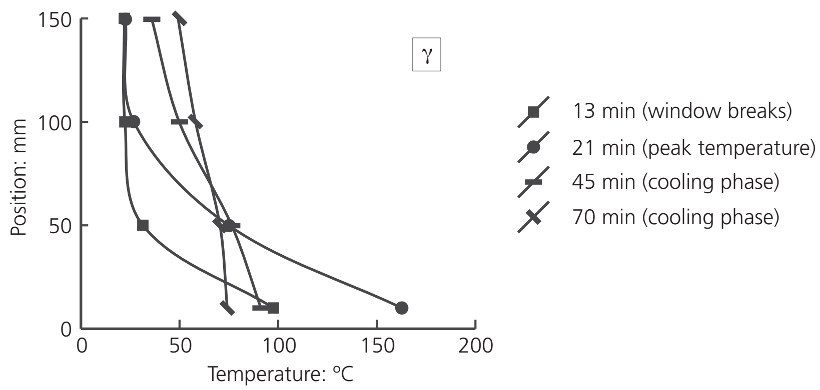


Figure 11. Temperature variation through the depth of the slab at location γ

All of the temperature corrections were based on the temperature recorded at location γ (Figure 6). If the temperature at a strain gauge had in fact been that of another temperature location, a maximum possible error of about 40 microstrain would result. The actual errors in the readings are likely to be substantially smaller than this in most cases because the strain gauges were located near the centre of the compartment where upper-surface temperatures were fairly uniform and similar to those at location γ. The

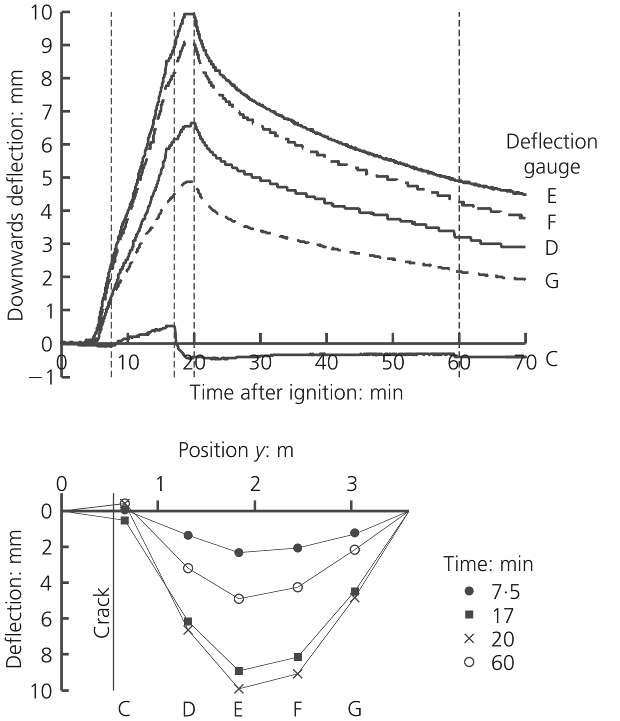


Figure 12. Vertical deflections along a north–south strip of the heated ceiling slab

