Full-scale fire tests on hollowcore floors

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Synopsis
The results from two large-scale fire tests on a hollowcore floor plate, supported on protected steelwork, are presented in this paper. The two tests were identical except for the connection details between the units and supporting steel beams, with Test 2 having a more robust detail to tie the units and beam together. The floor was purposely subjected to a very severe fire created by specifying unrealistically small ventilation openings, compared to modern office construction. The hollowcore floor plate performed very well supporting the full applied static load for the duration of the tests. A beneficial load path mechanism created by lateral thermal restraint to the floor units was highlighted, which has not previously been considered. The tests showed that the small-scale standard fire tests, used to assess fire resistance periods, can be very unrealistic and ignores the beneficial effects of whole building behaviour. The test results presented reinforce the experience gained from real fires that hollowcore floor slabs have good overall inherent fire resistance.

Introduction
The use of prestressed hollowcore floors are very popular in the construction market, accounting for approximately 50% of the market share for steel-framed buildings. The benefits of hollowcore floors include high strength, long spans, durability, immediate working platform, good thermal and sound insulation, generally thinner depths and good interaction with services. It is also generally accepted that hollowcore floors have good fire resistance properties. The design codes BS 8110-1[1], BS EN 1992-1-2[2] and the product code BS EN 1168[3], show that fire resistance periods up to 4h can easily be achieved, provided the tabulated minimum cover to strands and minimum slab thicknesses are specified.

However, some concern has been raised about the actual performance of hollowcore floors in fire following some examples of premature failure due to shear in standard (small-scale) fire resistance tests. A series of tests[4], conducted in a standard test furnace, showing premature failure were conducted by the Danish Institute of Fire Technology in the late 1990s. The tests were carried out on floor units, which had a minimum cover to the reinforcing strands of 25mm. Following the current codified design rules, the slabs should have achieved at least 60 minutes in a standard fire test. However, all tests failed prematurely, by vertical shear, between 21 and 26 minutes into the test. This observation from standard fire tests contradicts experience from real fires in actual buildings where the hollowcore floor system has shown to behave very well.

Van Acker[5] has previously provided an explanation for the vertical shear failure of hollowcore slabs experienced in standard small-scale fire tests. Due to the typical non-linear thermal gradient through a concrete slab, experienced during a fire, and the fact that plane-sections-must-remain-plane, thermal stresses are induced through the cross-section as shown in Fig 1. These thermal stresses comprise compression on the top and bottom of the cross-section and tension within the middle zone. Cracking of the concrete in the tension zone, coupled with possible slippage of the strands at the end of the slab, can lead to premature shear failure as experienced in the Danish tests[4]. Based on this explanation, Van Acker took a pragmatic approach by stating that for real buildings, which use hollowcore floors, premature shear failure is 'unlikely', which corresponds to the experience following real fires. He argues that practical detailing used in normal design, comprising additional tie reinforcement, will ensure that the whole floor plate will act as a coherent diaphragm with any tensile cracks remaining 'closed' and shear being transferred by aggregate interlock. In addition, Van Acker strongly promotes the use of a peripheral tie which will provide restraint to the thermal expansion of the slabs. The thermal restraint will induce compressive forces in the plane of the slab which will reduce the thermal tensile stresses

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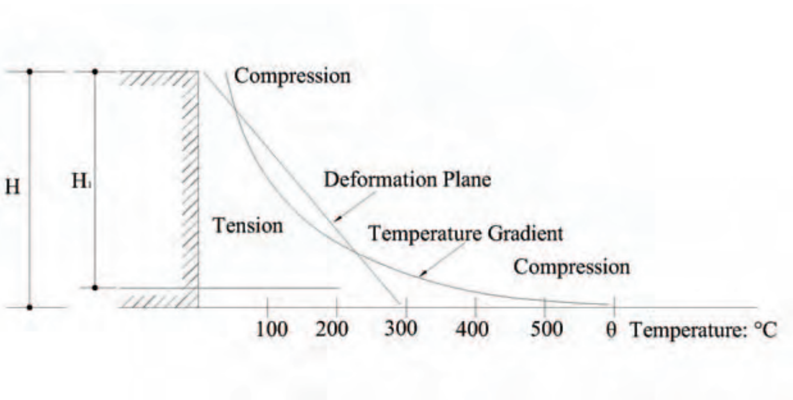

