Fire safety of prestressed hollowcore floors

Prestressed hollowcore units are now the most widely used type of precast flooring in Western Europe. This success is largely due to the highly efficient design and production methods, flexibility in use, surface finish and structural efficiency. However, some concern has been recently raised about the actual performance of hollowcore floors in fire, following some limited calamities in a few real fires. Also a few cases of premature shear failure in small scale standard fire tests were reported in the past. Henceforth, the question could be raised if this constitutes a real structural problem for this type of floor, or whether the reason lies in a lack of understanding of the behaviour of hollowcore slabs during heavy fire, resulting in poor design, also for small-scale laboratory test set-ups.

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The fire resistance of a concrete structure exposed to fire is a very complex phenomenon, whether it is a cast-in-situ structure or a precast one. This is due to the large number of parameters intervening, and all possible variations thereof. In the first place, there is the intensity of the fire, depending among others on the actual fire load and the combustibility of the materials.

Also the air supply is crucial, since combustion needs oxygen. Another parameter concerns the location of the fire within the structure, the size of the building and the extension of the fire. Also the structural lay-out and dimensions of the building and the components are influencing the stability during a fire. And finally there are the dimensions of the elements, the concrete composition, moisture content of the hardened concrete, axis distance to the reinforcement, etc.

During the last decade, the knowledge about the phenomenon playing a role in the shear capacity of hollowcore floors has increased drastically. Unfortunately it is not always known to all designers. In the framework of the European Standardisation, a Task Group has been established a couple of years ago, to redraft the annex G “Fire resistance” of the Product Standard EN 1168 on hollowcore units. The tabulated values related to minimum slab thickness and axis distance of the reinforcement, given in the previous publication, were considered unconservative and also the requirements on shear capacity were insufficient.

This paper provides a detailed analysis of the effect of fire on prestressed hollowcore floors and explains the important role of the connections with the supporting structure in the shear capacity at fire. A new calculation method for the shear resistance of prestressed hollow core floors is explained.

Phenomenon

When a concrete structure is exposed to heavy fire, two phenomenon occur simultaneously: a reduction of the material performances and a thermal expansion. The first phenomenon is well known and constitutes at present the basis for the fire design. It focusses primarily on minimum dimensions of the cross-section of the components and minimum axis distance of the reinforcement. The second phenomenon is less known and also much more complex. Thermal expansions occur not only in all directions but also over the cross-section of the components. As a consequence, induced thermal stresses may occur the magnitude of which depending on the size of the building, the stability concept, the structural lay-out, shape and cross-section of the units, etc.

At real fires in concrete structures, it is stated that collapse seldom occurs because of the decrease of the material performances at elevated temperatures, but nearly always because of the incompatibility of the structure to cope with important thermal deformations. Fortunately, concrete structures possess not only a high fire resistance, but also a large resilience because of its robustness and the possibility to redistribute the acting loading. Failures due to fire are seldom occurring in concrete structures. This is also the case for precast hollowcore floors.

Thermal expansion

When a fire occurs in the central part of a large floor, the thermal expansion of the units will be almost completely restrained by the rigidity of the surrounding floor. The restraint will mobilise important compressive forces in the fire exposed floor units. This was for example demonstrated during a real fire in a small storage room in
the middle of a large multi-storey building under construction. Due to the heavy fire of some barrels with fuel, extreme high local temperatures arised. The thermal expansion of the hollowcore floor above the fire was completely restrained by the surrounding cold floor. Due to the induced compressive force, large concrete spalling occurred in the underflange of a few floor units.

When a fire occurs at the edge of a floor, some restraining effect may also be generated through the rigidity of the edge construction. In the example of Figure 1, the thermal expansion of the floor above the fire is restrained by the edge column via the supporting beam.