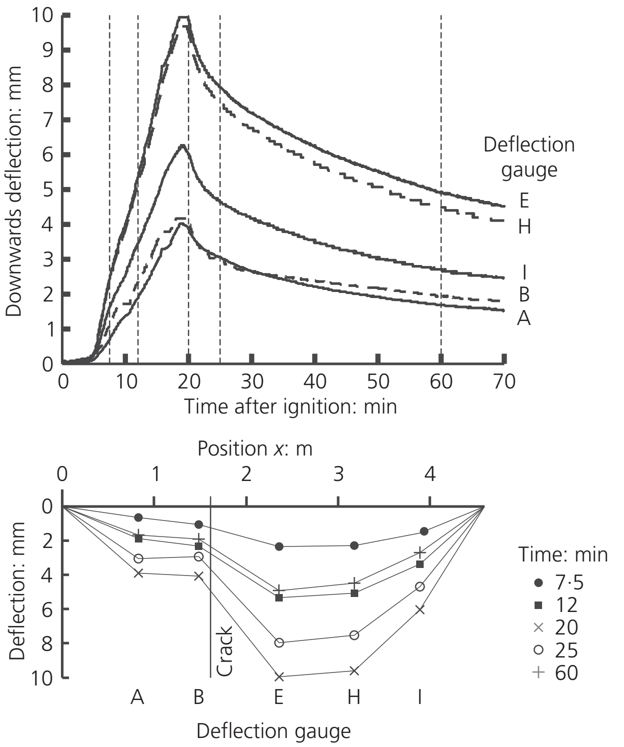


Figure 13. Vertical deflections along an east–west strip of the heated ceiling slab

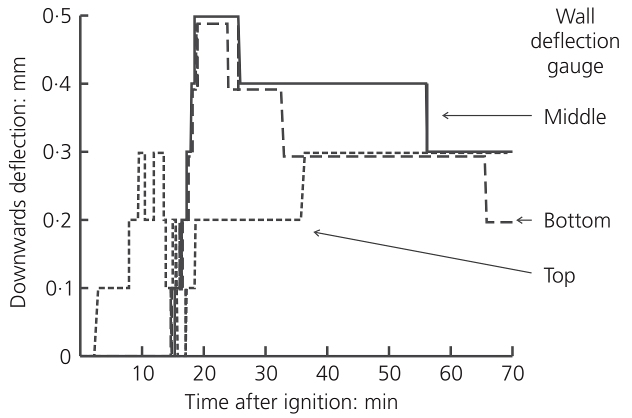


Figure 14. Horizontal deflections of the internal structural wall of the fire compartment

poor time resolution (5·8 min) of the strain measurements in Figure 15 was, unfortunately, not anticipated prior to the test and was a consequence of the very high number of data channels being logged during the test.

The strains in the slab resulted from two phenomena: thermal expansion of the heated concrete and deformation of the surrounding structure. The surrounding structure itself underwent thermal deformation but also restrained the deformation of the heated slab.

During the heating phase of the fire, the strains measured on the top of the slab increased rapidly. Heat from the fire caused the slab to expand, but it was the concrete in the lower parts of the slab that expanded, not the relatively cool upper surface where the strain gauges were located. The large compressive strains recorded on the top of the slab were dominated by geometrical compatibility with the surrounding structure and due to the internal self-equilibrating stresses created due to the thermal profile through the depth of the slab (Figure 11).

After the fire was extinguished at 19 min, the strains decayed rapidly as the bottom surface of the slab was cooled by water applied by the firefighters. However, the temperature in the upper parts of the slab continued to increase as residual heat conducted upwards (Figure 11). As a consequence, the compressive strains in gauges such as 3c and 7c (see Figure 7) again increased after around 30 min, due to the restrained thermal expansion of the top of the slab.

Figure 16 plots the strain profile with position along line 2 at selected time steps. During the heating phase (<19 min), the compressive strains are greater in the centre of the slab, as would be expected in a beam that is heated from underneath and is restrained against rotation and lateral expansion at either end. At 21·2 min (just after the fire was extinguished), there was a sudden jump in strain at gauge 2a and (to a lesser extent) gauge 2e. This