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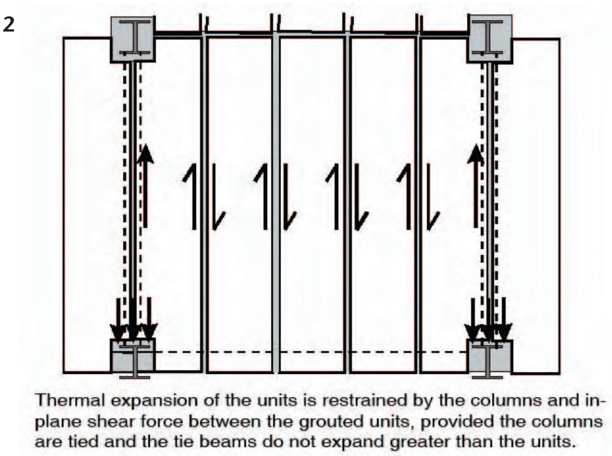

Fig 1. Generation of thermal stresses due to temperature gradient and plane sections remaining plane / Fig 2. Mechanism of providing restraint from a steel frame to thermal expansion of the units (only possible if the tie beams expand less than the units)

and thus reduce the possibility of shear failure. If Van Acker's hypothesis is correct then indeed his pragmatic solution is also correct and the problem is limited to unrealistic small-scale standard fire tests, which do not include local or peripheral tying details.

In the UK, current construction practice does not always use local or peripheral tying details proposed by Van Acker. Prior to December 2004 there was no need to tie the units together over supports for buildings below five stories for robustness. After this date the Approved Document A[6] was revised to provide more robustness to buildings to withstand accidental events. This revision was in-part due to the events of the World Trade Centre and generally results in greater robustness for buildings. However, even with the new rules there is no need to tie the floor units to the supporting structure (or to provide a peripheral tie) for Class 2A buildings comprising low-rise (under 4 stories) hotels, offices, flats industrial buildings etc, as specified in Approved Document A. It is current practice[7,8], and will continue to be, within the UK to sit units directly onto a steel frame without any tying between the units and frame for Class 2A buildings, which represents a significant proportion of the UK building stock.

Without any local or peripheral ties the pragmatic solutions proposed by Van Acker are invalid. This leads us back to the fundamental question of what is the real mechanics of shear failure witnessed in the Danish tests and how can we design the slabs such that failure will not occur in practice, without the need for expensive and arguably unnecessary tying? In addition it is known[7] that grouting the units together, and grouting the gap between the units and steel columns, provides diaphragm action for Class 2A buildings. The question then arises of whether this diaphragm action is sufficient to alleviate premature shear failure witnessed in the small-scale tests? To answer both these questions there was a clear need to carry out full-scale testing, based on current UK practice for Class 2A buildings. This involved placing hollowcore units directly on steel beams to investigate whether the inherent restraint to thermal expansion, created by grouting the units together and around the columns, is sufficient to alleviate shear failure (Fig 2). Restraint will be provided to the thermal expansion of the units provided the columns are tied and the tie beam does not expand greater than the units (Fig 2). In addition a second test was conducted, which was identical but with the units tied to the supporting steelwork to represent a Class 2B building. The two tests would allow the effects of tying to be assessed, under fire conditions.

The steel frame was protected using a minimum thickness of board to achieve 60mins standard fire resistance. A minimum thickness was specified to ensure that there was no inherent resistance in the tie beam created by reduced thermal expansion due to any increase in protection. Temporary bracing was provided to the steel frame during construction, which was removed once the compartment blockwall was constructed. This ensured that the bracing did not increase the restraint to the floor system during the tests, creating a worst case scenario. In practice the bracing within the structure will enhance the performance of the hollowcore floor during a fire. The cover to the strands in the hollowcore units was specified to achieve 60mins standard fire resistance. The ventilation and fuel load were designed to simulate the time–temperature response corresponding to the standard fire curve up to 60mins. The test was then continued through the cooling stage of the fire, creating a much more severe test compared to the standard fire test. No consideration was given to the design of the hollowcore units or protecting steelwork during the cooling phase of the fire prior to the tests. Overall the design of the test represented a worse possible case, removing any inherent resistance within the system (supporting steelwork and hollowcore floor) which will generally be present in normal design.

Design of the tests

The two fire tests were designed within a fire compartment of internal plan dimensions 7.02m × 17.76m, with an internal floor to soffit height of 3.6m (Fig 3). The units were supported on steel beams with the floor-plate area (measured from the centre-line of the beams) being 7.0m × 17.86m. The compartment was formed using 100mm thick blockwork, which was protected with 15mm thick Lafarge Megadeco fire board, with the unprotected hollowcore slabs forming the ceiling. Three ventilation openings were provided on the front face, each 2.2m wide × 1.6m high (Fig 4). The supporting steelwork (Fig 3) was protected using 15mm thick Lafarge Megadeco fire board. The fire protection was fitted, to Lafarge's specifications, using an approved contractor. It was reported that the fire protection would achieve at least 60mins fire resistance in a standard fire test. A total of 15 hollowcore units were used, 1200mm by 200mm deep, as shown in Fig 3. The units were supplied by a well-known UK manufacturer, with the geometry of each unit shown in Fig 5. The units, were designed to BS 8110-1 assuming durability class XC1 to BS 8500 and Class 2 serviceability criteria, and were reinforced with seven, 12.5mm diameter strand, as shown in Fig 5. To achieve 60 minutes fire resistance the units had 25mm cover (31.3mm axis distance) to the strands in accordance to BS 8110-2:1985[9].