![Fig. 3: Temperature distribution over the cross-section of a 265 mm thick hollowcore slab](image1)

![Fig. 4: Deformation versus thermal gradient and generation of transversal forces](image2)
which transfers the induced horizontal force to the floor structure above and below the fire-exposed floor. In case of a local fire at an edge bay of a building, the not exposed surrounding floor itself may also contribute to the restraint to expansion. Due to the reinforced structural topping and the transversal tie-reinforcement at the support zone, the floor is acting as a coherent diaphragm. Any expansion of a floor unit or a floor zone, will be restrained by the not exposed neighbouring units. The force is transferred partially via the longitudinal joints, [shear force capacity 0.25 to 0.40 N/mm² [4]], and partially through the reinforced topping and the transversal tie reinforcement with the edge structure.

Heavy heating of the underside of a floor may also lead to lateral expansion of the units. However, just as in the longitudinal direction, this lateral expansion of the bottom flange will be restrained by the surrounding structure in a more or less important way, depending on the structural lay-out. Slabs located in the middle of a large floor will be completely restrained by the surrounding floor structure. Slabs located at the edges of the floor will undergo a smaller transversal restraint, and the thermal expansion of the bottom flange may accumulate at the exterior slabs, and hence increase the transversal shear in the webs. Indeed, during standard fire tests, small horizontal cracks have been stated a few times in the outmost webs of the floor set-up, due to the diff-
rental expansion between the top flange and the bottom flange, especially in slabs with a reinforced structural topping. However, in most fire tests, the differential lateral expansion results in vertical cracks at the cores in the longitudinal direction through the slab, because of the weaker cross-section of the top and bottom flanges compared to the webs. In neither of the observed cases, those cracks were leading to failure of the test floor, even after 150 minutes ISO fire exposure.

Induced thermal stresses
Hollowcore units exposed to fire are subjected to induced thermal stresses over their cross-section. Measurements during ISO fire tests show that the temperature gradient has a strong curvature. Figure 3 shows the registered temperatures over the cross-section of a 265 mm thick slab respectively after 30, 60, 90 and 120 minutes of exposure. It can be seen that after 30 minutes the curvature of the temperature profile is stronger than for other fire exposure time.

Due to the typical non-linear thermal gradient through the hollowcore slab, and the fact that plane sections must remain plane, thermal stresses are induced through the cross-section as shown in Figure 4.

The following distribution of thermal stresses is shown: compression at the bottom of the slab, changing rapidly into tension higher up. Further upwards, the tensile capacity of the concrete is reached and the concrete is cracking. Still more upwards, the concrete is again subjected to tension and further up to compression in the top section.

Calculations shown that the tensile stresses in the central zone are exceeding the tensile capacity of the concrete after about 20 to 40 minutes, and that vertical cracks are appearing at regular intervals, situated at a distance of about 150 to 200 mm to each other. The results of a calculation example...
are given on Figure 5. The calculations were carried out by the Department of Mechanics and Civil Engineering of the University of Liège in Belgium [5]. The compressive stresses in the underflange of a 265 mm thick hollowcore slab were in the order of 8 to 16 N/mm². Figure 6 shows the cracks in the webs of a longitudinally cutted hollow core slab during a fire test at the laboratory of TNO in Delft, The Netherlands [6].

The reason for the distribution of the cracks over the whole length lies in the fact that the induced tensile stresses are appearing over the whole length of the slab. Since they are larger than the tensile capacity of the concrete, the webs will crack. At the crack the tensile stress drops to zero, but outside the crack it will again built up until it reaches again the tensile strength of the concrete. This happens at intervals of 150 to 200 mm.

Another consequence of the above mentioned stress distribution over the slab height is the appearance of shear stresses in the transition zone between the induced thermal compressive and tensile forces. Finally there are also the normal shear stresses due to the self weight and the variable loading of the slabs.

To sum up, the following complex stress situation is acting over the cross-section of a hollowcore unit exposed to ISO fire:

- Induced thermal stresses causing vertical cracks in the webs at regular intervals, in combination with horizontal tensile stresses inbetween the cracks;
- Horizontal shear stresses of thermal origin in the lower part of the cross-section, as a consequence of the transition of compressive forces into tensile forces;
- Shear stresses in the webs due to the self weight and imposed loading of the slab;
- Shear stresses due to the introduction of the prestressing force at the slab end.

Moment restrained floor support

During a fire, simply supported hollowcore floors will deflect because of the expansion of the bottom fibres. In a floor with moment restrained supports, the deflection will be limited by the top reinforcement.