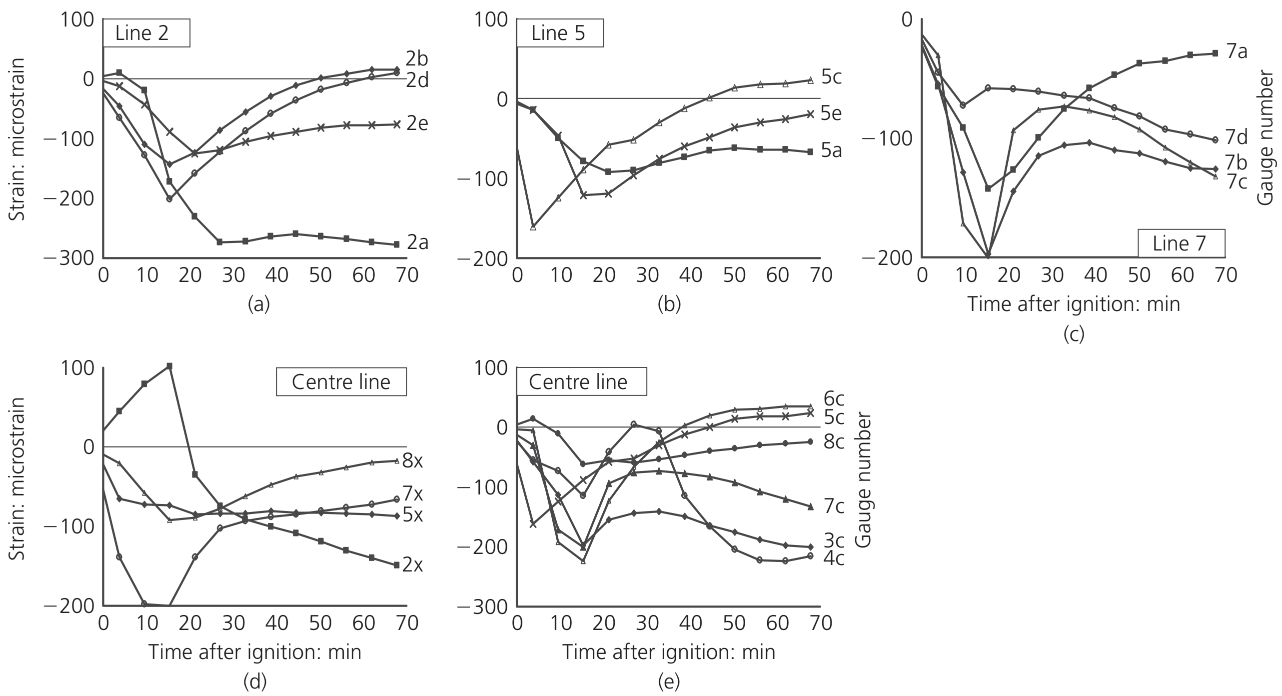


Figure 15. Strain gauge readings from the upper surface of heated slab at the locations indicated in Figure 7