The average cube strength of the units was 86 N/mm^2 at 28 days. No additives or air entraining agent was used, with the mix design (for 1m^3) comprising: 320kg OPC, 918kg 10mm limestone, 691kg sharp sand, 380kg 6mm limestone, 30kg grey water and 142kg cold water. The units were stored in an internal environment and, at the time of the test, the measured moisture content of the units was 2.8% by weight.

The two tests were identical except for the end restraint conditions to the hollowcore slabs. In the first test the slabs sat directly onto the supporting

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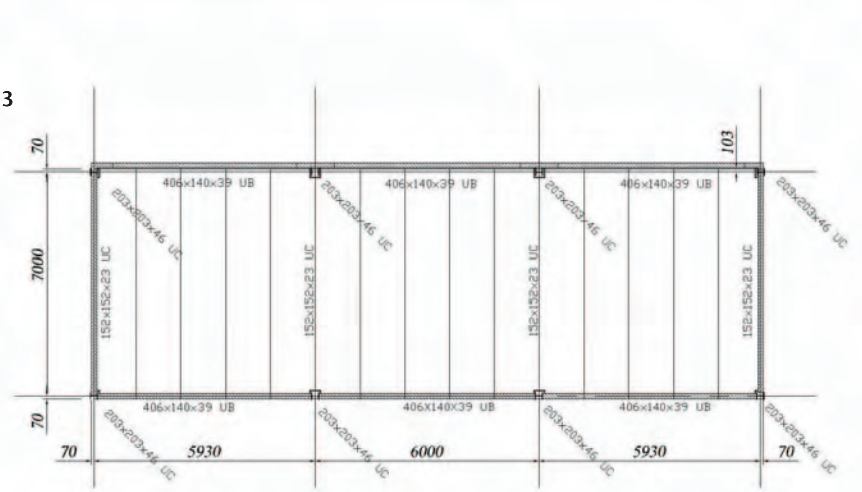

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Fig 3. Plan of the tests (Test 1 untied, Test 2 tied) / Fig 4. Picture showing the ventilation openings

Table 1: Applied static load			
Load type	**Characteristic load**	**Load factor at FLS**	**Load at FLS**
Live load	4.0kN/m^2	0.5	2.0kN/m^2
Partitions	1.0kN/m^2	1.0	1.0kN/m^2
Services & finishes	1.5kN/m^2	1.0	1.5kN/m^2
Total			4.5kN/m^2

beams with the units notched around the columns. The joints between the units, and the gaps around the columns and units, were infilled with grout comprising C25/30 concrete with 10mm aggregate. In the second test, T12-U-bars per unit end were placed in the cores and around a 19mm diameter shear stud fixed to the steel beam. The cores housing the rebars, the end of the slab, the gap between the units, and the gap between the units and steel columns were infilled with grout.

The design applied load is shown in Table 1, together with the partial load factors at the fire limit state.

The applied load of 4.5kN/m^2 was achieved using 60 sandbags (each weighing 1t) evenly positioned over the floor plate, as shown in Fig 6. Taking the floorplate area of 7.0m × 17.86m, this gave an applied load of 4.71kN/m^2. The self-weight of the units was 2.96kN/m^2, creating a total load of 7.67 kN/m^2, and an applied moment at the time of the fire of 56.37kNm per width of unit. This gave a load ratio of 0.34 for bending capacity and 0.26 for shear capacity.

The natural fire was designed using Annex A (parametric temperature-time curves) of BS EN 1991-1-2[10]. Assuming the design for an office, the fire load density was 570MJ/m^2 (80% fractile) corresponding to the value in the UK National Annex[11] and the value given in BS PD 7974-1[12]. This value is higher than the 511MJ/m^2 given in Annex E of BS EN 1991-1-2. The fire load was achieved using 40 standard (1m × 1m × 0.5m high) wooden cribs, comprising 50mm × 50mm × 1000mm wooden battens, positioned evenly around the compartment, as shown in Fig 7. The fire load equated to 32.5kg of wood/m^2.

The aim of the tests was to try to follow the standard fire curve up to 60mins, to investigate the structural behaviour and to enable the test to be compared against the structural performance in small-scale standard fire tests. To achieve the desired time-temperature relationship the ventilation conditions had to be specified by carrying out an iterative calculation procedure. It was found that three openings 2.2m wide × 1.6m high, along the front face were required (Fig 4). It should be noted that the openings are not representative of glazed areas found in modern offices, which will generally have greater ventilation openings. If the ventilation openings are increased, the fire duration will become shorter and the maximum temperature greater. This short duration, high temperature fire will be more beneficial to concrete members and protected steel members due to the time-lag required for these elements to increase in temperature. It can therefore be concluded that the design fire, used in the tests, is extremely severe when considering modern construction.