Hence, the support moment will increase as a function of the amount of reinforcement (Figure 8). Additional compressive forces will develop in the bottom flange of the floor units in the vicinity of the supports. For this reason, restrained supports of hollowcore floors are not recommended.

Failure mechanisms

Simply supported floors

The shear failure mode of hollowcore slabs due to fire, is shown on Figure 9. After a critical time, situated between about 20 and 40 minutes ISO fire exposure, vertical thermal cracks are appearing in the webs (Figure 9(a)). At further fire exposure also horizontal cracks originate in the weakest zone of the cross-section due to shear loading from thermal origin, self-weight, imposed loading, prestressing and thermal expansion (Figure 9(b)).

For circular cores, this section is situated in the middle of the cross-section, for more rectangular cores, it is situated towards the bottom of the slab. The failure occurs when the horizontal cracks are meeting the vertical cracks. The phenomenon is of course influenced by the level of the imposed loading, the level of prestressing and the total web width of the slabs. Also the slab thickness itself is intervening.

Possible strand slippage, observed during fire tests in laboratories on individual hollow core units, may also influence the phenomenon. The slippage is probably due to micro radial cracking of the concrete around the strands, generated at transfer of prestress at manufacture. During further hydration of the concrete, those cracks are bridged by further cristal growth, but they are possibly opening again under thermal deformations during fire. In addition, the tendons are often positioned in the zone of the cross-section where the induced thermal tensile stresses are appearing, hence aggravating the slippage.

The strand slippage is influenced by the diameter of the tendons (small diameters are much less subjected to slippage because of the smaller interface stresses), the tensile strength of the concrete, and confinement of the concrete around the strands. Tests on prestressed concrete units revealed that enclosing the concrete mass around the strands with a spiral reinforcement may totally prevent the slippage. The reinforcing bars in the tie-beam at the support of
Hollowcore slabs are to a certain extent acting as confinement to the concrete around the strands, since the cast in-situ concrete of the tie beam penetrates into the cores over 50 to 100 mm. The hindered expansion of the surrounding structure is also contributing in a positive way to the anchorage of the prestressing tendons.

Floors with restrained supports
Hollowcore slabs, with moment restrained supports and/or restraint to the longitudinal expansion, may fail during a heavy fire because of excessive compressive stresses in the exposed underflange.

There are different possible causes, which may act separately or in combination. The compression stress should be limited to the mean compressive strength of the concrete in the underflange after @ minutes ISO fire ($\sigma_{\text{d}, \theta} \leq f_{\text{cd}, \theta}$). The following causes may be at the origin of the compressive stresses in the bottomflange:

- Compressive stresses due to the support moment
- Additional negative flexural moments due to the restrained thermal deflection of the floor in the longitudinal directions
- Prestressing force
- Compressive stresses due to the temperature gradient over the cross-section
- Compressive stresses due to restraint of the thermal expansion by the surrounding structures
- Hindered vertical deformation because of a very stiff thick topping

Shear capacity of cracked concrete sections
In principle, cracked concrete sections can transfer shear as well as non-cracked sections on condition that the cracks are not opening. This arises because the crack borders are rough and shear forces can be transmitted by shear friction and aggregate interlock (Figure 10). The Figure illustrates the generation of transversal forces due to the wedging effect.

In fire conditions, the same principle remains valid. The decrease of the concrete strength at higher temperatures is not an issue since this happens only in the lower part of the cross-section. In the shear transfer of hollowcore slabs at fire, horizontal cracks in the webs are clearly playing a governing role. Indeed, the vertical cracks alone will not open because of the presence of compression stresses in the top and bottom flanges of the slabs, and the prestressing tendons in the bottom flange.

Only when the vertical cracks are propagating further, either through flexural cracks in the bottom flange, or through horizontal...
Cracks in the webs, the shear capacity becomes critical. However, as explained before, the web cross-section is usually much weaker than the bottom flange, and cracks are thus propagating horizontally rather than vertically. This has also been stated during fire tests. The solution of the problem lies in the realisation of the aggregate interlock effect through both horizontal and vertical cracks.