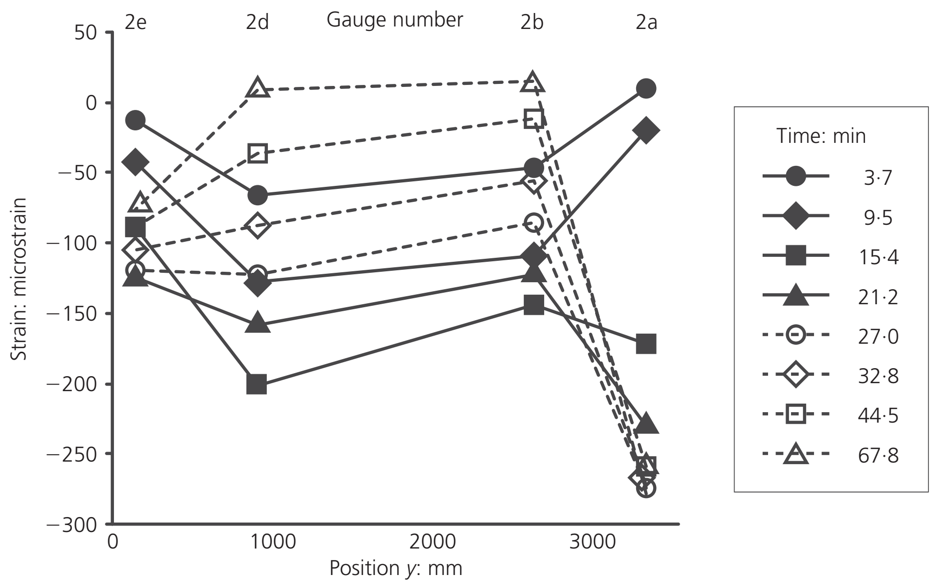


Figure 16. Strain profile with position along line 2 at selected time steps

is believed to be due to the formation of the hogging cracks across line 2 shown in Figure 9, resulting in compression close to the walls as the underside of the slab was cooled. The timing corresponds to the deflection profiles in Figure 12.

Similar general trends can be seen elsewhere in the strain data; however, some details are difficult to interpret. For example, it is not obvious why gauge 2x (orientated east–west) recorded tensile strains early in the test. Such results might at first seem anomalous, but are due to the non-uniform heating, complex boundary conditions and cracks present within the slab. This test was one of the first to measure strains in an existing concrete structure during a realistic fire. In future tests it is recommended that higher resolution strain measurements are made in both time and space so that a full picture is possible of how strain develops within the slab.

4. Discussion

The results highlight a number of points that are worthy of further consideration.

The data show that there will be considerable differences in temperature within a heated concrete structure due to the localised nature of compartment fires. This is in marked contrast to the assumptions made in most design procedures. Temperatures in structural elements are typically based on estimates from either simple 'natural fire' calculations or the results of zone models. Both these techniques assume that gases in a compartment fire are well mixed. While the results from this test show peak temperatures that are comparable to those that would be predicted by standard techniques, the location of these peak temperatures is highly localised.

The complex restrained thermal expansion due to localised heating could result in failure mechanisms that are not considered by current design methods, and which might exceed the available redistribution capacity of the slab. However, the test also suggests that estimates of the energy absorbed by structures in fire are likely to be very conservative if based on standard techniques. As a consequence, considerable savings in fire protection may be possible in performance-based design if methods of estimating structural temperatures take into account the travelling behaviour of fires. This is something that is currently not done.

The cracks that appeared on the cool surface of the heated slab suggest that detailing of reinforcement may have implications for the behaviour of concrete structures in fire. These cracks and the resulting deflected profiles would not be predicted by current design methods. Although no compartmentation failure occurred as a result of cracking in the test considered here, the chances of such a failure would be greater with the longer spans and thinner slabs that are present in many modern buildings. Since the cracks appeared at locations where the flexural stiffness of the slab changed abruptly due to reinforcement curtailment, it would appear that specifying graduated reinforcement curtailment could reduce the chances of cracks forming during a fire.

The spalling behaviour of concrete in fire is the subject of considerable research effort at present (e.g. Hertz and Sorensen, 2005). In general, it is found that fresh (wet) concrete and high-strength concretes are more susceptible to spalling. Given that the concrete in the heated slab was dry (due to its age) and of low strength by modern standards, the lack of spalling in the test described provides anecdotal evidence to support these findings. However, the concrete was also initially covered by a layer of plaster that would have affected its heating rate and would also have prevented direct impingement of flames in the early stages of the test. To date, it appears that the effect of neither of these phenomena on concrete spalling behaviour has been the subject of detailed study.

5. Conclusion

Observations from a full-scale fire test on an existing concrete structure show that the structure performed well in fire, with no structural or compartmentation failure and no spalling of the concrete. The set of data collected shows that the localised nature of compartment fires results in highly non-homogeneous heating of structures – a result at odds with the assumptions usually made in design practice. The collected deflection and strain data show the effect of heating on the slab during the fire, the spread of heat through the depth of the slab after the fire was extinguished and the complex mechanical behaviour that results due to the spatial and temporal variation in heating. The results from the test will allow benchmarking of both computer and analytical models of structural behaviour, heat transfer (combined with the results given by Rein *et al.* (2007)) and material behaviour.

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