Instrumentation was included in each test to measure atmosphere temperatures, the temperature distribution through the units, the temperature of the protected steel beams, and vertical and horizontal displacements. The atmosphere temperature was recorded (using a bead thermocouple) at 19 locations at a position 300mm below the underside of the floor units. At five locations an additional thermocouple was placed 600mm below the underside of the units. In total, 24 location readings, per test, were taken to monitor the atmosphere temperature.

The temperatures in the protected steel beams were recorded at 16 locations, spaced evenly around the compartment. At each location two thermocouples were placed on the top flange and two on the bottom flange with one thermocouple placed at mid-height of the web of the beam. A total of 90 thermocouples were used, per test, to measure the temperatures of the protected beams.

The temperature distribution through the units was measured at 27 locations, spaced evenly over the floorplate. At each location the temperature was measured at five points through the thickness of the slab, including at the loca-

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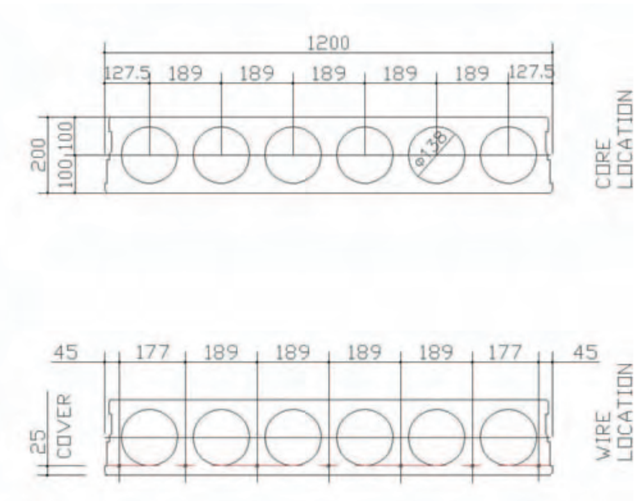

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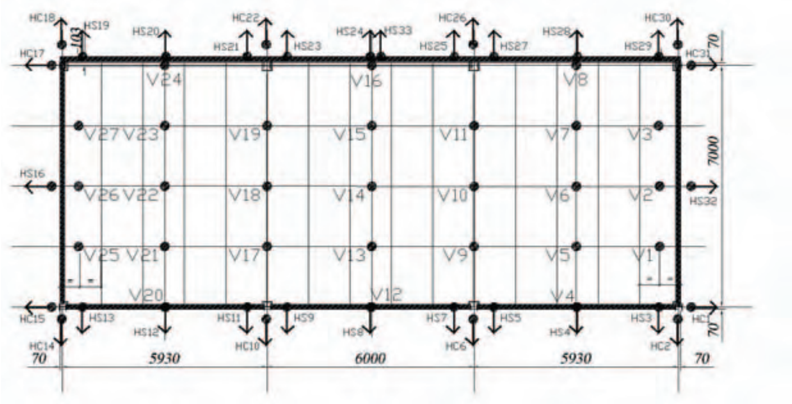

Fig 5. Geometry and strand location of the units / Fig 6. Vertical static load applied using 60 (1t) sandbags / Fig 7. Wooden cribs used for the fire load / Fig 8. Locations of vertical and horizontal measurements

tion of the strand. The thermocouples were placed in pre-cut-out holes in the units and infilled on site. For Test 1, the profile of the voids within the units was maintained using cardboard tubes. Due to time constraints, for Test 2 the pre-cut-out holes, at measurement locations, were totally infilled with the grout encroaching into the voids. A total of 140 thermocouples were used, per test, to measure the temperature distribution through the units.

The locations of horizontal and vertical measurements are shown in Fig 8. The horizontal and vertical measurements were taken from a self-supporting reference frame constructed around the test structure.

Test results

Both tests were carried out by the Building Research Establishment at its Laboratory in Middlesbrough in the UK. Test 1 and Test 2 were conducted on 23 March 2007 and 30 March 2007 respectively. The units were just over 4 months old (cast on 9 November 2006) and were stored in an internal environment over this period. In each test the wooden cribs were connected to each other by a steel channel holding porous fibre board. The board and cribs were soaked in paraffin prior to ignition. The cribs were ignited by three members of staff working from the back of the compartment towards the front. Pictures of the fire test at the height of the fire are shown in Fig 9.