The connections with the supporting structure are playing a crucial role in the shear capacity of hollowcore floors exposed to fire. Their principal function to keep the horizontal and vertical cracks closed. During fire tests, it has been stated that horizontal tie bars in casted sleeves alone are not sufficient. Figure 11 shows a test set-up at TNO fire laboratory in Delft, whereby a longitudinal reinforcing bar in a casted sleeve was anchored in a small transversal tie beam. During the fire test, premature shear failure was observed. Due to the absence of vertical connecting bars with the supporting beam, there was no interlocking effect in the horizontal cracks. The latter passed below the core filling and the slab collapsed. An additional proof of the effectivity of the aggregate interlock effect on shear capacity is given by the following experiments carried out at the VTT laboratory in Finland. Several fire tests have been carried out on hollowcore slabs, provided with transversal steel straps around their ends. The straps were intended to restrain the transversal thermal expansion. The slabs were 265 mm thick, spanning 5.18 m, and the imposed load was 10.8 kN/m². There were no connections with the supporting structure. No shear failure occurred in any test, even after 150 minutes fire exposure.

**Calculation method of the shear capacity**

The Precast Concrete Commission “TC 229” within the European Standard Institute CEN, has set up a Task Group to draft guidelines for the design of hollowcore floors with regard to shear at fire. Within this framework, the French research centre for the precast concrete industry - CERIB - has elaborated a calculation method. It is based on the formula for shear flexure given in the Eurocode EN 1992-1-1, section 6.2.

According to this Standard, the formula is only applicable for single span members without shear reinforcement in the regions cracked by bending. Hollowcore elements exposed to fire are subjected to vertical web cracking over the full span of the slabs, also at the support region. For this reason, the shear flexure formula has been chosen as basic model rather than the shear tension formula, which is only applicable for non-cracked sections. The formula has been adapted for the fire situation. The validity has been demonstrated by a finite element method analysis, and a very good correspondence with 9 test results where shear failure occurred.

The shear flexure equation for the fire situation is:

\[
V_{sl,c,fi} = \left[ C_{sl,c} \cdot k \cdot (100 \cdot \rho_{li} \cdot \sigma_{sp})^{(\frac{1}{4})} \cdot (1 + \sigma_{sp}) \right] \cdot b_w \cdot d
\]

where:

- \( \rho_{li} \) is the force-equivalent ratio of longitudinal reinforcement:
  \[
  \rho_{li} = \frac{\sum F_{R,li}}{500 \cdot b_w \cdot d}
  \]
- \( b_w \) is the total web thickness
- \( d \) is the effective depth at ambient temperature
- \( F_{R,li} \) is the force capacity of prestressing and ordinary reinforcement anchored at support:
  \[
  F_{R,li} = F_{R,li,p} + F_{R,li,s}
  \]
- \( F_{R,li,p} \) is the force capacity of the prestressing steel anchored at support
  \[
  F_{R,li,p} = \min(A_{pe} \cdot \frac{f_{p,k} \cdot k_f (\theta)}{\alpha_{p,e}}, A_{li} \cdot 0.9 \cdot f_{p,k} \cdot k_f (\theta))
  \]

**Tab. 1: Shear capacity of hollowcore slabs for different fire ratings.**

The values are expressed in percentage of the calculated shear capacity at ambient temperature according to the requirements of the European Product Standard EN 1168:

<table>
<thead>
<tr>
<th>Fire Rating</th>
<th>160</th>
<th>200</th>
<th>240-280</th>
<th>320</th>
<th>360-400</th>
</tr>
</thead>
<tbody>
<tr>
<td>REI 60</td>
<td>70</td>
<td>65</td>
<td>60</td>
<td>60</td>
<td>55</td>
</tr>
<tr>
<td>REI 90</td>
<td>65</td>
<td>60</td>
<td>60</td>
<td>55</td>
<td>50</td>
</tr>
<tr>
<td>REI 120</td>
<td>60</td>
<td>60</td>
<td>55</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

Note: the values of Table G.2 are given for prestressed hollowcore slabs with strands cut at the ends of the elements, and a section of 1,88 cm²/m of longitudinal tie reinforcement at the support.
FR,a,fi,s is the force capacity of ordinary reinforcement anchored at support.

\[
F_{R,a,fi,s} = A_s f_y k_s(\theta_m)
\]

where:
- \( A_s \) is the area of the longitudinal reinforcement
- \( f_y \) is the yield strength of the steel
- \( k_s(\theta_m) \) is the strength reduction factor for the ordinary reinforcement at temperature \( \theta_m \), according to EN 1992-1-2, clause 4.2.4.3

Note: the anchor reaction capacity of the longitudinal reinforcement embedded at support may be calculated taking into account the effect of concrete mass on temperature distribution by using the average temperature \( \theta_m \) of the strand at support.

Note: if the longitudinal reinforcing steel is situated approximately at mid-height of the slab, the strength reduction factor \( k_s \) can be taken equal to 1.
\( \sigma_{cp,h} \) is the average stress on concrete section for fire condition,

\[
\sigma_{cp,20} = \min \left( k_1 \sigma_{cp,30} \frac{f_{cd,c}}{A_c} \right)
\]

\( \sigma_{cp,20} \) is the concrete stress due to prestressing force at normal temperature

\( A_c \) is the concrete section area

\( f_{c,k,m} \) is the average strength of concrete at elevated temperature,

\( f_{c,k,m} \) can be taken equal to the strength of concrete for the temperature at mid height of the web

\[
k = 1 + \frac{200}{d} \leq 20 \quad (d \text{ in Millimeter})
\]

\( C_{rd,c} = 0.18 \)

\( k_1 = 0.15 \)

Note: The values of \( C_{rd,c} \) and \( k_1 \) or use in a Country may be found in its National Annex. The recommended value are \( C_{rd,c} = 0.18 \) \& \( k_1 = 0.15 \).

The above method has been used to draft table 1, giving shear capacity values for hollowcore units for different slab thicknesses and load ratios, in function of the required fire rate.

**Practical detailing**

There are different ways to realise the aggregate interlock function in hollowcore floors. The effectiveness has been proven in numerous fire tests in different laboratories.

a) Reinforcing bars connecting the slabs to the supporting structure (Figure 13)

The projecting stirrups from the supporting beam, together with the longitudinal tie bars inside the filled cores or in the longitudinal joints, assure the vertical restraint of the slab to the supporting beam, and prevent horizontal cracks in the lower part of the slab from opening.

b) Structural topping

The vertical restraint of the slab to the supporting beam can also indirectly be realised through the connection between the reinforcement in a structural topping, with the projecting stirrups from the floor beam (Figure 14).

c) Vertical compression force on the slab ends (Figure 15)

In bearing wall structures at low cost residential buildings, the hollowcore
floors are sometimes directly supported on the wall without further connection reinforcement. The compression force from the superposed panels may be sufficient to keep possible horizontal cracks closed and realise the aggregate interlock mechanism. The solution is only applicable for very limited shear loading from self weight, imposed loading and prestressing force.

However, the solution with projecting bars and tie beams as shown in Figure 13, is preferable.

Research

In the past, more than hundred fire tests in laboratories all over the world have been carried out on hollowcore floor units. They show that the flexural capacity for fire periods of 120 minutes, and even up to 180 minutes, can be obtained with normal hollowcore units, provided sufficient cover to the prestressing strands is respected. Much less research has been carried out on the shear capacity at fire. Experiences at laboratory tests and real fires demonstrate that also for this aspect sufficient bearing capacity is obtained, on condition that good connections with the supporting structure are available. However standard fire tests on hollowcore floors were seldom carried out with high shear loading, since the latter is seldom governing in normal design. Three important research projects, carried out in the past decades to study the shear capacity of hollowcore floors exposed to severe fire, are explained hereafter.