Fig 10 shows the comparison between the average atmosphere temperature for Test 1 and Test 2, together with the calculated parametric and standard fire curve. It can be seen that the maximum average atmosphere temperature was similar in both tests (1069ºC in Test 1 and 1047ºC in Test 2). However, in Test 2 the duration up to the maximum temperature was greater and the temperatures during the cooling stage were also greater, corresponding well with the parametric curve during this phase. The difference in the fire behaviour was probably due to the wind conditions on the day of the test. Test 1 was at the front of the hanger which was open to the elements and wind gusting across the open dock in front of the hanger. Test 2 was shielded by the Test 1 structure and was not subjected to the same wind conditions.

In terms of maximum temperature both tests were more severe than the standard fire curve (up to 60mins) and more severe that the parametric curve during the heating phase. The unconservatisim of the parametric fire curve raises some concern since this is being used in current fire engineering design methods. Further work is required to understand the reasons why the design parametric fire curve underestimated the severity of the fire for these tests.

Both tests supported the full applied load for the duration of the fire. The crack pattern on the underside of the slab for Tests 1 and 2 is shown in Fig 11. There was no significant spalling in any of the tests. There was some localised spalling in Test 2 to one of the units, which slightly exposed the strands, but this was over a small area (Fig 11) and did not affect the global structural behaviour.

Fig 12 shows the inside of the compartment following the test and Fig 13 shows the residual deformation of the slabs, in Test 1, with the sandbags removed. From Fig 12 it can be seen that in some places the protection to the steelwork did not remain in place for the full duration of the fire. In addition the protection lining to the blockwork wall did not remain in place.

Fig 14 shows a comparison between Tests 1 and 2 for the vertical displacement at position V14 (centre of the floorplate) against the average atmosphere temperature during both the heating and cooling phase. At the maximum average atmosphere temperature in Test 1 (1069ºC) the displacement at the centre of the floorplate was 161mm. During a part of the cooling stage, in Test 1, displacement data was lost between 1069°C and 276°C. However, records taken by the authors, from the monitoring data, showed that the maximum displacement during this part of the cooling stage was 410mm (span/17). The measured residual displacement following the cooling period was 264mm. For Test 2 the displacement at the centre of the floorplate was 174mm at the maximum atmosphere temperature (1047ºC). The maximum displacement at the centre of the floorplate during the cooling phase was 369mm (span/19), with the measured residual displacement being 242mm.

From Fig 14 it can be seen that the response of the floor slab is very similar in terms of vertical displacement, with Test 2 showing slightly higher vertical displacements during the heating phase and slightly lower displacements during the cooling stage. It can therefore be concluded that the different end conditions in the tests did not have a significant effect on the response of the units in terms of vertical displacements.

One of the main aims of the test was to investigate whether the protected steel frame would provide sufficient restraint to the expansion of the units to alleviate any possible shear failure. Fig 15 shows the recorded horizontal displacements around the compartment at 54mins (1053°C average atmosphere temperature) in Test 1. It can be seen that the columns are pushed out further than the units showing that the frame does not provide any longitudinal restraint to the units. Similar findings were found in Test 2, with Fig 16 showing the cracking behaviour around the middle edge column after the test, which highlights that the column was pushed out further than the units. The findings from the tests show that the steel frame does not provide longitudinal restraint to the thermal expansion of the units, which if present would have enhanced the unit's shear capacity. However, no shear failure occurred in the test, indicating that some other load-path mechanism was possibly occurring.

The average reinforcement strand temperature at the centre span of the units, for Test 1, is shown in Fig 17, together with the average atmosphere temperature. At the maximum average atmosphere temperature (1069°C) the average strand temperature was 343°C. The maximum average strand temperature (541°C) occurred well in to the cooling stage of the fire when the average atmosphere temperature was 554°C. It is worth mentioning that in standard fire tests the cooling stage is not considered and once the target time is reached (60mins in this case) the furnace is switched off and the load removed. Using the design method presented in BS EN 1992-1-2[3] the flexural resistance, based on an average maximum strand temperature of 541°C, is 45kNm. This value is significantly lower than the calculated applied moment of 54.8kNm. The temperature of the strand was generally lower in Test 2 which was due to the voids at the measurement locations being totally infilled and the voids within the units not being locally maintained at these locations.

The calculation of the moment resistance of the units, based on the principles of EN 1992-1-2, suggests that the units should have failed under pure flexural action, indicating again that some other load-path mechanism was occurring. There was evidence from the tests that there was some lateral restraint to the thermal expansion of the units at the column locations (Fig 18), which provided a compressive lateral 'strip' at the ends of the units. This

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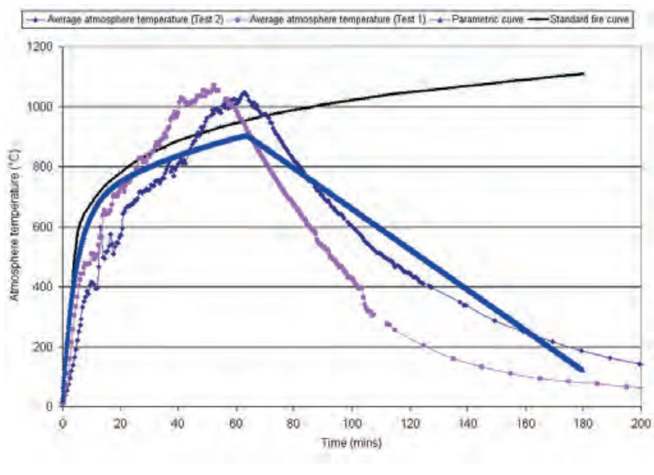

Fig 9. a) Test 1 / b) Test 2 / Fig 10. Average atmosphere temperature for Test 1 and Test 2

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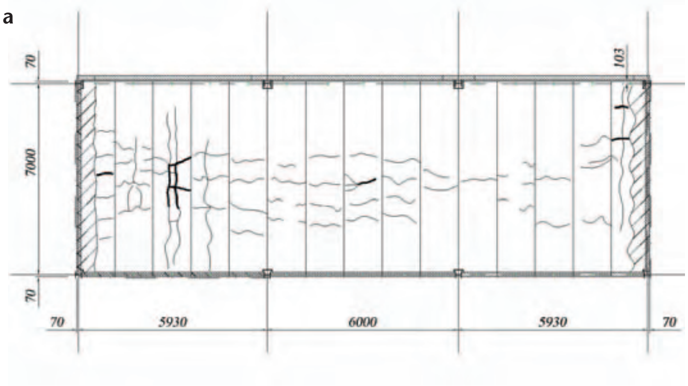

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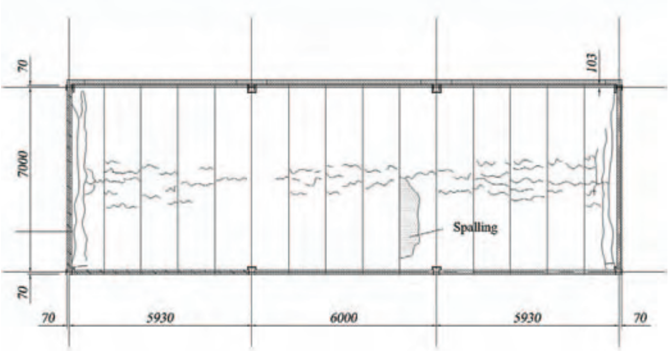

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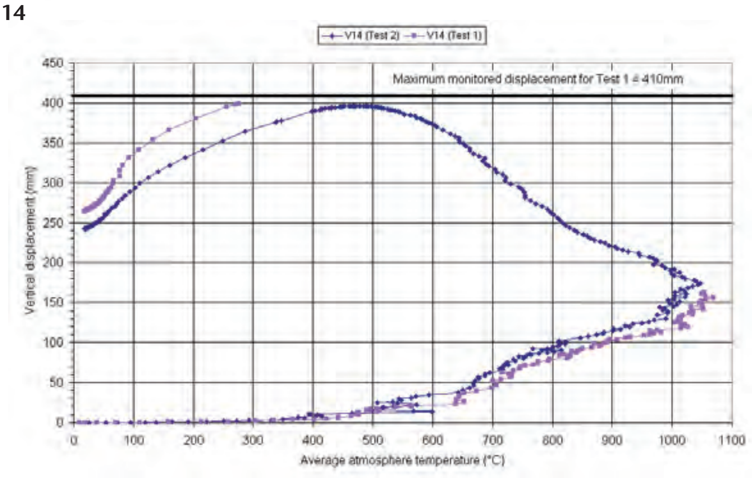

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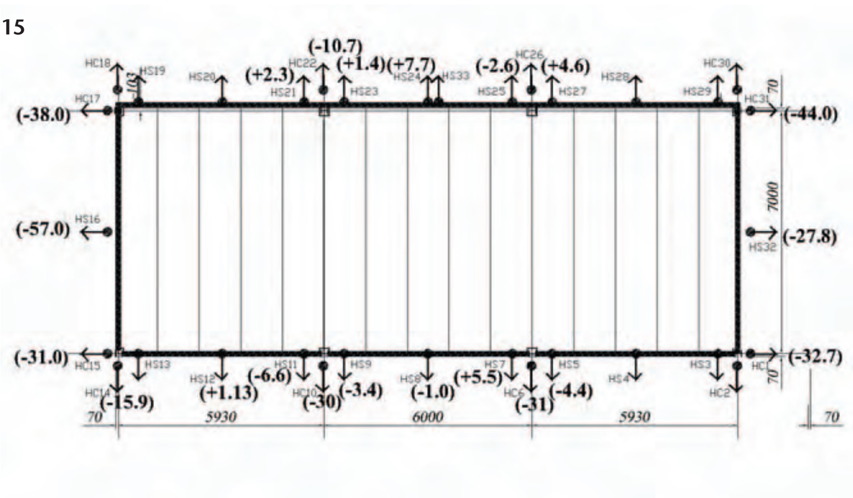