Laboratory tests in Belgium [3] [5]
The research project was carried out at the fire research departments of the Universities of Ghent and Liege. It comprised a theoretical study and 4 fire tests. The aim of the theoretical study was to evaluate through finite element analysis, the magnitude and location of thermal stresses caused by differential deformation of the concrete section for different fire exposure times, but also to examine the influence of parameters such as restraint of the thermal expansion, catenary effect of the deflection, size of the cross-section and more. The calculations were made with the computer program "Safari" at the University of Liège. The calculation results are visualised by means of graphs. Figure 5 shows a calculation example of the stress curves in a floor unit of 265 mm thickness, without longitudinal restrainment of the thermal expansion.

The practical part of the project comprised 4 standard fire tests of 2 h ISO fire at the laboratory of the University of Ghent. Since the objective of the research was to examine exclusively the shear capacity, the test set up was designed to enable the testing of a maximum number of parameters. The test furnace measured 6 x 3 m. The test set up of the four tests is shown on Figure 16. Each test comprised two floor spans of 3 m, supported on three beams, and a floor width of 2.40 m.

The four tests were conceived to simulate the conditions of a real hollowcore floor, and the following parameters were included: longitudinal, transverse and peripheral tie reinforcement and a structural topping, but also the restraining effect of a surrounding floor structure and the edge beams and columns.

The influence of the surrounding structure was simulated by a longitudinal bar at both sides of the floor, and a T-shaped ending of the transversal tie beams above the three supporting beams. The size of the longitudi-
nal bars was chosen to simulate partly the longitudinal tensile capacity of the tendons in one floor unit.

In the first and the fourth tests, a structural topping screed was cast on one of the two floor spans. In the first test, the restraining effect of the topping was clearly demonstrated by a longitudinal crack appearing at mid-depth of the lateral floor face. This was due to the large temperature difference between the hot underside of the floor and the cool upper side. The resulting differential expansion was hindered by the transversal reinforcement in the topping screed, and caused a small longitudinal crack at mid-depth of the floor. However, it appeared that the crack was limited to the first longitudinal void and did not affect the stability of the floor. The first test was interrupted after 83 min because of the appearance of a hole in the slab right under the pressure vessel.

The local failure was probably caused by a concentrated contact pressure under the vessel. All other tests were stopped after 120 minutes fire exposure. Immediately at the end of each test, the loading was increased for each floor span until failure, to check the remaining capacity after the fire. The results are given in Table 2. The failure load for the floor span with topping in the first test was 254 kN. The units failed in bending, which leads to the conclusion that the shear capacity was higher. The other floor span without topping, where a large hole was observed during the test, failed at a load of 178 kN, also in bending. Also in this case it can be concluded that the shear capacity was not reached. The following three tests were stopped at 120 min fire exposure. During the subsequent loading to failure, all floor
slabs failed in bending, except one span, showing a shear failure. The underside of the latter floor was more damaged by spalling than the others, which might explain the shear failure. All failure loads were very high in comparison to the test load of 100 kN, which has to be considered as a normal fire loading. The tests prove that hollowcore units dispose of a large bearing capacity at fire, both for bending and shear.

Laboratory tests in Denmark [7]
In April 2005, the Danish Precast Concrete Association carried out a series of tests on the fire resistance of hollowcore floors with high imposed shear loading. The purpose of the tests was to demonstrate that hollow core slabs exposed to a 60 minutes standard fire and the subsequent cooling phase can resist a shear loading corresponding to 65% of the ultimate shear capacity at ambient temperature.

Figure 18 shows the test set-up. It comprises a complete floor slab, composed of 5 hollowcore units of 265 mm thickness and 2.93 m span, supported on two beams and tied together by a peripheral ring beam. The idea was to simulate a real floor slab with normal connections to the supporting structure. The 6.18 m wide test floor was symmetrically positioned on the test furnace which was 2.35 m wide. Two cantilevering zones of each 1.775 m at both sides of the furnace were meant to simulate the restraining effect of a surrounding floor to the thermal expansion of the fire exposed floor. In order to allow vertical deflection of the exposed slabs, the test zone was separated from the buffer zones by cutting the two slabs at the longitudinal edges of the furnace over their full length. The joint was filled with insulating material. The exposed zone comprised hence one whole slab and two half slabs.

The applied load was achieved with five hydraulic jacks positioned at a distance of 0.66 m from the floor support (2.5 times the slab thickness). Three tests were performed with load levels corresponding to 65%, 75% and 80% of the ultimate shear capacity at ambient temperature.