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Fig 11. Crack pattern on underside of floorplate following the test a) Test 1 b) Test 2 / Fig 12. View inside the compartment following the fire (Test 1) / Fig 13. Deformation of the floor plate following removal of the sandbag loading / Fig 14. Measured vertical displacement at centre of floor plate against average atmosphere temperature for Test 1 and Test 2 / Fig 15. Measured horizontal displacements at 54min. in Test 1 / Fig 16. Cracking around internal edge column in Test 2 (note the gaps between the concrete and steel flanges)

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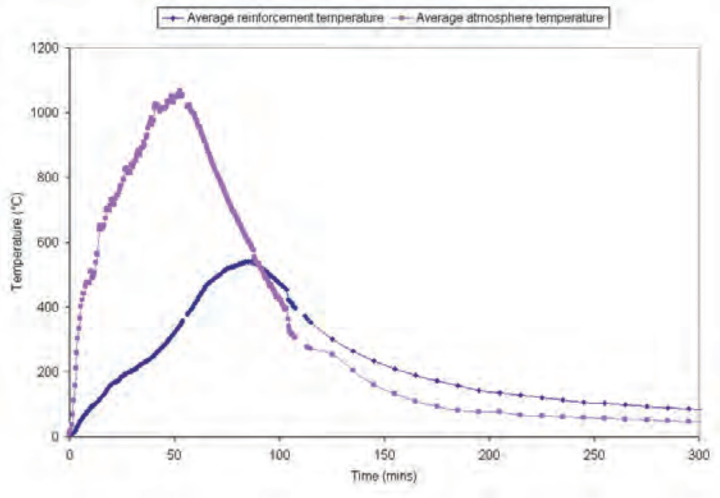

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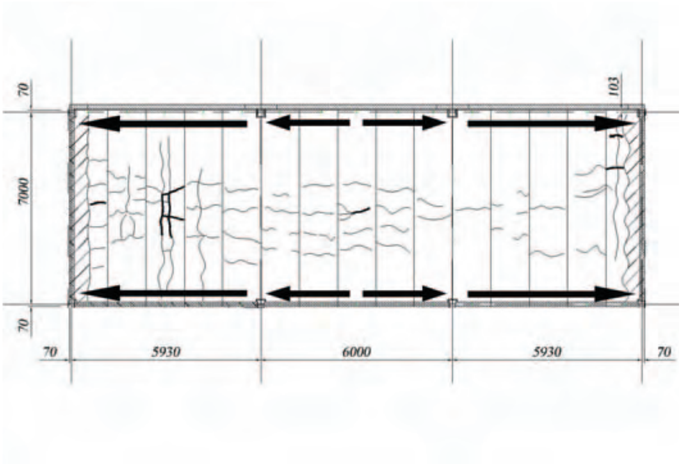

hypothesis is supported by the observed crushing of the edge units as shown in Fig 19. It is possible that the induced compressive strip provided restraint to the strands allowing them to partially support the load in catenary action. In addition the compressive strip would have reduced strand slippage and enhanced the shear capacity of the units. However, further work in terms of computer modelling would be needed to support this assumption, before it is utilised in practical design.

The average bottom flange temperatures of the protected steel tie beam in Tests 1 and 2 are shown in Fig 20. The maximum steel temperature was 709°C and 777°C in Test 1 and 2 respectively, which occurred during the cooling stage of the fire. For a fully loaded isolated member, simple design[13] suggests that the beam should remain below a temperature of 620°C. The steel beams remained below this temperature during the heating phase of the fire, but exceeded this target temperature during the cooling phase of the fire. Similar to the hollowcore units no consideration was given to the design of the supporting steelwork during the cooling phase of the fire prior to the test.

During the cooling stage of the fire the edge units in both tests fractured along their span. In Test 1 one edge unit fractured, as shown in Fig 21, at 87mins (average atmosphere temperature of 576°C) and the other fractured at 103mins (average atmosphere temperature of 394°C). In Test 2 fracture occurred, as shown in Fig 22, in one edge unit at 76mins (average atmosphere temperature of 870°C) and at 95mins (average atmosphere temperature of 645°C) in the other edge unit. Due to the tying reinforcement provided in Test 2 the fracture of the edge units occurred earlier during the cooling phase. However, the restraint provided support to the unit resulting in a large crack forming, whereas in Test 1 the outer proportion of the edge unit collapsed after fracture occurred (Fig 21). It is felt that the fracture of the units is predominantly due to transverse bending of the units due to the large vertical displacement of the floor plate experienced during the cooling phase of the fire. The fracture of the edge units was localised and overall stability of the floor was maintained. The plastic sandbags on the floor did not ignite following localised fracture of the edge units.