The heating regime followed the standard time temperature curve during 60 minutes, followed by a 90 minute cooling phase under full loading. The temperature registrations for tests 1 and 2 are shown on Figure 19. After 60 minutes, the furnace was switched off and the test continued for a further 60-90 minutes cooling phase with the load still being applied.

The two first tests, with shear loading of 65% and 75%, supported the full applied load for the duration of the fire and the complete cooling period. During the third test with a loading of 80%, shear failure occurred after 45 minutes fire exposure. There was no significant spalling in any of the tests.

Figure 20 shows the deflection of the hollowcore units in the first test. The lower dark curve corresponds to the deflection of the slab in the middle of the test zone. The other two red curves show the deflections in the middle of the two half slabs.

The results from the three tests prove that the hollowcore slabs were able to resist a shear load at 60 minutes standard fire exposure of 75% of the shear capacity at ambient temperature. The slabs continued to carry the load during a subsequent cooling period of 60 – 90 minutes.

Full-scale tests in UK [8]
Prestressed hollowcore floors are very popular in the UK construction market for apartment buildings with steel frame structure. It is current practice to sit units directly onto the steel frame without any tying between the units and frame for low rise buildings (under 4 storeys).

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. The failures
during the tests were attributed to the lack of connection detailing between the floors and the supporting structure used in practical design.

The purpose of the study was 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. Two full-scale fire tests were carried out by the Building Research Establishment at its laboratory in Middlesborough on hollowcore floors supported on steelwork.

The two fire tests were designed within a fire compartment of internal plan dimensions 7.02 x 17.76 m, with an internal floor to soffit height of 3.6 m (Figure 21). The units were supported on steel beams with the floor plate area 7.0 x 17.86 m. The compartment was formed using 100 mm thick blockwork, which was protected with 15 mm thick fire board, with unprotected hollowcore slabs forming the ceiling. Three ventilation openings were provided on the front face of each compartment, each 2.2 m wide x 1.6 m high (Figure 22). The supporting steel work was protected using 15 mm thick fire board. A total of 15 hollowcore units were used, 1200 mm by 200 mm deep.

The two tests were identical except for the end restraint conditions to the hollowcore slabs. In the first test the slab units sat directly onto the supporting beams with the units notched around the columns. The joints between the units, and the gaps around the columns and the units, were filled with grout. In the second test, 2 T12 bars per unit were placed in the cores and around a 19 mm 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 filled with grout.

The applied load of 4.5 kN/m² was achieved using 60 sandbags (each weighing 1 t) evenly positioned over the floor plate, as shown in Figure 23. This gave an applied load of 4.71 kN/m². The self-weight of the units was 2.96 kN/m², creating a total load of 7.67 kN/m², and an applied moment at the time of the fire of 56.37 kNm per with of unit. This gave a load ratio of 0.34 for bending and 0.26 for shear capacity.

The natural fire was designed according to the British Standard BS EN 1991-1-2. Assuming the design for an office, the fire load density was 570 MJ/m² (80% fractile). The fire load was achieved using 40 standard (1m x 1m x 0.5m high) wooden cribs, comprising 50mm x 50 mm x 1000 mm wooden battens, positioned evenly around the compartment. The fire load was 33.25 kg of wood/m². The aim of the test was to try to follow the standard fire curve up to 60 minutes to investigate the structural behaviour and to enable the...
test to be compared against the structural performance in small-scale standard fire tests. Figure 24 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 (1069 °C in Test 1 and 1047 °C in Test 2).

All hollowcore units performed very well during the heating phase of the fire, which was more severe than the standard fire curve. The floor, as a whole, performed also well during the cooling phase of the fire. The different end 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.

There was evidence of 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 restrain 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 slipage. Further work is underway to investigate this beneficial behaviour to enable it to be utilised in practical design.

The hollowcore floor performed very well supporting the full applied static load for the duration of the tests. There was no significant spalling in any of the tests. 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 reinforce the experience gained from real fires that hollow core floor slabs have good overall inherent fire resistance.

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