Conclusions

The results from the two tests has highlighted the inherent fire resistance of hollowcore floor systems when subjected to very severe fire scenarios. The results reinforce the experience from real fires that hollowcore floors behave very well under fire conditions. Based on the results from the two tests presented in this paper, the following conclusions are drawn.

All hollowcore units performed very well during the heating phase of the fire, which was more severe than the standard fire curve over 31 to 62 mins in Test 1, and 47 to 70 mins in Test 2. The maximum average atmosphere tempera-

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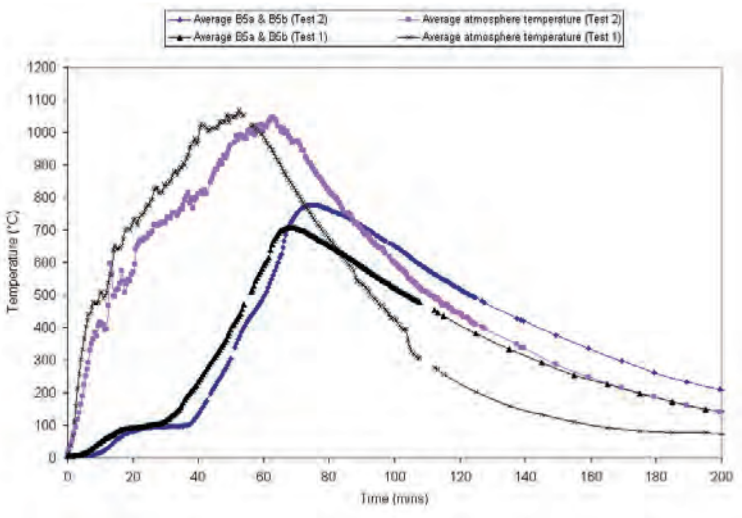

Fig 17. Average reinforcement temperature at centre span of the units for Test 1 / Fig 18. Possible restraint to slabs creating compressive 'strip' / Fig 19. Compressive failure of edge units / Fig 20. Measured bottom flange steel temperatures in one of the tie beams

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ture was 1069°C in Test 1 and 1047°C in Test 2.

The floor, as a whole, performed well during the cooling phase of the fire. The applied load was supported for the full duration of the fire even though the system was not designed for the cooling stage of the fire prior to the test. The edge units did fracture locally during the cooling phase of the fire but this did not lead to loss of overall load carrying capacity. In addition, the plastic sandbags on top of the heated floor did not ignite following fracture of the edge unit.

The fire was very severe and, in terms of ventilation openings, was unrealistic when compared against modern office construction. With greater ventilation the fire would have been hotter but of shorter duration. A hotter fire, of short duration, would have been beneficial to the performance of the hollowcore units and protected steelwork, since the temperatures of the structural elements would have been lower.

The parametric design fire curve, as specified in BS EN 1991-1-2, produced unconservative results, in terms of lower temperatures during the heating phase, when compared against the test results. Further work is needed to address this unconservatisim possibly leading to a revision in the codified approach.

Although the 28 day cube strength was 85.7N/mm^2 there was no significant spalling to the units, which may occur with high strength concretes (over 60N/mm^2). There was some localised spalling in Test 2 to one of the units, which slightly exposed the strands, but this was over a small area and did not affect the global structural behaviour.

The different end restraint conditions did not affect the measured vertical displacement. In addition the columns in both tests were pushed out further than the units suggesting that there was nominal longitudinal thermal restraint to the units. The restraint conditions in Test 2 were beneficial in keeping the outer proportion of the edge unit in place when it fractured along its length. Although the restraint did cause this fracture to occur earlier during the cooling phase of the fire.

There was evidence of a lateral compressive strip forming at the ends of the units caused by restraint to thermal expansion. This 'strip' would have enhanced the flexural capacity of the units since it would have restrained the ends of the strands allowing some catenary action to occur. In addition the compressive strip could enhance the shear capacity of the units by reducing the strand slippage. Further work is underway to investigate this beneficial behaviour to enable it to be utilised in practical design. It should also be noted that the vertical end displacement of the units needs to be nominal to allow this behaviour to occur.

The applied design live load was based on 4.0 + 1.0 kN/m^2, with a load factor of 0.5 corresponding to the fire limit state. This load is significantly higher than the minimum office load of 2.5 + 1.0 kN/m^2. The calculated load ratios were 0.34 for bending and 0.26 for shear. Work is currently underway to develop simple design guidance to cover a range of possible load ratios.

Acknowledgments

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Fig 21. Fracture of the edge unit during the cooling stage in Test 1 / Fig 22. Fracture of the edge unit during the cooling phase in Test 2